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CENG
a joint venture of Constellation Energy · EDF

March 31, 2014

U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

ATTENTION: Document Control Desk

SUBJECT:
Calvert Cliffs Nuclear Power Plant, Units 1 and 2
Renewed Facility Operating License Nos. DPR-53 and DPR-69
Docket Nos. 50-317 and 50-318
R.E. Ginna Nuclear Power Plant
Renewed Facility Operating License No. DPR-18
Docket No. 50-244
Nine Mile Point Nuclear Station, Units 1 and 2
Renewed Facility Operating License Nos. DPR-63 and NPF-69
Docket Nos. 50-220 and 50-410

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

REFERENCES:

(a) Letter from E. J. Leeds (NRC) and M. R. Johnson (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, 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 (ML12053A340)

(b) Letter from A. R. Pietrangelo (NEI) to D. L. Skeen, (NRC), Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations, dated April 9, 2013 (ML13101A379)

(c) Letter from E. J. Leeds (NRC) to J. E. Pollock (NEI), 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 (ML13106A331)

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(d) 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 (ML12333A170) and February 2013 (Revision Requested by NRC in Reference (e))

(e) Letter from D. L. Skeen (NRC) to J. E. Pollock (NEI), Endorsement of EPRI Final Draft Report 1025287, "Seismic Evaluation Guidance," dated February 15, 2013 (ML12319A074)

(f) Letter from E. J. Leeds (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status on the Enclosed List, Supplemental Information Related to Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident, dated February 20, 2014 (ML14030A046)

(g) Letter from J. A. Spina (CENG) to NRC Document Control Desk, Response to Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Seismic Aspects of Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident – 1.5 Year Response for CEUS Sites, dated September 12, 2013 (ML13259A044)

(h) Letter from M. G. Korsnick (CENG) to NRC Document Control Desk, Updated Response to Request for Information Pursuant to 10 CFR 50.54(f) Regarding the Seismic Aspects of Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident – 1.5 Year Response for CEUS Sites, dated February 12, 2014 (ML14051A107)

(i) Letter from M. G. Korsnick (CENG) to NRC Document Control Desk, Supplement to Response to Request for Information: Near-Term Task Force Recommendation 2.1, Seismic Reevaluation, dated June 28, 2013 (ML13190A471)

(j) Letter from M. G. Evans (NRC) to M. Korsnick (CENG), Status of 60-Day Response to Issuance of Seismic Evaluation Guidance Related to the Near-Term Task Force Recommendation 2.1, Seismic, for Calvert Cliffs Nuclear Power Plant, Units 1 and 2; Nine Mile Point Nuclear Station, Units 1 and 2; and R.E. Ginna Nuclear Power Plant, dated August 2, 2013 (ML13204A422)

(k) Letter from K. A. Keithline (NEI) to D. L. Skeen (NRC), Relay Chatter Reviews for Seismic Hazard Screening, dated October 3, 2013 (ML13281A308)

(l) Letter from M. G. Korsnick (CENG) to NRC Document Control Desk, Response to Request for Information: Near-Term Task Force Recommendation 2.1, Seismic Reevaluation, dated April 26, 2013 (ML13120A105)

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

In Reference (b), 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 (c).

Reference (d) 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 (e).

Constellation Energy Nuclear Group, LLC (CENG) provides the Seismic Hazard Evaluation and Screening Reports for Calvert Cliffs Nuclear Power Plant, LLC (CCNPP), R.E. Ginna Nuclear Power Plant, LLC (Ginna), and Nine Mile Point Nuclear Station, LLC, (NMPNS) Units 1 (NMP1) and 2 (NMP2) in Attachments (1) through (3), respectively. These reports provide the information described in Section 4 of Reference (d) and Reference (f) in accordance with the schedule identified in Reference (b), and as committed to the letter dated September 12, 2013 (Reference g) that provided the description of the base case velocity profile and supporting subsurface materials and properties. Ginna submitted a revised description of the base case velocity profile and supporting subsurface materials and properties on February 12, 2014 (Reference h).

In Reference (i) CENG committed to submit Expedited Seismic Evaluation Process (ESEP) reports for the CENG sites/units by December 31, 2014. The following modifications to this commitment are made:

- As described in Attachment 1 and Reference (c), CCNPP screens into the ESEP, and will prepare a report summarizing the ESEP evaluation and results by December 31, 2014.
- As described in Attachments 2 and 3 and Reference (c), Ginna, NMP1, and NMP2 screen out of the ESEP and do not need to prepare reports summarizing the ESEP evaluations and results. Per the screening process, an ESEP Report will not be submitted for Ginna, NMP1, or NMP2. Since Ginna, NMP1 and NMP2 screen out based on the ESEP screening criteria, no ESEP modifications will be completed for Ginna, NMP1, and NMP2.

In Reference (i) CENG committed that:

- If CCNPP does not screen out based on the SPID screening criteria, CCNPP will complete the Seismic Risk Evaluation (with a Spent Fuel Pool (SFP) Evaluation and High Frequency Confirmation) by December 31, 2019. As described in Attachment 1 and Reference (d), CCNPP screens into performing a Seismic Risk Evaluation (with a SFP Evaluation and a High Frequency Confirmation). The commitment in Reference (i) is modified to state the CCNPP will perform a SFP Evaluation as determined by the NRC prioritization process following submittal of all nuclear power plant Seismic Hazard Reevaluations per the 50.54(f) letter (Reference j). In addition, CCNPP will perform a full scope relay review in accordance with the schedule provided in the letter from the industry to the NRC dated October 3, 2013 (Reference k).

- If Ginna does not screen out based on SPID screening criteria, Ginna will complete a Seismic Risk Evaluation (with a SFP Evaluation and High Frequency Confirmation) by June 30, 2017. As described in Attachment 2 and Reference (d), Ginna screens out of performing a Seismic Risk Evaluation and a SFP Evaluation. The commitment in Reference (i) is modified to state that Ginna will perform a High Frequency Confirmation as determined by NRC prioritization process following submittal of all nuclear power plant Seismic Hazard Reevaluations per the 50.54(f) letter (Reference j).

- If NMP1 or NMP2 do not screen out based on SPID screening criteria, NMP1 or NMP2 will complete a Seismic Risk Evaluation (with a SFP Evaluation and High Frequency Confirmation) by December 31, 2019. As described in Attachment 3 and Reference (d), NMP1 and NMP2 screen out of performing a Seismic Risk Evaluation and a SFP Evaluation. The commitment in Reference (i) is modified to state that NMP1 and NMP2 will perform a High Frequency Confirmation as determined by NRC prioritization process following submittal of all nuclear power plant Seismic Hazard Reevaluations per the 50.54(f) letter (Reference j).

As requested in Reference (j), this letter communicates changes to the schedules provided in the 60-day response letters (References i and l). Attachment 4 lists the regulatory commitments contained in this correspondence.

If there are any questions regarding this submittal, please contact Bruce Montgomery, Manager-Nuclear Safety and Security, at 443-532-6533.

I declare under penalty of perjury that the foregoing is true and correct. Executed on March 31, 2014.

Sincerely,

Mary G. Korsnick

Mary G. Korsnick

MGK/STD/EMT/bjd

Attachments: (1) Seismic Hazard and Screening Report in Response to the 50.54(f) Information Request regarding Fukushima Near-Term Task Force Recommendation 2.1: Seismic for Calvert Cliffs Nuclear Power Plant, Units 1 & 2

(2) Seismic Hazard and Screening Report in Response to the 50.54(f) Information Request regarding Fukushima Near-Term Task Force Recommendation 2.1: Seismic for R. E. Ginna Nuclear Power Plant

(3) Seismic Hazard and Screening Report in Response to the 50.54(f) Information Request regarding Fukushima Near-Term Task Force Recommendation 2.1: Seismic for Nine Mile Point Nuclear Station, Units 1 & 2

(4) Regulatory Commitments Contained in this Correspondence

cc: NRC Project Manager, Calvert Cliffs
NRC Project Manager, Ginna
NRC Project Manager, Nine Mile Point
W. M. Dean, NRC
Resident Inspector, Calvert Cliffs
Resident Inspector, Ginna
Resident Inspector, Nine Mile Point
S. Gray, DNR

ATTACHMENT (1)

SEISMIC HAZARD AND SCREENING REPORT IN RESPONSE TO THE 50.54(F) INFORMATION REQUEST REGARDING FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: SEISMIC FOR CALVERT CLIFFS NUCLEAR POWER PLANT, UNITS 1 & 2

Constellation Energy Nuclear Group, LLC
March 31, 2014

Executive Summary

PURPOSE

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the Nuclear Regulatory Commission (NRC) issued a 50.54(f) letter (Reference 1) requesting information in response to NRC Near-Term Task Force (NTTF) recommendations intended to clarify and strengthen the regulatory framework for protection against natural phenomena. The 50.54(f) letter requests that licensees and holders of construction permits under Title 10 Code of Federal Regulations Part 50 (Reference 2) reevaluate the seismic hazards at their sites using updated seismic hazard information and present-day regulatory guidance and methodologies. This report provides the information requested in items (1) through (7) of the "Requested Information" in Enclosure 1 of the 50.54(f) letter pertaining to NTTF Recommendation 2.1 for Calvert Cliffs Nuclear Power Plant (CCNPP) in accordance with the documented intention of Constellation Energy Nuclear Generating, LLC (CENG) transmitted to the NRC via letter dated April 26, 2013 (Reference 14).

SCOPE

In response to the 50.54(f) letter and following the Screening, Prioritization, and Implementation Details (SPID) industry guidance document (Reference 3), a seismic hazard reevaluation for CCNPP was performed to develop a Ground Motion Response Spectrum (GMRS) for comparison with the Safe Shutdown Earthquake (SSE) and the CCNPP Individual Plant Examination of External Events (IPEEE) high-confidence-of-low-probability-of-failure (HCLPF) Spectra. The new GMRS represents a beyond-design-basis seismic alternative demand developed by more modern techniques than were used for plant licensing. It does not constitute a change in the plant design or licensing basis.

Section 1 contains the report introduction. Section 2 provides a summary of the CCNPP regional and local geology, seismicity, other major inputs to the seismic hazard reevaluation, and detailed seismic hazard results including definition of the GMRS. Seismic hazard analysis for CCNPP, including the site response evaluation and GMRS development (Sections 2.2, 2.3, and 2.4 of this report) was performed by the Lettis Consultants International (LCI via EPRI) (Reference 13). A more in-depth discussion of the calculation methods used in the seismic hazard reevaluation is not included in this report but can be found in References 3, 7, 8, 10 and 18. Section 3 describes the characteristics of the appropriate plant-level SSE and IPEEE HCLPF Spectrum (IHS) for CCNPP. Section 4 provides a comparison of the GMRS to the SSE and IHS for CCNPP. Sections 5 and 6 discuss interim actions and conclusions, respectively, for CCNPP.

CONCLUSIONS

The screening evaluation comparison demonstrates that the GMRS exceeds the SSE but is not considerably higher than the SSE in the frequency range of 1 – 10 Hz. However, the IHS exceeds the GMRS in the frequency range of 1 – 10 Hz. Based on the comparison of the IHS and GMRS, a seismic risk evaluation is not required to be performed for CCNPP. CCNPP will perform a spent fuel pool evaluation since the GMRS exceeds the SSE in the frequency range of 1 – 10 Hz. Additionally, CCNPP will also perform the Expedited Seismic Evaluation Process (ESEP) interim action per the ESEP guidance. (Reference 4)

The GMRS exceeds the SSE in the frequency range beyond 10 Hz. However, the GMRS does not exceed the IHS in the frequency range beyond 10 Hz and therefore, additional high frequency confirmations are not required.

CCNPP is a focused scope IPEEE plant per NUREG-1407 (Reference 17). The SPID guidance requires a full scope relay review for plants which use the IHS for screening. The full scope relay review will be performed in accordance with the schedule provided in the letter from the industry to the NRC dated October 3, 2013 (Reference 15).

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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 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 the risk assessment results, 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 Calvert Cliffs Nuclear Power Plant Units 1 and 2 (CCNPP), located in Calvert County, Maryland. In providing this information, CENG 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. The Augmented Approach, Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic (ESEP), has been developed as the process for evaluating critical plant equipment as an interim action to demonstrate additional plant safety margin, prior to performing the complete plant seismic risk evaluations.

The original geologic and seismic siting investigations for the CCNPP were performed in accordance with Appendix A of Title 10 Code of Federal Regulations Part 100 (Reference 5) and meet General Design Criterion 2 in Appendix A of Reference 2. The Safe Shutdown Earthquake (SSE) ground motion was developed in accordance with Appendix A of Reference 5 and is used for the design of seismic Category I systems, structures and components. See Section 3 of this report for further discussion on the development of the SSE.

In response to the 50.54(f) letter and following the SPID guidance, a seismic hazard reevaluation for CCNPP was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed.

The GMRS does not exceed the IHS in the 1 to 10 Hz frequency range. Therefore, a seismic risk evaluation is not required at CCNPP. The GMRS does not exceed the IHS in the frequency range beyond 10 Hz. Therefore, additional high frequency confirmations are not required.

The GMRS exceeds the SSE spectrum in the 1 to 10 Hz frequency range. A spent fuel pool evaluation and the ESEP are required based on this screening. These evaluations will be conducted on the schedule for Central and Eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013 (Reference 6).

2 Seismic Hazard Reevaluation

CCNPP is located on the west shore of the Chesapeake Bay about three miles north of Lusby, MD in southeastern Calvert County, MD. The site is located approximately 50 miles east of the Fall Zone within the Coastal Plain Physiographic Province, which is characterized in general by low-lying gently rolling terrain. The site is underlain by about 2,500 ft of southeasterly dipping Cretaceous and Tertiary age sedimentary strata over crystalline and metamorphic rocks of Precambrian and Early Paleozoic age. (Section 2.4, Reference 11)

The CCNPP site and surrounding sedimentary strata of the Coastal Plain have remained essentially undeformed since they were deposited. There are no known faults within the Cretaceous and Tertiary sedimentary deposits or the deep crystalline rocks underlying the site. The nearest known faults are more than 50 miles west of the site located in Precambrian and Early Paleozoic rocks of the Piedmont Physiographic Province. Most earthquake activity in the region can be attributed to these faults. The minor earthquake activity in the Coastal Plain of Maryland may be due to concealed local projections of these faults beneath the Coastal Plain strata toward the general location of the CCNPP site. (Section 2.4, Reference 11)

The original investigation of historical seismic activity in the region determined a design safe shutdown earthquake (SSE) which is defined as the occurrence of a Modified Mercalli Intensity (MMI) of VII originating in the basement rock near the site. CCNPP determined this corresponds to horizontal and vertical design ground accelerations of 0.15 g and 0.10 g, respectively, at foundation level. (Section 2.6, Reference 11)

2.1 Regional and Local Geology

The CCNPP site is located on the west shore of the Chesapeake Bay within the Coastal Plain Physiographic Province. It is about 50 miles east of the Fall Zone which separates the low-lying gently rolling terrain of the Coastal Plain from the higher elevations of the Piedmont Physiographic Province. The Coastal Plain in Maryland rises from sea level to an elevation of about 250 ft and is characterized by a series of broad, step-like terraces. The average ground surface at the site is at an elevation of approximately 100 ft with the CCNPP located in an area where the preexisting ground elevation was about 65 ft and the final grade elevation is about 45 ft.

The Piedmont Physiographic Province consists of igneous and metamorphic rocks of Precambrian and Early Paleozoic age with areas of sedimentary and igneous rocks of Triassic age. These rocks continue under the Coastal Plain Province but are concealed by younger sedimentary strata of Cretaceous and Tertiary age. The younger sedimentary strata of the Coastal Plain Province form a wedge-shaped mass which thickens to the southeast and thins out to the northwest as it approaches the Fall Zone. At the location of the CCNPP site, there is approximately 250 ft of dense, relatively impervious sandy and clayey silt of Miocene age over dense, relatively pervious sand and silt of Eocene age. Precambrian and Early Paleozoic age bedrock is located about 2,500 ft beneath the site of CCNPP.

There is no known fault or geologic evidence of faulting in the underlying crystalline rock at the CCNPP site. This is supported by the absence of deformation in the overlying sedimentary strata of the Coastal Plain Province. The only observed deformation is about 40 miles south of the site, where there are local, very shallow folds in the Coastal Plain Province sediment. These folds are possibly a result of depositional conditions rather than tectonic activity. An examination of the strata exposed along the Chesapeake Bay shoreline shows no signs of faulting or deformation. The nearest known faults are located more than 50 miles west in the Precambrian and Early Paleozoic age rocks of the Piedmont Physiographic Province (west of the Fall Zone), with the closest being 60 miles southwest of the CCNPP site. Minor earthquake activity in the Coastal Plain of Maryland could theoretically be caused by concealed faults that have projected from the Piedmont Province under the Costal Plain sedimentary strata. However, these theorized faults would be local rather than regionally continuous, and as stated above, there is no evidence of faulting near the CCNPP site. (Section 2.4, Reference 11)

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, 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 (Reference 7) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (Reference 8). For the PSHA, a lower-bound moment magnitude of 5.0 was used, as specified in the 50.54(f) letter.

For the PSHA, the CEUS-SSC background seismic sources within a distance of 400 miles (640 km) around Calvert Cliffs were included. This distance exceeds the 200 mile (320 km) recommendation contained in USNRC Reg. Guide 1.208 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. Mesozoic and younger extended prior – narrow (MESE-N)
4. Mesozoic and younger extended prior – wide (MESE-W)
5. Midcontinent-Craton alternative A (MIDC_A)
6. Midcontinent-Craton alternative B (MIDC_B)
7. Midcontinent-Craton alternative C (MIDC_C)
8. Midcontinent-Craton alternative D (MIDC_D)
9. Northern Appalachians (NAP)
10. Non-Mesozoic and younger extended prior – narrow (NMESE-N)
11. Non-Mesozoic and younger extended prior – wide (NMESE-W)
12. Paleozoic Extended Crust narrow (PEZ_N)
13. Paleozoic Extended Crust wide (PEZ_W)
14. St. Lawrence Rift, including the Ottawa and Saquenay grabens (SLR)
15. Study region (STUDY_R)

For sources of large magnitude earthquakes, designated Repeated Large Magnitude Earthquake (RLME) sources in CEUS-SSC, the following sources lie within 621 miles (1000 km) of the site and were included in the analysis:

1. Charleston
2. 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, 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 in Section 2.3.7 at the SSE control point elevation.

2.3 SITE RESPONSE EVALUATION

Following the guidance contained in Enclosure 1 of the 50.54(f) Request for Information and in the SPID for nuclear power plant sites that are not sited on hard rock (defined as 9285 ft/s), a site response analysis was performed for CCNPP.

2.3.1 Description of Subsurface Material

The CCNPP site consists of about 245 ft of dense sands and clay/silt of the Chesapeake Formation. Beneath this formation is over 2200 ft of Eocene and older sands and deeper soil transition layers that overlie about 20 ft of a bedrock transition zone. Bedrock is estimated to be at a depth of about 2511 ft.

CCNPP consists of two units with the containment buildings supported on Stratum II-B: Chesapeake Cemented Sand (Reference 16). Table 2.3.1-1 shows the geotechnical properties for the site.

Table 2.3.1-1: Summary of Geotechnical Profile Data for GMRS at CCNPP (Ref. 16)

Depth Range [(1)] (feet)	Soil/Rock Description	Density [(2)] (pcf)	Shear Wave Velocity [(3)] (fps)	Compressional Wave Velocity [(3)] (fps)	Poisson's Ratio
0 - 17	Stratum II-A Chesapeake Clay/Silt	140	800	2283	0.43
17 - 65	Stratum II-B Chesapeake Cemented Sand	164	1600	4566	0.43
65 - 245	Stratum II-C Chesapeake Clay/Silt	164	1250	2752	0.37
245 - 265	Stratum III Nanjemoy Sand 1	125	1790	5937	0.45
265 - 275	Stratum III Nanjemoy Sand 2	125	2330	6274	0.42
275 - 315	Stratum III Nanjemoy Sand 3	125	2030	5793	0.43
315 - 362	Stratum III Nanjemoy Sand 4	125	1930	5896	0.44
362 - 1045	Deep Soil Transition 1	115	2200	5389	0.40
1045 - 1545	Deep Soil Transition 2	115	2330	5707	0.40
1545 - 2045	Deep Soil Transition 3	115	2550	6246	0.40
2045 - 2491	Deep Soil Transition 4	115	2800	6859	0.40
2491 - 2501	Bedrock Transition 1	162	5000	9354	0.30
2501 - 2511	Bedrock Transition 2	162	7000	13096	0.30
2511+	Bedrock	162	9200	17212	0.30

NOTES:
(1) Measured from plant grade (El. 45 ft)
(2) Based on the proposed Calvert Cliffs Unit 3 (CC3) Combined License Application (COLA) investigation
(3) Determined from Uphole measurements and CC3 Investigation
* SSE control point is located at elevation -1 ft (depth 46 ft) within Stratum II-B

The soils at the site can be divided into the following stratigraphic units (Reference 16):

- Stratum I: Terrace Sand – light brown to brown sand with varying amounts of silt, clay, and/or gravel, sometimes with silt or clay interbedded layers.
- Stratum IIa: Chesapeake Clay/Silt – light to dark gray clay and/or silt, predominantly clay, with varying amounts of sand.
- Stratum IIb: Chesapeake Cemented Sand – interbedded layers of light to dark gray silty/clayey sands, sandy silts, and low to high plasticity clays, with varying amounts of shell fragments and with varying degrees of cementation. At the CC3 site Stratum IIb was further divided into three sub-layers with variation in Standard Penetration Test (SPT) and VS values. Stratum IIb is the bearing material for both CC1&2 and CC3.
- Stratum IIc: Chesapeake Clay/Silt – gray to greenish gray clay/silt soils, they contain interbedded layers of sandy silt, silty sand, and cemented sands with varying amount of shell fragments.
- Stratum III: Nanjemoy Sand – primarily dark greenish-gray glauconitic sand with interbedded layers of silt, clay, and cemented sands with varying amounts of shell fragments and varying degrees of cementation.

Both operating units at Calvert Cliffs are resting directly over Stratum II-B: Chesapeake Cemented Sand. A detailed subdivision of Stratum II-B was introduced during the CC3 COLA investigation. In the CCNPP 1&2 UFSAR, Stratums I and II are referred to as the Chesapeake group soils. (Reference 16)

Deeper geologic units are identified in the CC3 FSAR and were required to estimate the depth at which the shear wave velocity has a value of 9,200 fps. According to Section 2.5.2.5.1.3 of the CC3 FSAR, the shear-wave velocity of 9,200 fps (for bedrock) is estimated at a depth of approximately 2531 ft. As described in the CC3 FSAR, various geologic records were reviewed and communication made with staff at the Maryland Geological Survey, the United States Geological Survey, and the Triassic-Jurassic Study Group of Lamont-Doherty Earth Observatory, Columbia University. Soils below 400 ft consist of Coastal Plain sediments of Eocene, Paleocene, and Cretaceous eras, extending to an estimated depth of about 2,500 ft below the ground surface. These soils contain sequences of sand, silt, and clay. Given their geologic age, they are expected to be competent soils, consolidated to at least the weight of the overlying soils. (Reference 16)

Several available geologic records were also reviewed in order to obtain information on both the depth to bedrock and the bedrock type. Accordingly, the estimated depth to bedrock in the proximity of the site was set to 2,555 ft, which is consistent with the depth of 2,500 ft reported in the CCNPP UFSAR. (Reference 16)

At CC1&2, plant grade is at El. 45. Consistent with the recommendation of the CC3 FSAR, it is recommended, for the CC1&2 site amplification analyses, to place 9,200 fps bedrock at an elevation of El. -2466. (Reference 16)

2.3.2 Development of Base Case Profiles and Nonlinear Material Properties

Table 2.3.1-1 shows the recommended shear-wave velocities and unit weights along with elevations and corresponding stratigraphy. The SSE control point is at elevation -1 ft within Stratum II-B, the firm cemented sands of the Chesapeake Formation with a measured average shear-wave velocity of 1,600 ft/s (487.7 m/s). The source for deeper velocity estimates is listed as uphole measurements and CC3 investigation (Table 2.3.1-1). Depth to Crystalline basement of about 2,465 ft (751.3 m) was estimated (Table 2.3.1-1), likely based on regional geology or available nearby deep boreholes. (Reference 13)

The shear-wave velocity in the top 20 ft of the Stratum II – B (below the SSE elevation) was measured at the CC1&2 site at 1,600 ft/s (487.7 m/s) and confirmed by recent measurements at the nearby CC3 site between 1,400 ft/s (426.7 m/s) and 1,800 ft/s (548.6m/s). Below that depth (Stratum II-C and deeper) shear-wave velocities were based on both measurements and very likely estimates for deeper portions at the nearby CC3 site. To accommodate epistemic uncertainty in shear-wave velocities a scale factor of 1.57 was assumed appropriate for the materials adopted from CC3. For the top 20 ft of the Chesapeake, measured at the CC1&2 sites, the range of about 10% based upon CC3 measurements was considered too small to specifically incorporate. The epistemic uncertainty taken over the roughly 2,500 ft (762m) of the profile was considered to reflect an adequate range in amplification. The scale factor of 1.57 reflects a σ_{ln} of about 0.35, based on the SPID 10th and 90th fractiles which implies a 1.28 scale factor on σ_{μ}. (Reference 13)

Using the shear-wave velocities specified in Table 2.3.1-1 three base-profiles were developed using the scale factors of 1.00 for the top 20 ft of the Chesapeake sands and 1.57 for the deeper layers (soil and bedrock transition layers 1 and 2). (Reference 13)

The specified shear-wave velocities were taken as the mean or best estimate base-case profile (P1) with lower and upper range base-cases profiles P2 and P3, respectively. Profiles extended to a depth (below the SSE) of 2,465 ft (751 m), randomized ± 740 ft (± 225 m). The upper-range profile P3 encountered hard rock shear-wave velocity of 9,285 ft/s (2,890 m/s) at a depth below the SSE of about 2,455 ft (748 m). The base-case profiles (P1, P2, and P3) are shown in Figure 2.3.2-1 and listed in Table 2.3.2-1. The upper-range profile P3 encountered hard rock shear-wave velocities (9,285 ft/s, 2,890 m/s) at a depth of about 3,455 ft (748 m). The depth randomization reflects ± 30% of the depth and was included to provide a realistic broadening of the fundamental resonance at deep sites rather than reflect actual random variations to basement shear-wave velocities across a footprint. (Reference 13)

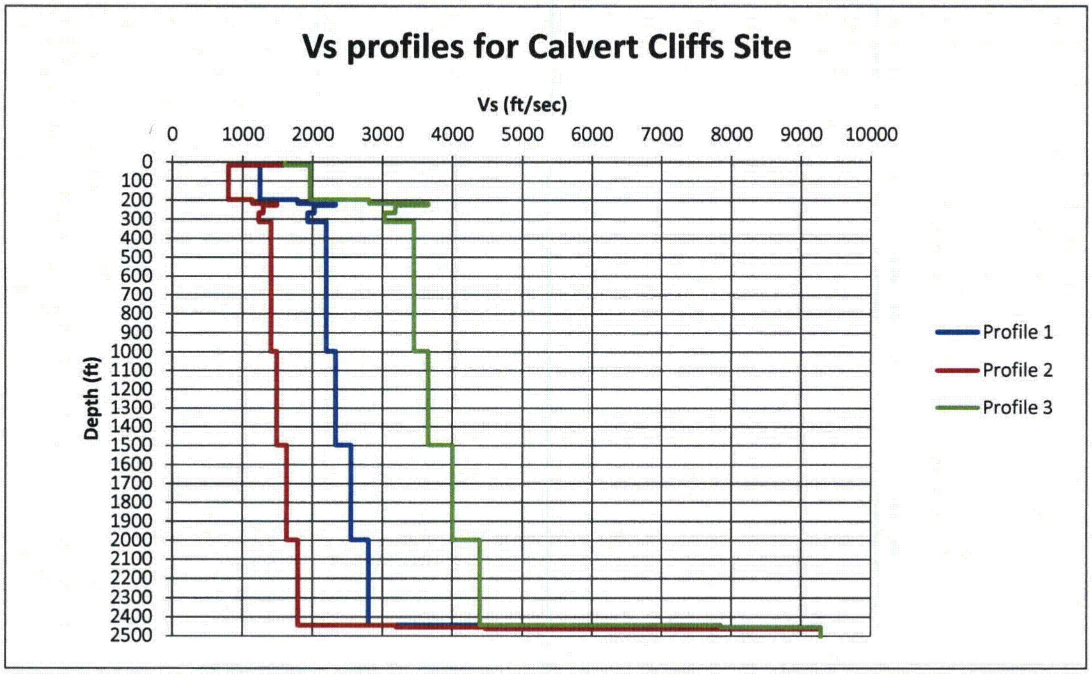


Figure 2.3.2-1: Shear-wave velocity (Vs) profiles for CCNPP (Ref. 13)

Table 2.3.2-1: Layer thicknesses, depths, and shear-wave velocities (Vs) for 3 profiles at CCNPP (Ref. 13)

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	1600		0	1600		0	1600
3.8	3.8	1600	3.8	3.8	1600	3.8	3.8	1600
3.8	7.6	1600	3.8	7.6	1600	3.8	7.6	1600
3.8	11.4	1600	3.8	11.4	1600	3.8	11.4	1600
3.8	15.2	1600	3.8	15.2	1600	3.8	15.2	1600
3.8	19.0	1600	3.8	19.0	1600	3.8	19.0	1600
1.0	20.0	1250	1.0	20.0	800	1.0	20.0	1962
8.0	28.0	1250	8.0	28.0	800	8.0	28.0	1962
8.0	35.9	1250	8.0	35.9	800	8.0	35.9	1962
8.0	43.9	1250	8.0	43.9	800	8.0	43.9	1962
8.0	51.8	1250	8.0	51.8	800	8.0	51.8	1962
8.0	59.8	1250	8.0	59.8	800	8.0	59.8	1962
8.0	67.7	1250	8.0	67.7	800	8.0	67.7	1962
8.0	75.7	1250	8.0	75.7	800	8.0	75.7	1962
8.0	83.6	1250	8.0	83.6	800	8.0	83.6	1962
8.0	91.6	1250	8.0	91.6	800	8.0	91.6	1962
8.0	99.5	1250	8.0	99.5	800	8.0	99.5	1962
8.0	107.5	1250	8.0	107.5	800	8.0	107.5	1962
8.0	115.5	1250	8.0	115.5	800	8.0	115.5	1962
4.0	119.4	1250	4.0	119.4	800	4.0	119.4	1962
8.0	127.4	1250	8.0	127.4	800	8.0	127.4	1962
8.0	135.3	1250	8.0	135.3	800	8.0	135.3	1962
8.0	143.3	1250	8.0	143.3	800	8.0	143.3	1962
8.0	151.3	1250	8.0	151.3	800	8.0	151.3	1962
8.0	159.2	1250	8.0	159.2	800	8.0	159.2	1962
8.0	167.2	1250	8.0	167.2	800	8.0	167.2	1962
8.0	175.1	1250	8.0	175.1	800	8.0	175.1	1962
8.0	183.1	1250	8.0	183.1	800	8.0	183.1	1962
8.0	191.0	1250	8.0	191.0	800	8.0	191.0	1962
8.0	199.0	1250	8.0	199.0	800	8.0	199.0	1962
6.7	205.6	1790	6.7	205.6	1146	6.7	205.6	2810
6.7	212.3	1790	6.7	212.3	1146	6.7	212.3	2810
6.7	219.0	1790	6.7	219.0	1146	6.7	219.0	2810
10.0	229.0	2330	10.0	229.0	1491	10.0	229.0	3658
10.0	239.0	2030	10.0	239.0	1299	10.0	239.0	3187
10.0	249.0	2030	10.0	249.0	1299	10.0	249.0	3187
10.0	259.0	2030	10.0	259.0	1299	10.0	259.0	3187
10.0	269.0	2030	10.0	269.0	1299	10.0	269.0	3187
9.4	278.4	1930	9.4	278.4	1235	9.4	278.4	3030
9.4	287.8	1930	9.4	287.8	1235	9.4	287.8	3030

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)
9.4	297.2	1930	9.4	297.2	1235	9.4	297.2	3030
9.4	306.6	1930	9.4	306.6	1235	9.4	306.6	3030
9.4	316.0	1930	9.4	316.0	1235	9.4	316.0	3030
11.4	327.4	2200	11.4	327.4	1408	11.4	327.4	3454
11.4	338.7	2200	11.4	338.7	1408	11.4	338.7	3454
11.4	350.1	2200	11.4	350.1	1408	11.4	350.1	3454
11.4	361.5	2200	11.4	361.5	1408	11.4	361.5	3454
11.4	372.9	2200	11.4	372.9	1408	11.4	372.9	3454
11.4	384.3	2200	11.4	384.3	1408	11.4	384.3	3454
11.4	395.7	2200	11.4	395.7	1408	11.4	395.7	3454
11.4	407.0	2200	11.4	407.0	1408	11.4	407.0	3454
11.4	418.4	2200	11.4	418.4	1408	11.4	418.4	3454
11.4	429.8	2200	11.4	429.8	1408	11.4	429.8	3454
11.4	441.2	2200	11.4	441.2	1408	11.4	441.2	3454
11.4	452.6	2200	11.4	452.6	1408	11.4	452.6	3454
11.4	464.0	2200	11.4	464.0	1408	11.4	464.0	3454
11.4	475.3	2200	11.4	475.3	1408	11.4	475.3	3454
11.4	486.7	2200	11.4	486.7	1408	11.4	486.7	3454
11.4	498.1	2200	11.4	498.1	1408	11.4	498.1	3454
166.9	665.0	2200	166.9	665.0	1408	166.9	665.0	3454
166.9	832.0	2200	166.9	832.0	1408	166.9	832.0	3454
166.9	998.9	2200	166.9	998.9	1408	166.9	998.9	3454
250.0	1248.9	2330	250.0	1248.9	1491	250.0	1248.9	3658
250.0	1498.9	2330	250.0	1498.9	1491	250.0	1498.9	3658
200.0	1698.9	2550	200.0	1698.9	1632	200.0	1698.9	4003
300.0	1998.9	2550	300.0	1998.9	1632	300.0	1998.9	4003
198.2	2197.1	2800	198.2	2197.1	1792	198.2	2197.1	4396
247.8	2444.9	2800	247.8	2444.9	1792	247.8	2444.9	4396
10.0	2454.9	5000	10.0	2454.9	3200	10.0	2454.9	7850
10.0	2464.9	7000	10.0	2464.9	4480	10.0	2464.9	9285
3280.8	5745.7	9285	3280.8	5745.7	9285	3280.8	5745.7	9285

2.3.2.1 Shear Modulus and Damping Curves

No site-specific nonlinear dynamic material properties were determined in the initial siting of the CCNPP for soils or bedrock transition layers. The firm soil material over the upper 500 ft (152 m) was assumed to have behavior that could be modeled with either EPRI cohesionless soil or Peninsular Range G/G_{max} and hysteretic damping curves (Reference 3). Consistent with the SPID, the EPRI soil curves (model M1) were considered to be appropriate to represent the more nonlinear response likely to occur in the materials at this site. The Peninsular Range (PR) curves (Reference 3) for soils (model M2) were assumed to represent an equally plausible alternative more linear response across loading level. For the firm rock analyses, Q_s of 40 was used as the constant damping values for firm rock since they were at a depth below 500 ft. (Reference 13)

2.3.2.2 Kappa

Base-case kappa estimates were determined using Section B-5.1.3.1 of the SPID for a shallow (< 3000 ft (1000 m)) CEUS soil site. Kappa for a soil site with less than 3,000 ft (1 km) of soil may be estimated from the Campbell's relationship in Section B-5.1.3.1 of the SPID. For the Calvert Cliffs site, with about 2,445 ft (745 m) of soil and about 20 ft (6 m) of firm rock, the kappa estimate from this relationship is 0.051s. Consistent with the SPID a maximum base-case kappa of 0.04s was assumed (Table 2.3.2-2). Epistemic uncertainty in profile damping (kappa) was considered to be accommodated at design loading levels by the multiple (2) sets of G/G_{max} and hysteretic damping curves. (Reference 13)

Table 2.3.2-2: Kappa Values and Weights Used for Site Response Analyses (Ref. 13)

Velocity Profile	Kappa(s)
P1	0.040
P2	0.040
P3	0.040
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[1] shear-wave velocity profiles has been incorporated in the site response calculations. For the CCNPP 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, 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 (Reference 10) 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, correlation of shear wave velocity between layers was modeled using the footprint correlation model. In the correlation model, a limit of +/- 2 standard

[1] Assumptions discussed in Section 2 are engineering judgments made by EPRI engineers in accordance with implementation of the SPID methodology.

deviations about the median value in each layer was assumed[1] for the limits on random velocity fluctuations. (Reference 13)

2.3.4 Input Spectra

Consistent with the guidance in Appendix B of the SPID, 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 1.5 g) were used in the site response analyses. The characteristics of the seismic source and upper crustal attenuation properties assumed[1] for the analysis of the Calvert Cliffs site were the same as those identified in Tables B-4, B-5, B-6 and B-7 of the SPID as appropriate for typical CEUS sites. (Reference 13)

2.3.5 Methodology

To perform the site response analyses for the CCNPP 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. The guidance contained in Appendix B of the SPID 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 CCNPP site. (Reference 13)

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 Appendix B of 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 soil 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 CCNPP soil site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2). Figures 2.3.6-1 and Figure 2.3.6-2 respectively show only a minor difference for the 0.4g loading level and below. Above about the 0.4g loading level, the differences increase mainly in frequencies between about 1 and 20 Hz. (Reference 13)

[1] Assumptions discussed in Section 2 are engineering judgments made by EPRI engineers in accordance with implementation of the SPID methodology.

Tabulated values of the amplification factors are provided in Tables A2-b1 and A2-b2 in the attached Appendix.

Figure 2.3.6.1

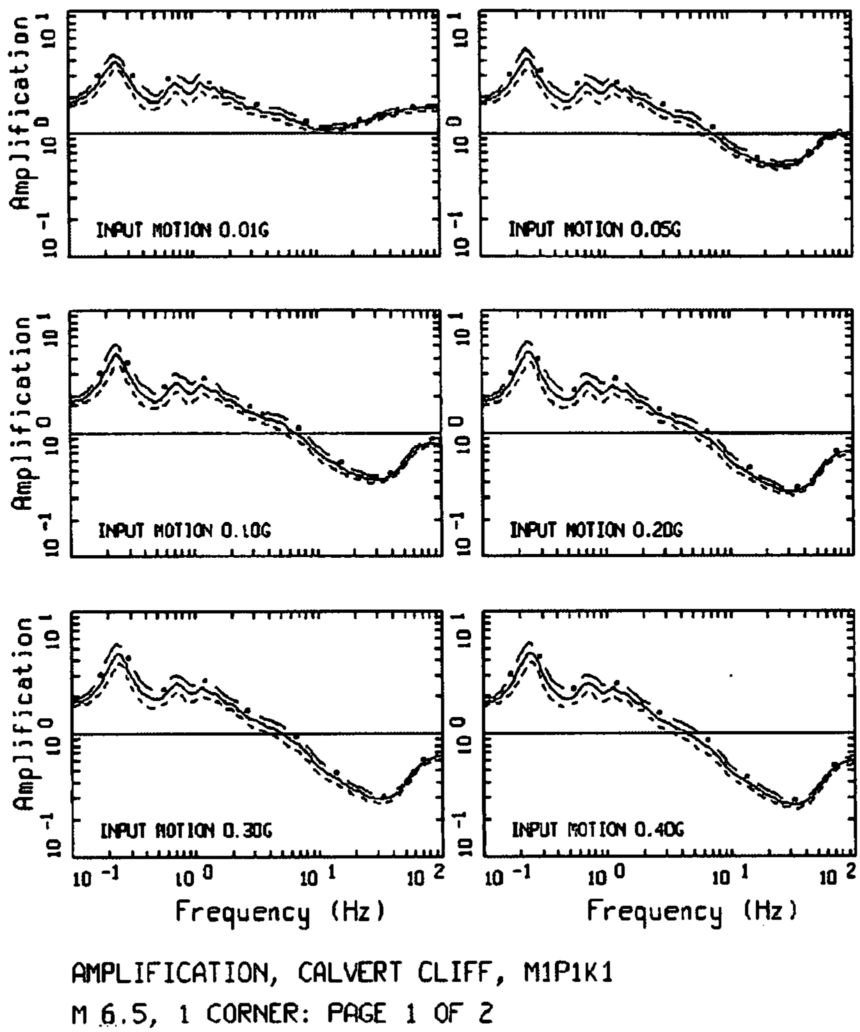
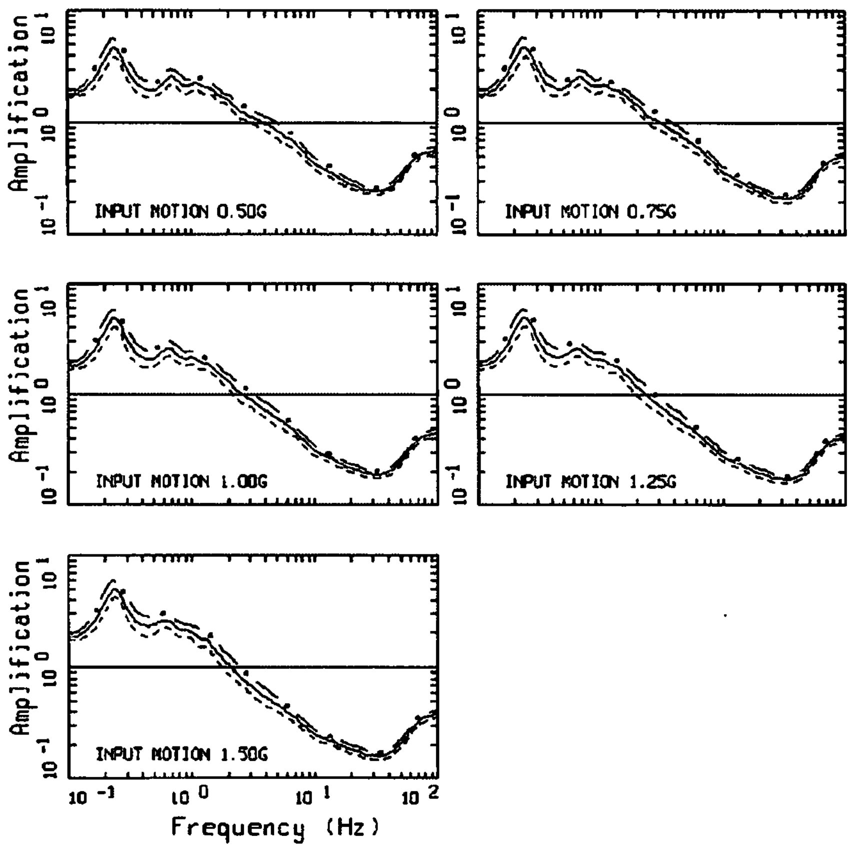


Figure 2.3.6-1: Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), EPRI soil and rock modulus reduction and hysteretic damping curves (model M1), and base-case kappa at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g. **M** 6.5 and single-corner source model (Ref. 3)

Figure 2.3.6-1 (cont'd)



AMPLIFICATION, CALVERT CLIFF, M1P1K1
M 6.5, 1 CORNER: PAGE 2 OF 2

Figure 2.3.6-2

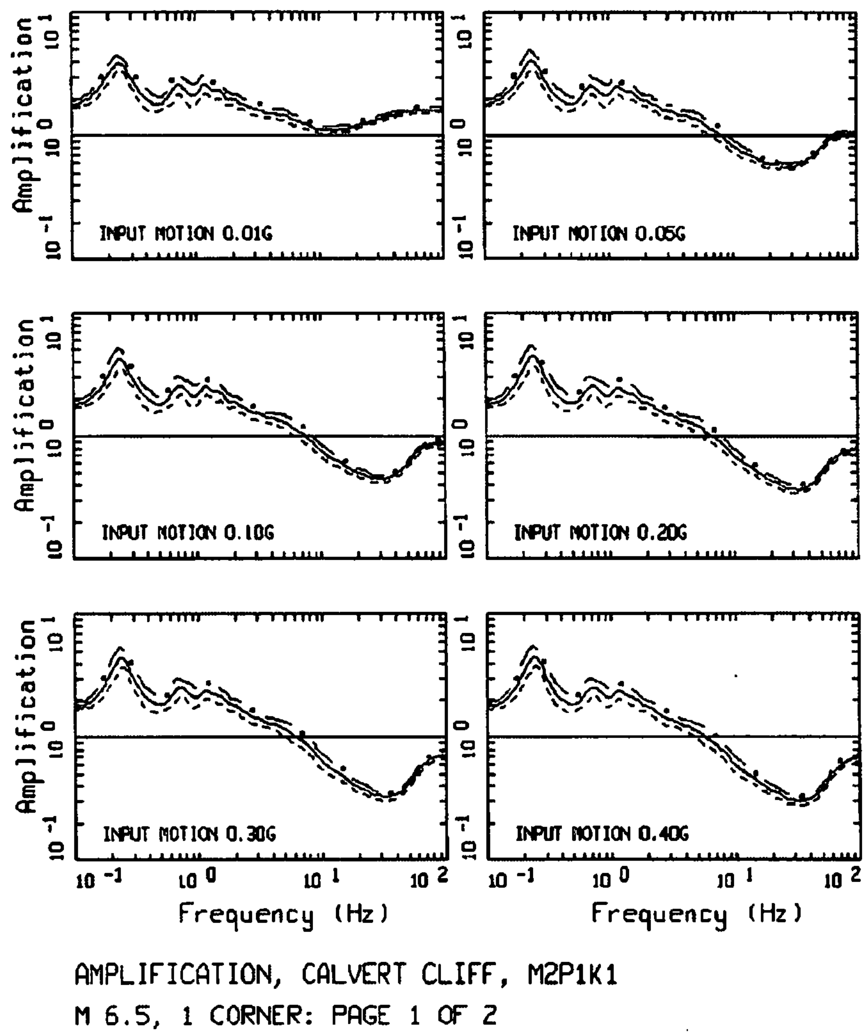
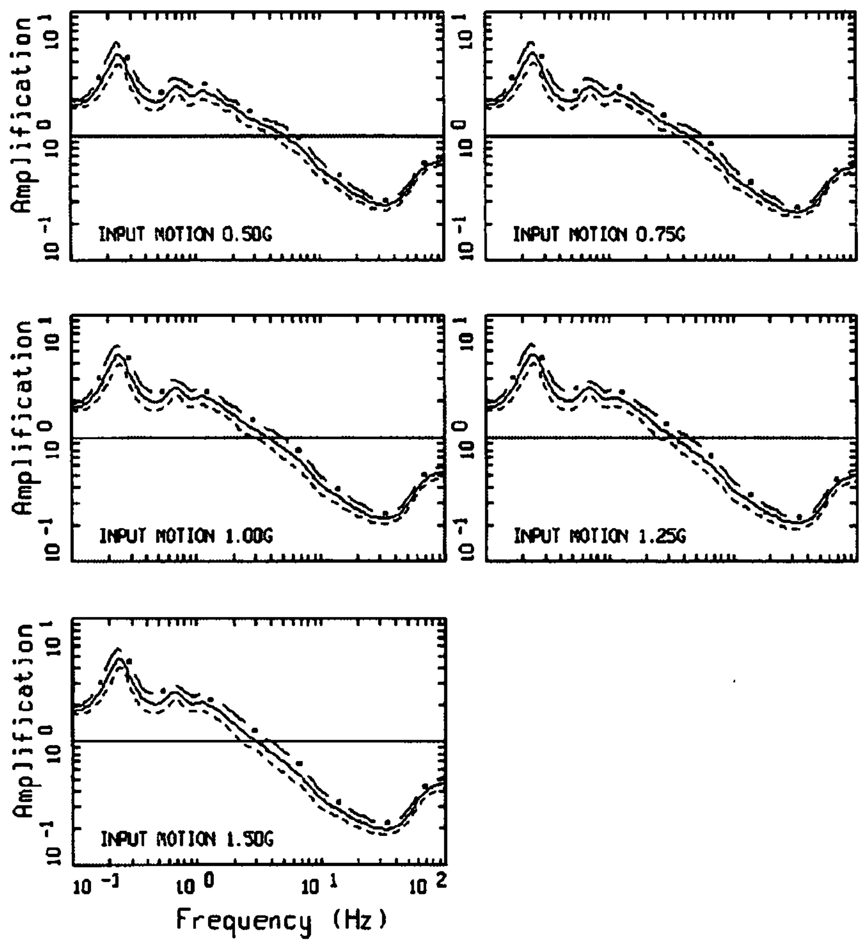


Figure 2.3.6-2: Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), Peninsular Range modulus reduction and hysteretic damping curves for soil and linear site response for rock (model M2), and base-case kappa at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g. **M** 6.5 and single-corner source model (Ref. 3)

Figure 2.3.6-2 (cont.)



AMPLIFICATION, CALVERT CLIFF, M2P1K1

M 6.5, 1 CORNER: PAGE 2 OF 2

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. 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 Calvert Cliffs are shown in Figure 2.3.7-1 for the seven spectral frequencies for which GMM is defined. Tabulated values of mean and fractile seismic hazard curves and site response amplification functions are provided in Appendix A.

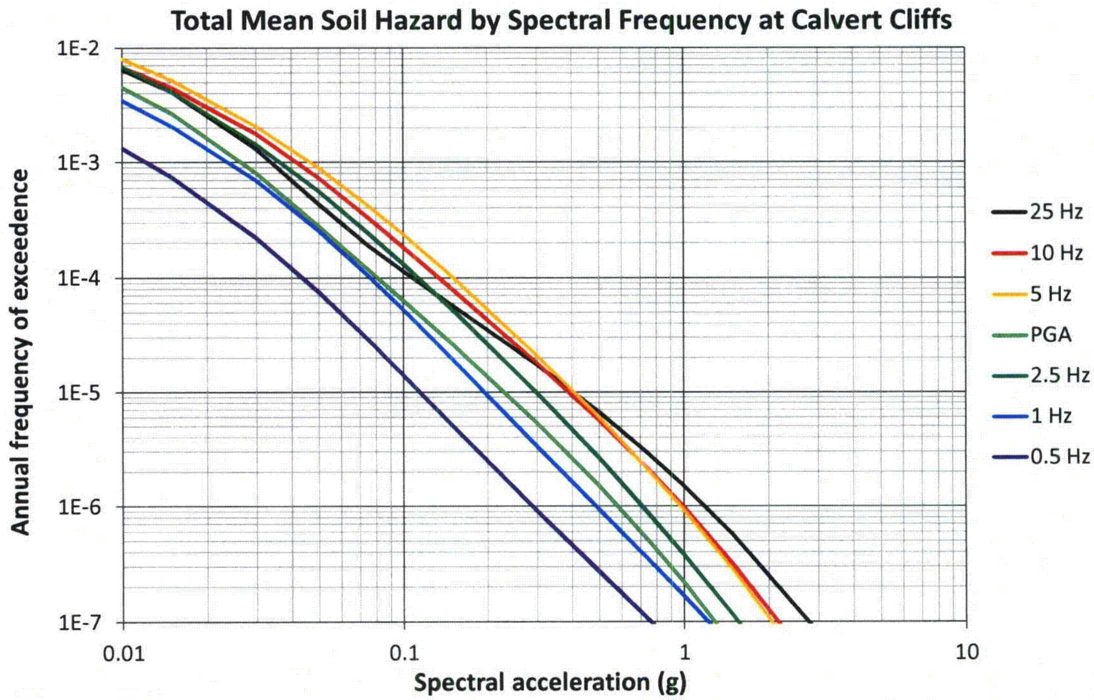


Figure 2.3.7-1: Control point mean hazard curves for spectral frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz (PGA) at Calvert Cliffs (Ref. 13)

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 spectral 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 spectral accelerations.

Table 2.4-1: UHRS for 1E-4 and 1E-5 and GMRS at control point for CCNPP (Ref. 13)

Freq. (Hz)	10^{-4} UHRS (g)	10^{-5} UHRS (g)	GMRS (g)
100	8.11E-02	2.29E-01	1.12E-01
90	8.23E-02	2.38E-01	1.15E-01
80	8.36E-02	2.47E-01	1.19E-01
70	8.52E-02	2.59E-01	1.24E-01
60	8.71E-02	2.73E-01	1.30E-01
50	8.95E-02	2.92E-01	1.38E-01
40	9.33E-02	3.18E-01	1.49E-01
35	9.63E-02	3.37E-01	1.57E-01
30	1.01E-01	3.63E-01	1.68E-01
25	1.08E-01	4.02E-01	1.85E-01
20	1.10E-01	3.86E-01	1.80E-01
15	1.19E-01	3.84E-01	1.82E-01
12.5	1.25E-01	3.94E-01	1.88E-01
10	1.34E-01	3.88E-01	1.88E-01
9	1.39E-01	4.00E-01	1.94E-01
8	1.45E-01	4.11E-01	2.00E-01
7	1.49E-01	4.19E-01	2.04E-01
6	1.49E-01	4.14E-01	2.03E-01
5	1.51E-01	4.05E-01	2.00E-01
4	1.39E-01	3.75E-01	1.85E-01
3.5	1.32E-01	3.57E-01	1.76E-01
3	1.24E-01	3.33E-01	1.64E-01
2.5	1.13E-01	2.99E-01	1.48E-01
2	1.09E-01	2.90E-01	1.43E-01
1.5	9.90E-02	2.59E-01	1.28E-01
1.25	9.23E-02	2.35E-01	1.17E-01
1	7.67E-02	1.95E-01	9.70E-02
0.9	6.84E-02	1.75E-01	8.70E-02
0.8	6.52E-02	1.66E-01	8.27E-02
0.7	6.18E-02	1.60E-01	7.95E-02
0.6	4.93E-02	1.32E-01	6.50E-02
0.5	4.38E-02	1.15E-01	5.68E-02
0.4	3.50E-02	9.18E-02	4.54E-02
0.35	3.06E-02	8.04E-02	3.98E-02
0.3	2.63E-02	6.89E-02	3.41E-02
0.25	2.19E-02	5.74E-02	2.84E-02
0.2	1.75E-02	4.59E-02	2.27E-02
0.15	1.31E-02	3.44E-02	1.70E-02
0.125	1.09E-02	2.87E-02	1.42E-02
0.1	8.75E-03	2.30E-02	1.14E-02

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.

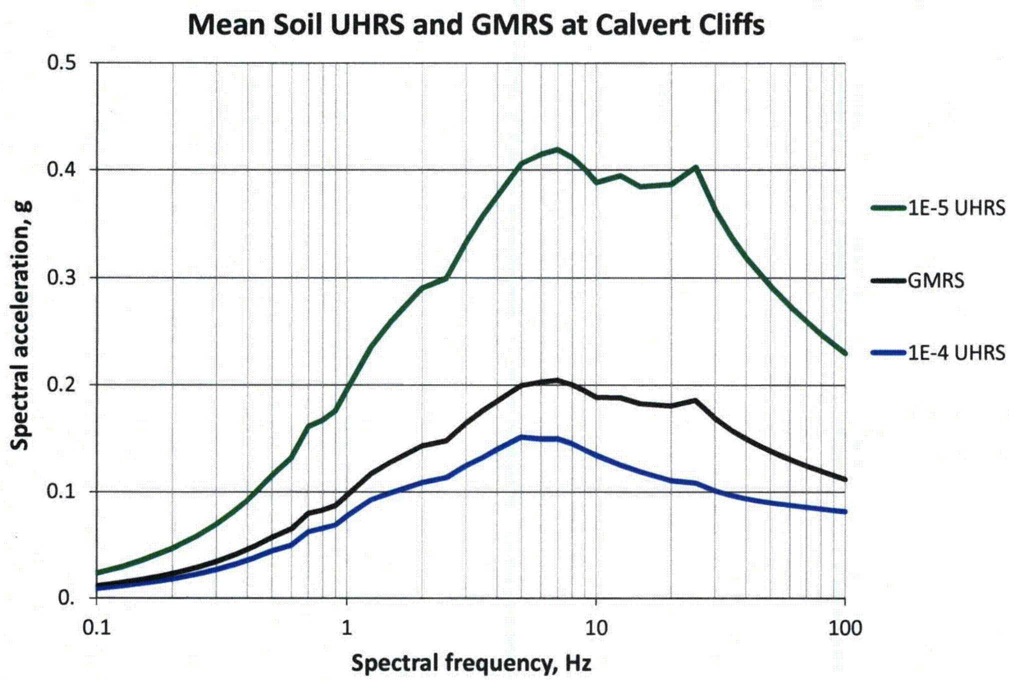


Figure 2.4-1: UHRS for 1E-4 and 1E-5 and GMRS at control point for CCNPP (Ref. 13)

3

Plant Design Basis [and Beyond Design Basis Evaluation Ground Motion]

The design basis earthquake for CCNPP is identified in Section 2.6.5 of the UFSAR (Reference 11).

An evaluation for beyond design basis (BDB) ground motions was performed in the Individual Plant Examination of External Events (IPEEE). The IPEEE plant level HCLPF response spectrum is included below for screening purposes.

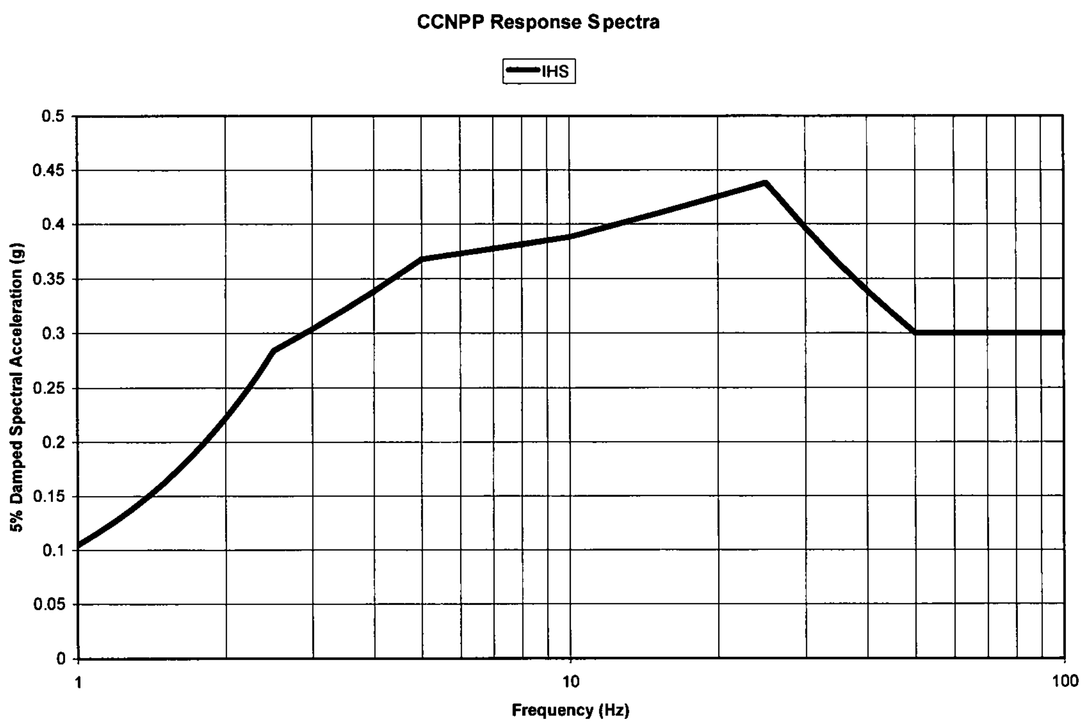


Figure 3.0-1: IHS Response Spectrum for CCNPP

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 recommended SSE was conservatively defined as the occurrence of a Modified Mercalli Intensity of VII (Magnitude 5 to 5.5) originating in the basement rock near the site. This SSE design earthquake Intensity was chosen based on the uncertainty in the geologic cause of the 1927 New Jersey earthquake and the epicentral location of the 1871 Wilmington, DE earthquake (Reference 11).

A 1992 independent evaluation of the region seismicity performed for the Diesel Generator Project identified more recent earthquakes than those considered in the original development of the SSE. However, none of these earthquakes were larger or in significantly different areas from those used to develop the SSE basis. (Section 2.6, Reference 11)

The SSE is defined in terms of a PGA and a design response spectrum. For seismic hazard screening purposes, the SSE is defined in Section 2.6.5.4 of the Calvert Cliffs UFSAR with a horizontal PGA of 0.15 g. The 5% damped horizontal SSE for CCNPP is compared with the GMRS in the screening evaluation in Section 4. Table 3.1-1 and Figure 3.1-1 show the spectral acceleration values as a function of frequency for the 5% damped horizontal SSE.

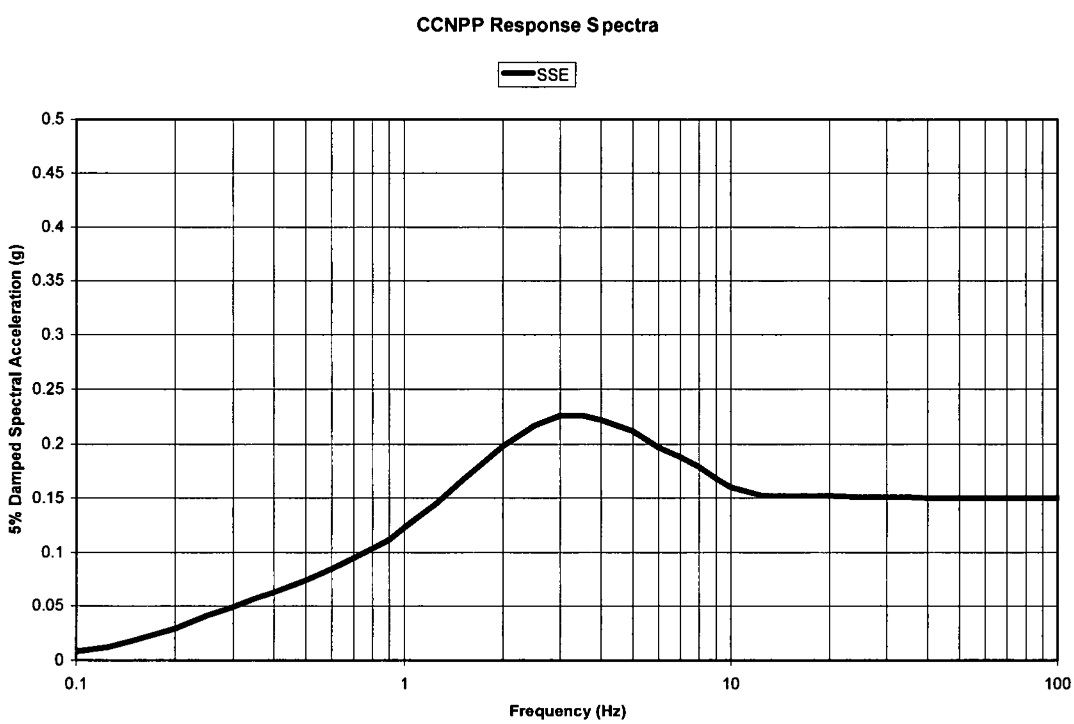


Figure 3.1-1: SSE Response Spectrum for CCNPP

Table 3.1-1: Horizontal Safe Shutdown Earthquake response spectrum for CCNPP

Freq. (Hz)	SSE (g)
0.1	0.008
0.125	0.012
0.15	0.018
0.2	0.029
0.25	0.041
0.3	0.049
0.35	0.057
0.4	0.063
0.5	0.074
0.6	0.085
0.7	0.095
0.8	0.104
0.9	0.112
1	0.123
1.25	0.145
1.5	0.166
2	0.198
2.5	0.217
3	0.226
3.5	0.226
4	0.222
5	0.212
6	0.197
7	0.188
8	0.179
9	0.168
10	0.160
12.5	0.152
15	0.152
20	0.152
25	0.151
30	0.151
35	0.151
40	0.150
50	0.150
60	0.150
70	0.150
80	0.150
90	0.150
100	0.150

3.2 CONTROL POINT ELEVATION

The control point elevations for Unit 1 and Unit 2 of the CCNPP are not stated in the UFSAR. In accordance with Section 2.4.2 of the SPID, for a soil site with generally uniform, horizontally layered stratigraphy and where the key structures are soil-founded, the SSE control point is defined as the highest point where a safety-related structure is founded. This control point definition is applied to the main power block area at the site. Per Section 2.7.6.2 of the UFSAR, this corresponds to the base of the CCNPP containment structure foundations at an elevation of -1 ft, and therefore, the control point is set to this elevation for use in the GMRS comparisons.

3.3 IPEEE DESCRIPTION AND CAPACITY RESPONSE SPECTRUM

A focused-scope Seismic Probabilistic Risk Assessment (SPRA) was performed to support the IPEEE for CCNPP. The results of the IPEEE were submitted to the NRC in Reference 12, and the results of the NRC review are documented in the Staff Evaluation Report (SER) (Reference 24). The IPEEE for CCNPP (Reference 12) demonstrates plant seismic capacity beyond the SSE.

The CCNPP Seismic IPEEE was performed using the Seismic Probabilistic Risk Assessment option per the methodology of NUREG-1407. With this method, the contribution and significance of seismic initiated events to the total plant risk can be assessed. The guidelines of Table 2-4 of EPRI NP-6041-SL (Reference 9) are used for component screening. Equipment and components are screened at either 0.3 g or 0.5 g based on which caveats of the table are met. For structures and components, the fragilities are estimated using a Review Level Earthquake (RLE) defined by the median shape Uniform Hazard Spectrum (UHS) for CCNPP from NUREG-1488 (Reference 22) with a 10,000-year return period and a PGA of 0.4 g. The PGA of 0.4 g is 4 times the associated median PGA of 0.1 g from NUREG-1488, and 2.67 times the Design Basis Earthquake of 0.15 g. (Reference 12)

The plant HCLPF seismic capacity is defined in Section 3.3.2 of the SPID as the IHS. Consistent with Section 3.3.2 of the SPID the CCNPP IHS is generated based on the 1E-4 return period Uniform Hazard Spectrum (UHS) shape used in the IPEEE SPRA. As stated in Section 2.4 of the Probabilistic Soil-Structure Interaction Analysis (referred to as PSSI from here on) (Reference 21), the median UHS shape for CCNPP is used as the basis for the SPRA reference earthquake. Per PSSI, the median spectral shape for CCNPP is provided in NUREG-1488. The IHS is anchored to the plant HCLPF value of 0.3 g.

The IPEEE was reviewed for adequacy utilizing the guidance provided in Section 3.3 of the SPID. The IPEEE Adequacy Determination according to SPID Section 3.3.1 is included in Appendix B.

The results of the review have shown, in accordance with the criteria established in SPID Section 3.3, that the IPEEE is adequate to support screening of the updated seismic hazard for CCNPP. The review also concluded that the risk insights obtained from the IPEEE are still valid under the current plant configuration.

The full scope detailed review of relay chatter required in SPID Section 3.3.1 has not been completed. NEI letter, "Relay Chatter Reviews for Seismic Hazard Screening" dated October 3, 2013 states that full scope relay chatter reviews will be completed on the same schedule as the High Frequency Confirmation as proposed in the NEI letter to NRC dated April 9, 2013 and accepted in NRC's response dated May 7, 2013 (Reference 23)

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

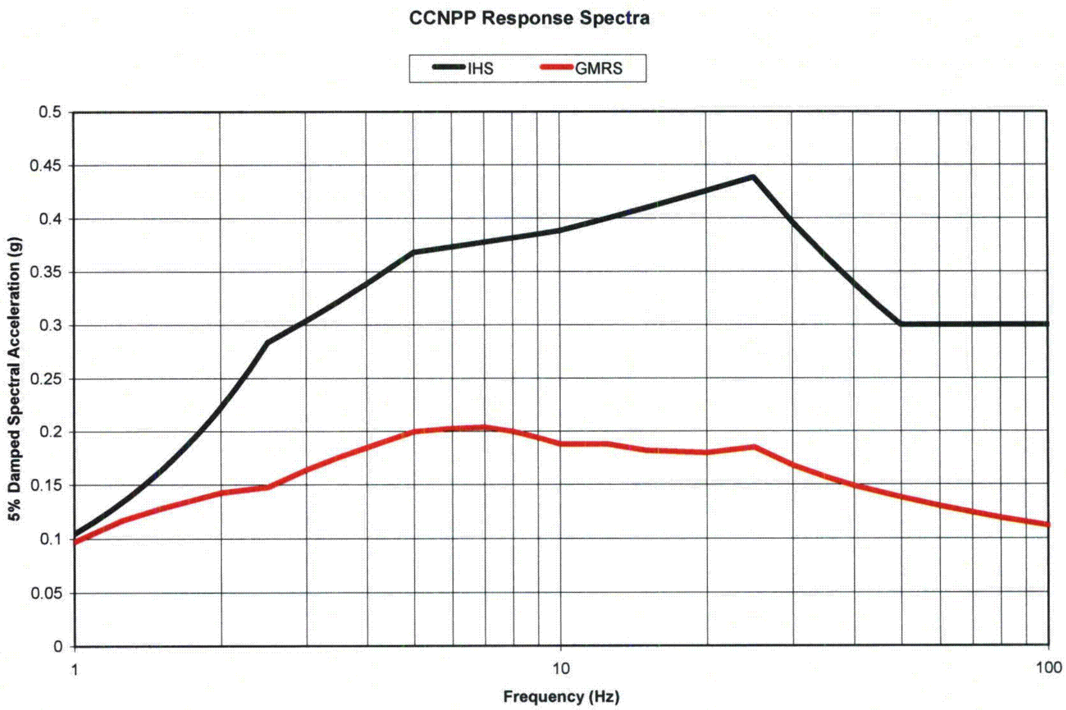


Figure 3.3-1: GMRS and IHS Response Spectra for CCNPP

Table 3.3-1: IHS for CCNPP

Freq. (Hz)	IHS (g)
1	0.105
1.1	0.116
1.2	0.128
1.3	0.139
1.4	0.151
1.5	0.163
1.6	0.175
1.7	0.186
1.8	0.198
1.9	0.211
2	0.223
2.1	0.235
2.2	0.247
2.3	0.259
2.4	0.272
2.5	0.284
3	0.304
3.5	0.322
4	0.339
5	0.368
6	0.373
7	0.378
8	0.382
9	0.385
10	0.388
12.5	0.400
15	0.410
20	0.426
25	0.438
30	0.397
35	0.365
40	0.339
45	0.318
50	0.300
100	0.300

4 Screening Evaluation

In accordance with the 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 IHS exceeds the GMRS. Based on this comparison, a risk evaluation will not be performed.

4.2 High Frequency Screening (> 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)

In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE. Therefore, the plant screens in for a spent fuel pool evaluation.

5 Interim Actions

Based on the screening evaluation outcome described in Section 4 of this report, the IHS exceeds the GMRS in the frequency range of 1 to 10 Hz. Therefore, a risk evaluation will not be performed for CCNPP. However, the GMRS exceeds the SSE in the frequency range of 1 to 10 Hz. Therefore, interim actions will be performed in accordance with the ESEP guidance.

5.1 Expedited Seismic Evaluation Process

Based on the results of the screening evaluation, the ESEP described in EPRI 3002000704 will be performed as proposed in a letter to NRC dated April 9, 2013 and agreed to by NRC in a letter dated May 7, 2013.

5.2 Interim Evaluation of Seismic Hazard

Consistent with NRC letter dated February 20, 2014, (Reference 25) the seismic hazard reevaluations presented herein are distinct from the current design and licensing bases of CCNPP. 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 (Reference 26) 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 (Reference 27):

> 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.

CCNPP station is included in the March 12, 2014 risk estimates (Reference 26). Using the methodology described in the NEI letter, all plants were shown to be below 10^{-4}/year; thus, the above conclusions apply.

5.3 SEISMIC WALKDOWN INSIGHTS

In response to NTTF Recommendation 2.3, seismic walkdowns for the CCNPP station have been completed as documented in References 19 and 20. All potentially degraded, nonconforming, or unanalyzed conditions identified as a result of the seismic walkdowns have been entered into the site corrective action program. The seismic walkdowns identified a limited set of minor conditions with no discernible trend. Other than those minor conditions, the seismic walkdowns identified no degraded, nonconforming, or unanalyzed conditions that required either immediate or follow-on action with one exception. During the Unit 2 Supplemental walkdowns of inaccessible equipment Containment Air Cooling Units (CAC) and Iodine Removal Units (IRU) were found to have missing welds for mounting to structural steel. These issues were entered into the Corrective Action Program. An Operability Determination (OD) was performed and found that the CACs and IRUs are operable without the welds in place. Additional welds were added to the Unit 2 CACs and IRUs to restore them to the full design requirements per the design drawings. The OD described above is applicable to both Unit 1 and 2.

The seismic walkdowns also verified that no major plant vulnerabilities or physical plant improvements were outstanding from the station IPEEE submittal. The identified issues were not determined to be potentially adverse seismic conditions because in all cases the anomaly or issue would not prevent the equipment from performing its safety-related function. No planned or newly identified protection or mitigation features have resulted from the efforts to address the 50.54(f) letter.

5.4 BEYOND DESIGN BASIS SEISMIC INSIGHTS

The Individual Plant Examination of External Events (IPEEE) was performed as a focused scope PRA using the NUREG-1407 methodologies. The results of the IPEEE were submitted to the NRC in Reference 12. The PRA indicated that the overall plant core damage frequency is 1.29E-5/year for Unit 1 and 1.52E-5/year for Unit 2. Both of these fall below the subsidiary objective of 10E-4/year discussed in Section 5.2.

6 Conclusions

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

The screening evaluation comparison demonstrates that the GMRS exceeds the SSE in the frequency range of 1 – 10 Hz. Further, CCNPP has performed the IPEEE Adequacy Review evaluation in accordance with the SPID guidance and determined that the IHS is adequate for screening (see Appendix B). The IHS exceeds the GMRS in the frequency range of 1 – 10 Hz. Based on the comparison of the IHS and GMRS, as shown and described in Section 3, a risk evaluation will not be performed for CCNPP.

CCNPP will perform a spent fuel pool integrity evaluation since the GMRS exceeds the SSE in the frequency range of 1 – 10 Hz. The spent fuel pool evaluation will be performed on a schedule consistent with NRC prioritization and the NEI letter dated April 9, 2013 as endorsed by the NRC in the letter to NEI dated May 7, 2013.

CCNPP will also perform near-term ESEP evaluations following the ESEP guidance. These evaluations will be conducted on the schedule for Central and Eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013. This is an interim action to establish beyond design basis safety margin.

The GMRS exceeds the SSE in the frequency range beyond 10 Hz. However, the GMRS does not exceed the IHS in the frequency range beyond 10 Hz and therefore, additional high frequency confirmations are not required.

CCNPP is a focused scope IPEEE plant. The SPID guidance requires a full scope relay review for plants which use the IHS for screening. The full scope relay review will be performed in accordance with the schedule provided in the letter from the industry to the NRC dated October 3, 2012.

7 References

1. NRC Letter (E. J. Leeds) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* March 2012.
2. Title 10 Code of Federal Regulations Part 50.
3. EPRI Technical Report 1025287, *Seismic Evaluation Guidance: Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, February 2013.
4. EPRI Technical Report 3002000704, *Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, May 2013.
5. Title 10 Code of Federal Regulations Part 100.
6. NEI Letter (A. R. Pietrangelo) to the NRC, *Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations,* April 9, 2013.
7. EPRI Technical Report 1021097 (NUREG-2115), *Central and Eastern United States Seismic Source Characterization for Nuclear Facilities*, January 2012.
8. EPRI Technical Report 3002000717, *EPRI (2004, 2006) Ground-Motion Model (GMM) Review Project*, June 2013.
9. EPRI NP-6041-SL, *A Methodology for Assessment of Nuclear Power Plant Seismic Margin (Revision 1)*, August 1991.
10. Silva, W.J., N. Abrahamson, G. Toro and C. Costantino, *Description and validation of the stochastic ground motion model*, Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573, 1997.
11. *Calvert Cliffs Updated Final Safety Analysis Report (UFSAR)*, Revision 46.
12. BGE, *Calvert Cliffs Nuclear Power Plant, Probabilistic Risk Assessment, Individual Plant Examination of External Events*, August 1997.
13. Lettis Consultants International (LCI), Inc., Project No. 1041, *Calvert Cliffs Hazard and Screening Report*, 2013.

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14. Constellation Nuclear Energy Group (CENG) letter to the NRC, *Response to Request for Information: Near-Term Task Force Recommendation 2.1, Seismic Reevaluation*, April 26, 2013.

15. NEI Letter to the NRC, *Relay Chatter Reviews for Seismic Hazard Screening,* October 3, 2012.

16. FZ Ares, LLC (2012), *Data Request for Site Amplification Analysis—CCNPP Units 1 and 2*, Project No. 12-0809 Rev. 1 August 30, 2012.

17. USNRC, NUREG-1407, *Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities*, June 1991.

18. EPRI Report 1015108, *Program on Technology Innovation: The Effects of High-Frequency Ground Motion on Structures, Components and Equipment in Nuclear Power Plants*, June 2007.

19. Stevenson & Associates, *Seismic Walkdown Report in Response to the 10 CFR 50.54(f) Information Request Regarding Fukushima Near-Term Task Force Recommendation 2.3: Seismic for the Calvert Cliffs Nuclear Power Plant Unit 1*, November 11, 2012

20. Stevenson & Associates, Seismic *Walkdown Report in Response to the 10 CFR 50.54(f) Information Request Regarding Fukushima Near-Term Task Force Recommendation 2.3: Seismic for the Calvert Cliffs Nuclear Power Plant Unit 2*, November 11, 2012

21. Stevenson & Associates, *Probabilistic Soil-Structure Interaction Analysis and Fragility Calculations for Selected Structures and Buildings at the Calvert Cliffs Nuclear Power Plant for use in the IPEEE SPRA*, December 27, 1994.

22. USNRC, NUREG-1488, *Revised Livermore Seismic Hazard Estimates for Sixty-Nine Nuclear Power Plant Sites East of the Rocky Mountains*, April 1994.

23. USNRC Letter to NEI, *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*, May 7, 2013.

24. USNRC, *Staff Evaluation Report of Individual Plant Examination of External Events (IPEEE) Submittal on the Calvert Cliffs Nuclear Power Station*, June 8, 2001.

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25. NRC Letter (E. J. Leeds) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Supplemental Information Related to Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-Ichi Accident*, February 20, 2014.

26. NEI Letter (A. R. Pietrangelo) to NRC, *Seismic Risk Evaluations for Plants in the Central and Eastern United States*, March 12, 2014.

27. NUREG-0933, *A Prioritization of Generic Safety Issues*, Supplement 34, Resolution of Generic Safety Issues, Revision 1, September, 2011

A

Additional Tables

The following additional tables are included in Appendix A:

- Table A-1a: Mean and Fractile Seismic Hazard Curves for 100 Hz at CCNPP
- Table A-1b: Mean and Fractile Seismic Hazard Curves for 25 Hz at CCNPP
- Table A-1c: Mean and Fractile Seismic Hazard Curves for 10 Hz at CCNPP
- Table A-1d: Mean and Fractile Seismic Hazard Curves for 5 Hz at CCNPP
- Table A-1e: Mean and Fractile Seismic Hazard Curves for 2.5 Hz at CCNPP
- Table A-1f: Mean and Fractile Seismic Hazard Curves for 1 Hz at CCNPP
- Table A-1g: Mean and Fractile Seismic Hazard Curves for 0.5 Hz at CCNPP
- Table A-2a: Medians and Logarithmic Sigmas of Amplification Factors for CCNPP
- Table A-2b1: Median AFs and Sigmas for Model 1, 2 PGA Levels
- Table A-2b2: Median AFs and Sigmas for Model 2, 2 PGA Levels

Table A-1a: Mean and Fractile Seismic Hazard Curves for PGA (100 Hz) at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.19E-02	1.67E-02	2.53E-02	3.23E-02	3.90E-02	4.25E-02
0.001	2.41E-02	1.18E-02	1.82E-02	2.39E-02	3.09E-02	3.47E-02
0.005	8.63E-03	3.73E-03	5.66E-03	8.12E-03	1.13E-02	1.57E-02
0.01	4.50E-03	1.72E-03	2.53E-03	4.07E-03	6.00E-03	9.65E-03
0.015	2.69E-03	9.24E-04	1.34E-03	2.32E-03	3.68E-03	6.45E-03
0.03	8.08E-04	1.95E-04	2.96E-04	5.91E-04	1.16E-03	2.49E-03
0.05	2.81E-04	4.56E-05	7.77E-05	1.72E-04	4.19E-04	1.01E-03
0.075	1.18E-04	1.44E-05	2.80E-05	6.73E-05	1.77E-04	4.37E-04
0.1	6.39E-05	6.73E-06	1.44E-05	3.63E-05	9.65E-05	2.29E-04
0.15	2.64E-05	2.42E-06	5.91E-06	1.57E-05	4.01E-05	8.98E-05
0.3	5.39E-06	3.79E-07	1.15E-06	3.37E-06	8.72E-06	1.74E-05
0.5	1.51E-06	8.00E-08	2.72E-07	9.24E-07	2.53E-06	4.83E-06
0.75	5.04E-07	2.04E-08	7.23E-08	2.88E-07	8.47E-07	1.69E-06
1.	2.18E-07	6.64E-09	2.53E-08	1.16E-07	3.63E-07	7.89E-07
1.5	6.10E-08	1.23E-09	4.56E-09	2.80E-08	9.93E-08	2.49E-07
3.	4.95E-09	9.11E-11	1.87E-10	1.46E-09	7.13E-09	2.46E-08
5.	5.63E-10	4.56E-11	8.12E-11	1.57E-10	7.77E-10	3.19E-09
7.5	8.17E-11	4.01E-11	5.05E-11	9.11E-11	1.53E-10	5.20E-10
10.	1.89E-11	4.01E-11	4.01E-11	8.35E-11	9.11E-11	1.67E-10

Table A-1b: Mean and Fractile Seismic Hazard Curves for 25 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.30E-02	1.92E-02	2.76E-02	3.33E-02	3.90E-02	4.31E-02
0.001	2.56E-02	1.40E-02	2.01E-02	2.53E-02	3.19E-02	3.68E-02
0.005	1.06E-02	5.27E-03	7.23E-03	1.01E-02	1.36E-02	1.84E-02
0.01	6.35E-03	2.80E-03	3.95E-03	5.83E-03	8.35E-03	1.20E-02
0.015	4.13E-03	1.69E-03	2.39E-03	3.73E-03	5.58E-03	8.35E-03
0.03	1.30E-03	4.56E-04	6.54E-04	1.10E-03	1.79E-03	3.09E-03
0.05	4.34E-04	1.25E-04	1.84E-04	3.33E-04	6.17E-04	1.25E-03
0.075	1.91E-04	4.07E-05	6.93E-05	1.40E-04	2.84E-04	5.91E-04
0.1	1.14E-04	2.01E-05	3.79E-05	8.23E-05	1.72E-04	3.57E-04
0.15	5.74E-05	9.11E-06	1.84E-05	4.19E-05	8.85E-05	1.72E-04
0.3	1.74E-05	2.39E-06	5.58E-06	1.34E-05	2.76E-05	4.63E-05
0.5	6.63E-06	7.23E-07	2.10E-06	5.20E-06	1.08E-05	1.72E-05
0.75	2.86E-06	2.46E-07	8.35E-07	2.19E-06	4.77E-06	7.77E-06
1.	1.50E-06	1.20E-07	4.01E-07	1.11E-06	2.53E-06	4.25E-06
1.5	5.57E-07	3.73E-08	1.25E-07	3.90E-07	9.65E-07	1.69E-06
3.	7.90E-08	3.37E-09	1.18E-08	4.77E-08	1.36E-07	2.80E-07
5.	1.46E-08	4.37E-10	1.44E-09	7.23E-09	2.46E-08	5.91E-08
7.5	3.23E-09	1.13E-10	2.64E-10	1.36E-09	5.27E-09	1.40E-08
10.	1.01E-09	7.66E-11	1.08E-10	3.84E-10	1.64E-09	4.56E-09

Table A-1c: Mean and Fractile Seismic Hazard Curves for 10 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.61E-02	2.60E-02	3.09E-02	3.63E-02	4.13E-02	4.43E-02
0.001	2.90E-02	1.90E-02	2.35E-02	2.88E-02	3.47E-02	3.84E-02
0.005	1.17E-02	6.36E-03	8.35E-03	1.13E-02	1.51E-02	1.84E-02
0.01	6.72E-03	3.28E-03	4.50E-03	6.36E-03	8.85E-03	1.15E-02
0.015	4.46E-03	2.07E-03	2.84E-03	4.19E-03	5.91E-03	8.00E-03
0.03	1.77E-03	7.23E-04	9.93E-04	1.57E-03	2.46E-03	3.57E-03
0.05	7.34E-04	2.53E-04	3.63E-04	6.26E-04	1.05E-03	1.62E-03
0.075	3.32E-04	9.24E-05	1.42E-04	2.68E-04	4.98E-04	8.00E-04
0.1	1.84E-04	4.19E-05	6.93E-05	1.44E-04	2.88E-04	4.70E-04
0.15	7.89E-05	1.38E-05	2.57E-05	5.83E-05	1.27E-04	2.16E-04
0.3	1.79E-05	2.19E-06	4.98E-06	1.29E-05	2.96E-05	5.12E-05
0.5	5.62E-06	5.42E-07	1.44E-06	3.95E-06	9.51E-06	1.67E-05
0.75	2.08E-06	1.62E-07	4.83E-07	1.44E-06	3.57E-06	6.17E-06
1.	9.72E-07	6.45E-08	2.07E-07	6.64E-07	1.67E-06	2.96E-06
1.5	3.09E-07	1.60E-08	5.58E-08	2.01E-07	5.35E-07	9.93E-07
3.	3.54E-08	1.04E-09	3.79E-09	1.82E-08	6.17E-08	1.34E-07
5.	6.14E-09	1.32E-10	3.73E-10	2.22E-09	1.05E-08	2.60E-08
7.5	1.37E-09	6.54E-11	9.79E-11	3.95E-10	2.22E-09	6.17E-09
10.	4.43E-10	4.37E-11	8.12E-11	1.40E-10	7.13E-10	2.10E-09

Table A-1d: Mean and Fractile Seismic Hazard Curves for 5 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.89E-02	3.01E-02	3.37E-02	3.90E-02	4.37E-02	4.70E-02
0.001	3.33E-02	2.29E-02	2.72E-02	3.37E-02	3.95E-02	4.31E-02
0.005	1.44E-02	7.55E-03	1.02E-02	1.40E-02	1.87E-02	2.16E-02
0.01	7.94E-03	3.73E-03	5.35E-03	7.66E-03	1.07E-02	1.27E-02
0.015	5.15E-03	2.32E-03	3.37E-03	4.90E-03	7.03E-03	8.60E-03
0.03	2.05E-03	8.85E-04	1.27E-03	1.90E-03	2.84E-03	3.73E-03
0.05	8.91E-04	3.63E-04	5.20E-04	8.12E-04	1.25E-03	1.74E-03
0.075	4.21E-04	1.53E-04	2.22E-04	3.73E-04	6.09E-04	8.72E-04
0.1	2.38E-04	7.66E-05	1.16E-04	2.07E-04	3.52E-04	5.20E-04
0.15	1.01E-04	2.57E-05	4.25E-05	8.47E-05	1.57E-04	2.35E-04
0.3	2.10E-05	3.09E-06	6.36E-06	1.57E-05	3.52E-05	5.66E-05
0.5	5.97E-06	4.83E-07	1.21E-06	3.95E-06	1.07E-05	1.82E-05
0.75	2.05E-06	9.11E-08	2.76E-07	1.18E-06	3.73E-06	6.93E-06
1.	9.21E-07	2.57E-08	9.37E-08	4.70E-07	1.69E-06	3.28E-06
1.5	2.76E-07	4.13E-09	1.92E-08	1.13E-07	5.05E-07	1.08E-06
3.	2.82E-08	1.92E-10	8.85E-10	7.13E-09	4.83E-08	1.23E-07
5.	4.38E-09	7.77E-11	1.08E-10	7.45E-10	6.45E-09	1.98E-08
7.5	8.91E-10	4.07E-11	8.12E-11	1.44E-10	1.16E-09	3.95E-09
10.	2.70E-10	4.01E-11	5.05E-11	9.11E-11	3.28E-10	1.23E-09

Table A-1e: Mean and Fractile Seismic Hazard Curves for 2.5 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.86E-02	3.01E-02	3.33E-02	3.84E-02	4.37E-02	4.70E-02
0.001	3.28E-02	2.25E-02	2.64E-02	3.28E-02	3.95E-02	4.31E-02
0.005	1.33E-02	6.93E-03	9.11E-03	1.29E-02	1.77E-02	2.07E-02
0.01	6.77E-03	3.09E-03	4.37E-03	6.45E-03	9.24E-03	1.13E-02
0.015	4.11E-03	1.77E-03	2.53E-03	3.84E-03	5.75E-03	7.34E-03
0.03	1.43E-03	5.58E-04	8.12E-04	1.31E-03	2.04E-03	2.80E-03
0.05	5.62E-04	2.01E-04	2.96E-04	4.98E-04	8.12E-04	1.18E-03
0.075	2.47E-04	7.77E-05	1.21E-04	2.13E-04	3.68E-04	5.35E-04
0.1	1.32E-04	3.73E-05	6.00E-05	1.13E-04	2.01E-04	2.96E-04
0.15	5.26E-05	1.21E-05	2.07E-05	4.31E-05	8.35E-05	1.27E-04
0.3	9.91E-06	1.42E-06	2.88E-06	7.34E-06	1.64E-05	2.76E-05
0.5	2.65E-06	2.29E-07	5.66E-07	1.72E-06	4.50E-06	8.23E-06
0.75	8.69E-07	4.25E-08	1.25E-07	4.83E-07	1.51E-06	3.05E-06
1.	3.80E-07	1.11E-08	3.63E-08	1.79E-07	6.73E-07	1.42E-06
1.5	1.12E-07	1.21E-09	4.56E-09	3.84E-08	1.95E-07	4.70E-07
3.	1.20E-08	7.89E-11	1.18E-10	1.77E-09	1.74E-08	5.66E-08
5.	1.94E-09	4.01E-11	8.12E-11	1.69E-10	2.16E-09	9.11E-09
7.5	4.04E-10	4.01E-11	5.05E-11	9.11E-11	3.68E-10	1.82E-09
10.	1.24E-10	4.01E-11	4.77E-11	9.11E-11	1.32E-10	5.42E-10

Table A-1f: Mean and Fractile Seismic Hazard Curves for 1 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	3.17E-02	1.92E-02	2.42E-02	3.23E-02	3.90E-02	4.25E-02
0.001	2.38E-02	1.21E-02	1.67E-02	2.39E-02	3.05E-02	3.52E-02
0.005	7.34E-03	2.72E-03	4.25E-03	6.93E-03	1.04E-02	1.31E-02
0.01	3.50E-03	1.04E-03	1.74E-03	3.19E-03	5.20E-03	7.03E-03
0.015	2.08E-03	5.27E-04	9.24E-04	1.82E-03	3.23E-03	4.56E-03
0.03	6.97E-04	1.34E-04	2.49E-04	5.50E-04	1.13E-03	1.77E-03
0.05	2.58E-04	4.07E-05	8.00E-05	1.90E-04	4.25E-04	7.13E-04
0.075	1.05E-04	1.42E-05	2.92E-05	7.34E-05	1.77E-04	3.05E-04
0.1	5.32E-05	6.45E-06	1.36E-05	3.52E-05	8.98E-05	1.57E-04
0.15	1.94E-05	1.95E-06	4.31E-06	1.21E-05	3.37E-05	6.09E-05
0.3	3.36E-06	2.13E-07	5.35E-07	1.77E-06	5.66E-06	1.20E-05
0.5	9.43E-07	3.52E-08	1.07E-07	4.25E-07	1.55E-06	3.73E-06
0.75	3.44E-07	7.45E-09	2.68E-08	1.31E-07	5.50E-07	1.44E-06
1.	1.67E-07	2.32E-09	9.51E-09	5.42E-08	2.60E-07	7.23E-07
1.5	5.80E-08	4.56E-10	1.95E-09	1.42E-08	8.60E-08	2.68E-07
3.	8.34E-09	8.98E-11	1.42E-10	1.02E-09	9.79E-09	4.01E-08
5.	1.71E-09	4.50E-11	8.12E-11	1.55E-10	1.53E-09	7.77E-09
7.5	4.29E-10	4.01E-11	5.05E-11	9.11E-11	3.37E-10	1.84E-09
10.	1.50E-10	4.01E-11	4.83E-11	9.11E-11	1.38E-10	6.26E-10

Table A-1g: Mean and Fractile Seismic Hazard Curves for 0.5 Hz at CCNPP (Ref. 13)

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	1.89E-02	1.01E-02	1.32E-02	1.87E-02	2.46E-02	2.84E-02
0.001	1.20E-02	5.58E-03	7.77E-03	1.16E-02	1.62E-02	1.95E-02
0.005	2.99E-03	7.55E-04	1.32E-03	2.68E-03	4.63E-03	6.36E-03
0.01	1.33E-03	2.16E-04	4.25E-04	1.07E-03	2.22E-03	3.37E-03
0.015	7.45E-04	8.98E-05	1.92E-04	5.42E-04	1.31E-03	2.13E-03
0.03	2.22E-04	1.62E-05	3.90E-05	1.32E-04	3.95E-04	7.45E-04
0.05	7.54E-05	3.90E-06	1.01E-05	3.84E-05	1.34E-04	2.72E-04
0.075	2.90E-05	1.16E-06	3.19E-06	1.31E-05	5.12E-05	1.11E-04
0.1	1.42E-05	4.77E-07	1.36E-06	5.75E-06	2.49E-05	5.66E-05
0.15	5.09E-06	1.31E-07	3.95E-07	1.79E-06	8.60E-06	2.19E-05
0.3	9.15E-07	1.15E-08	4.56E-08	2.39E-07	1.34E-06	4.37E-06
0.5	2.71E-07	1.60E-09	8.12E-09	5.42E-08	3.47E-07	1.38E-06
0.75	1.04E-07	3.23E-10	1.90E-09	1.55E-08	1.18E-07	5.58E-07
1.	5.22E-08	1.32E-10	6.45E-10	6.00E-09	5.35E-08	2.80E-07
1.5	1.92E-08	9.11E-11	1.62E-10	1.44E-09	1.60E-08	1.01E-07
3.	3.03E-09	4.25E-11	8.12E-11	1.42E-10	1.55E-09	1.44E-08
5.	6.61E-10	4.01E-11	5.05E-11	9.11E-11	2.60E-10	2.76E-09
7.5	1.74E-10	4.01E-11	4.01E-11	9.11E-11	9.93E-11	6.54E-10
10.	6.29E-11	4.01E-11	4.01E-11	8.12E-11	9.11E-11	2.42E-10

Table A-2a: Medians and Logarithmic Sigmas of Amplification Factors for CCNPP (Ref. 13)

PGA (100 Hz)	**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	1.38E+00	6.84E-02	1.30E-02	1.07E+00	6.80E-02	1.90E-02	9.80E-01	9.79E-02	2.09E-02	1.35E+00	1.30E-01
4.95E-02	8.98E-01	7.44E-02	1.02E-01	5.00E-01	8.02E-02	9.99E-02	7.57E-01	1.39E-01	8.24E-02	1.24E+00	1.41E-01
9.64E-02	7.59E-01	7.74E-02	2.13E-01	5.00E-01	8.61E-02	1.85E-01	6.94E-01	1.49E-01	1.44E-01	1.18E+00	1.43E-01
1.94E-01	6.49E-01	8.16E-02	4.43E-01	5.00E-01	9.20E-02	3.56E-01	6.21E-01	1.57E-01	2.65E-01	1.09E+00	1.47E-01
2.92E-01	5.91E-01	8.48E-02	6.76E-01	5.00E-01	9.57E-02	5.23E-01	5.72E-01	1.63E-01	3.84E-01	1.03E+00	1.52E-01
3.91E-01	5.51E-01	8.73E-02	9.09E-01	5.00E-01	9.81E-02	6.90E-01	5.33E-01	1.66E-01	5.02E-01	9.77E-01	1.56E-01
4.93E-01	5.21E-01	9.00E-02	1.15E+00	5.00E-01	1.01E-01	8.61E-01	5.01E-01	1.70E-01	6.22E-01	9.32E-01	1.61E-01
7.41E-01	5.00E-01	9.40E-02	1.73E+00	5.00E-01	1.04E-01	1.27E+00	5.00E-01	1.76E-01	9.13E-01	8.41E-01	1.69E-01
1.01E+00	5.00E-01	9.77E-02	2.36E+00	5.00E-01	1.07E-01	1.72E+00	5.00E-01	1.79E-01	1.22E+00	7.70E-01	1.74E-01
1.28E+00	5.00E-01	1.02E-01	3.01E+00	5.00E-01	1.10E-01	2.17E+00	5.00E-01	1.84E-01	1.54E+00	7.11E-01	1.79E-01
1.55E+00	5.00E-01	1.06E-01	3.63E+00	5.00E-01	1.12E-01	2.61E+00	5.00E-01	1.87E-01	1.85E+00	6.67E-01	1.85E-01
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	1.65E+00	1.29E-01	1.27E-02	2.24E+00	2.01E-01	8.25E-03	2.19E+00	1.57E-01			
7.05E-02	1.56E+00	1.33E-01	3.43E-02	2.17E+00	1.95E-01	1.96E-02	2.16E+00	1.51E-01			
1.18E-01	1.51E+00	1.35E-01	5.51E-02	2.13E+00	1.93E-01	3.02E-02	2.15E+00	1.49E-01			
2.12E-01	1.43E+00	1.37E-01	9.63E-02	2.08E+00	1.90E-01	5.11E-02	2.13E+00	1.50E-01			
3.04E-01	1.36E+00	1.39E-01	1.36E-01	2.05E+00	1.87E-01	7.10E-02	2.12E+00	1.51E-01			
3.94E-01	1.31E+00	1.41E-01	1.75E-01	2.02E+00	1.85E-01	9.06E-02	2.11E+00	1.49E-01			
4.86E-01	1.26E+00	1.44E-01	2.14E-01	2.00E+00	1.82E-01	1.10E-01	2.10E+00	1.47E-01			
7.09E-01	1.16E+00	1.51E-01	3.10E-01	1.96E+00	1.78E-01	1.58E-01	2.09E+00	1.49E-01			
9.47E-01	1.08E+00	1.58E-01	4.12E-01	1.94E+00	1.76E-01	2.09E-01	2.09E+00	1.58E-01			
1.19E+00	1.02E+00	1.64E-01	5.18E-01	1.92E+00	1.76E-01	2.62E-01	2.09E+00	1.65E-01			
1.43E+00	9.92E-01	1.66E-01	6.19E-01	1.90E+00	1.76E-01	3.12E-01	2.09E+00	1.70E-01			

Tables A-2b1 and A-2b2 are tabular versions of the typical amplification factors provided in Figures 2.3.6-1 and 2.3.6-2. Values are provided for two input motion levels at approximately 10^{-4} and 10^{-5} mean annual frequency exceedance. These factors are unverified and are provided for information only. The figures should be considered the governing information.

Table A-2b1: Median AFs and Sigmas for Model 1, 2 PGA Levels

M1P1K1 PGA=0.096				M1P1K1 PGA=0.292			
freq	PGA	med AF	sigma ln(AF)	freq	PGA	med AF	sigma ln(AF)
100.0	0.096	1.279	0.096	100.0	0.292	1.046	0.092
87.1	0.096	1.255	0.096	87.1	0.292	1.018	0.092
75.9	0.096	1.214	0.097	75.9	0.292	0.971	0.092
66.1	0.096	1.138	0.097	66.1	0.292	0.885	0.093
57.5	0.096	1.007	0.098	57.5	0.292	0.751	0.093
50.1	0.096	0.862	0.100	50.1	0.292	0.625	0.095
43.7	0.096	0.746	0.102	43.7	0.292	0.535	0.097
38.0	0.096	0.686	0.107	38.0	0.292	0.498	0.100
33.1	0.096	0.658	0.110	33.1	0.292	0.486	0.105
28.8	0.096	0.673	0.113	28.8	0.292	0.504	0.106
25.1	0.096	0.701	0.126	25.1	0.292	0.528	0.111
21.9	0.096	0.760	0.136	21.9	0.292	0.590	0.124
19.1	0.096	0.813	0.142	19.1	0.292	0.634	0.137
16.6	0.096	0.924	0.159	16.6	0.292	0.718	0.149
14.5	0.096	1.039	0.164	14.5	0.292	0.837	0.168
12.6	0.096	1.092	0.169	12.6	0.292	0.911	0.174
11.0	0.096	1.141	0.173	11.0	0.292	0.965	0.166
9.5	0.096	1.312	0.165	9.5	0.292	1.070	0.171
8.3	0.096	1.561	0.137	8.3	0.292	1.308	0.161
7.2	0.096	1.707	0.167	7.2	0.292	1.463	0.149
6.3	0.096	2.032	0.159	6.3	0.292	1.696	0.175
5.5	0.096	2.489	0.144	5.5	0.292	2.051	0.182
4.8	0.096	2.312	0.238	4.8	0.292	2.186	0.190
4.2	0.096	1.938	0.257	4.2	0.292	1.988	0.221
3.6	0.096	1.855	0.226	3.6	0.292	1.901	0.243
3.2	0.096	1.957	0.190	3.2	0.292	1.936	0.225
2.8	0.096	2.212	0.180	2.8	0.292	2.154	0.202
2.4	0.096	2.188	0.121	2.4	0.292	2.230	0.160
2.1	0.096	2.217	0.131	2.1	0.292	2.237	0.149
1.8	0.096	2.510	0.103	1.8	0.292	2.534	0.120
1.6	0.096	2.425	0.105	1.6	0.292	2.565	0.110
1.4	0.096	2.177	0.096	1.4	0.292	2.326	0.111
1.2	0.096	2.012	0.094	1.2	0.292	2.129	0.103
1.0	0.096	1.882	0.087	1.0	0.292	1.967	0.094

Table A-2b1: Median AFs and Sigmas for Model 1, 2 PGA Levels (cont'd)

M1P1K1 PGA=0.096				M1P1K1 PGA=0.292			
freq	PGA	med AF	sigma ln(AF)	freq	PGA	med AF	sigma ln(AF)
0.91	0.096	1.909	0.086	0.91	0.292	1.973	0.093
0.79	0.096	1.927	0.074	0.79	0.292	1.979	0.079
0.69	0.096	1.870	0.069	0.69	0.292	1.914	0.072
0.60	0.096	1.764	0.096	0.60	0.292	1.801	0.096
0.52	0.096	1.616	0.102	0.52	0.292	1.648	0.101
0.46	0.096	1.507	0.096	0.46	0.292	1.534	0.095
0.10	0.096	1.318	0.039	0.10	0.292	1.325	0.039

Table A-2b2: Median AFs and Sigmas for Model 2, 2 PGA Levels

M2P1K1 PGA=0.096				M2P1K1 PGA=0.292			
freq	PGA	med AF	sigma ln(AF)	freq	PGA	med AF	sigma ln(AF)
100.0	0.096	1.324	0.103	100.0	0.292	1.110	0.100
87.1	0.096	1.300	0.103	87.1	0.292	1.082	0.101
75.9	0.096	1.259	0.103	75.9	0.292	1.032	0.101
66.1	0.096	1.180	0.104	66.1	0.292	0.942	0.101
57.5	0.096	1.045	0.105	57.5	0.292	0.800	0.103
50.1	0.096	0.897	0.108	50.1	0.292	0.668	0.106
43.7	0.096	0.779	0.112	43.7	0.292	0.575	0.109
38.0	0.096	0.720	0.119	38.0	0.292	0.540	0.115
33.1	0.096	0.691	0.122	33.1	0.292	0.530	0.121
28.8	0.096	0.715	0.126	28.8	0.292	0.555	0.121
25.1	0.096	0.748	0.147	25.1	0.292	0.592	0.134
21.9	0.096	0.815	0.161	21.9	0.292	0.662	0.151
19.1	0.096	0.879	0.152	19.1	0.292	0.724	0.153
16.6	0.096	1.012	0.159	16.6	0.292	0.841	0.171
14.5	0.096	1.117	0.169	14.5	0.292	0.959	0.162
12.6	0.096	1.150	0.176	12.6	0.292	1.016	0.156
11.0	0.096	1.201	0.175	11.0	0.292	1.058	0.166
9.5	0.096	1.420	0.167	9.5	0.292	1.217	0.167
8.3	0.096	1.650	0.140	8.3	0.292	1.486	0.140
7.2	0.096	1.794	0.162	7.2	0.292	1.608	0.157
6.3	0.096	2.173	0.145	6.3	0.292	1.909	0.173
5.5	0.096	2.638	0.138	5.5	0.292	2.357	0.162
4.8	0.096	2.276	0.252	4.8	0.292	2.236	0.189
4.2	0.096	1.901	0.254	4.2	0.292	1.913	0.222
3.6	0.096	1.854	0.225	3.6	0.292	1.865	0.246
3.2	0.096	1.992	0.186	3.2	0.292	1.968	0.222

Table A-2b2: Median AFs and Sigmas for Model 2, 2 PGA Levels (cont'd)

M2P1K1 PGA=0.096				M2P1K1 PGA=0.292			
freq	PGA	med AF	sigma ln(AF)	freq	PGA	med AF	sigma ln(AF)
2.8	0.096	2.252	0.169	2.8	0.292	2.223	0.196
2.4	0.096	2.192	0.121	2.4	0.292	2.213	0.143
2.1	0.096	2.224	0.136	2.1	0.292	2.226	0.144
1.8	0.096	2.510	0.101	1.8	0.292	2.521	0.111
1.6	0.096	2.398	0.111	1.6	0.292	2.482	0.117
1.4	0.096	2.147	0.090	1.4	0.292	2.236	0.101
1.2	0.096	1.989	0.093	1.2	0.292	2.058	0.098
1.0	0.096	1.867	0.084	1.0	0.292	1.917	0.088
0.91	0.096	1.897	0.082	0.91	0.292	1.935	0.084
0.79	0.096	1.918	0.071	0.79	0.292	1.949	0.072
0.69	0.096	1.863	0.069	0.69	0.292	1.889	0.071
0.60	0.096	1.758	0.098	0.60	0.292	1.782	0.098
0.52	0.096	1.612	0.104	0.52	0.292	1.633	0.104
0.46	0.096	1.503	0.096	0.46	0.292	1.522	0.096
0.10	0.096	1.317	0.040	0.10	0.292	1.320	0.040

Calvert Cliffs Nuclear Power Plant

Units 1 and 2

Individual Plant Examination of External Events (IPEEE)

Adequacy Review

Report No.: SL-012330

Revision 0

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Issue Summary

Sections 1.0, 2.0, 3.0, 4.3, 4.4, 4.5, 4.7, 4.8, 4.9, 5.0, and 6.0:

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Sections 4.1 and 4.2:

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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 Nuclear Regulatory Commission (NRC) established a Near-Term Task Force (NTTF). The NTTF was tasked with conducting a systematic review of NRC processes and regulations 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 (Ref. 24). Subsequently, the NRC issued a 50.54(f) letter requesting information to assure these recommendations would be addressed by all U.S. nuclear power plants (Ref. 22). The 50.54(f) letter requests that licensees and holders of construction permits under Title10 Code of Federal Regulations Part 50 reevaluate the seismic hazards at their sites using updated seismic hazard information and present-day regulatory guidance and methodologies. Depending on the outcome of the comparison between the reevaluated seismic hazard and the current design basis, performance of a seismic risk assessment may be necessary.

Calvert Cliffs Nuclear Power Plant (CCNPP) is located in Southern Maryland on the banks of the Chesapeake Bay. CCNPP is a deep soil site, with a Safe Shutdown Earthquake (SSE) of 0.15 g and an Operating Basis Earthquake (OBE) of 0.08 g as stated in Section 2.6 of the UFSAR (Ref. 11). CCNPP performed a focused scope Seismic Probabilistic Risk Assessment (SPRA) during the Individual Plant Examination of External Events (IPEEE) process (Ref. 1).

The guidance for developing the updated seismic hazard, performing the seismic hazard screening, and performing the subsequent seismic risk assessment work is provided in 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" (Ref. 2). A Ground Motion Response Spectra (GMRS) using up to date seismic hazard data and source characterization is developed for each site. This new GMRS is compared to the site design basis response spectra using the SPID guidance. The first method for seismic screening is based on a comparison of GMRS to the site design basis Safe Shutdown Earthquake (SSE) spectrum. The second method for seismic screening is to compare the GMRS to the site Individual Plant Examination of External Events (IPEEE) High Confidence of a Low Probability of Failure (HCLPF) spectrum (IHS). Plants that do not screen out must perform a seismic risk assessment.

In order to perform the GMRS to IHS screening, the site IPEEE is subject to an adequacy review to ensure that the IPEEE is of sufficient quality. The adequacy review guidance is provided in Section 3.3.1 of the SPID.

In accordance with Section 3.3.1 of the SPID this evaluation is being performed to determine the adequacy of the CCNPP IPEEE for use as a screening document to determine if a further seismic risk assessment is required.

This report is written from a Unit 1 perspective. Key differences between the Units are described in Section 1.2.

1.1 SPID Requirements for IPEEE Adequacy

Nuclear power plant licensees were required to perform the Individual Plant Examination of External Events for Severe Accident Vulnerabilities per Generic Letter No. 88-20, Supplement 4 (Ref. 23). Seismic hazards were one of the external events evaluated in the IPEEE program. Guidance for performing the IPEEE analysis was provided in NUREG-1407 (Ref. 10). The seismic IPEEE was accomplished by utilizing either a Seismic Probabilistic Risk Assessment (SPRA) or Seismic Margins Method (SMM) (also referred to as Seismic Margins Assessment (SMA)).

The SPID defines four categories which must be addressed in order to use the IHS for seismic hazard screening. The four categories are:

- General Considerations
- Prerequisites
- Adequacy Demonstration
- Documentation

Under General Considerations, the focused scope IPEEE can not be used for screening. A focused scope IPEEE review must be enhanced to include (1) a full scope review of relay chatter, and (2) a full review of soil failure modes.

Four prerequisites are defined in the SPID which must be confirmed and documented in the hazard submittal to the NRC. These prerequisites generally relate to closure of any open items from the IPEEE submittal including commitments, plant improvements/modifications, and addressing any weaknesses from the IPEEE submittal. The final prerequisite requires a review of plant modifications since the IPEEE submittal to confirm that the conclusions of the IPEEE are not impacted.

Adequacy Demonstrations must be performed on nine different items from the IPEEE submittal. Each of the Adequacy Demonstration items must evaluate (1) the methodology used, (2) whether the analysis was conducted in accordance with NUREG-1407, and (3) a statement, if applicable, as to whether the results are adequate for screening purposes.

Licensees are also requested to have documentation of the Prerequisites and Adequacy Demonstration and the information used to assess these items available for review at the site for potential NRC staff audits.

1.2 Calvert Cliffs IPEEE Seismic Description

A seismic PRA was performed for CCNPP using the approaches described in NUREG-1407 and EPRI NP-6041-SL (Ref. 3). Contractors with specific seismic expertise were utilized. For example, EQE International performed the component walkdown, HCLPF screening and fragility calculations. Stevenson & Associates performed all the soil-structure related seismic analyses.

A detailed component walk down list is first developed followed by a screening process using the results from extensive plant walkdowns. The results from the USI A-46 project are also utilized for screening. HCLPF (high-confidence-of-low-probability of failure) calculations are performed for the second screening followed by detailed fragility calculations for all the non-screened components. A HCLPF of 0.3 g is used for screening. Soil liquefaction, soil-structure interaction and structural fragility analyses are also performed and the results are used as input for the component HCLPF and fragility calculations.

The seismic initiating events are developed using the results from the fragility calculations. The annual probability of exceedance for peak ground acceleration from NUREG-1488 (Ref. 9) for CCNPP is used for the initiating events binning. All the non-screened components are grouped as a surrogate component which is assumed to lead directly to core damage when failed. The seismic impact in terms of core damage frequency and containment performance are quantified using a modified version of CCNPP's IPE submittal model.

A Unit 2 assessment for the internal events PRA determined that there are only minor differences between Unit 1 and Unit 2, and they do not warrant the completion of a Unit 2 specific PRA. For the seismic analysis, the differences between the units are noted in Ref. 5 and per the IPEEE none are judged to be significant (Section 3.1.5.7 of Ref. 1).

The most significant difference between Unit 1 and Unit 2 is the emergency diesel generator (EDG) configuration. Unit 2 has increased dependency on service water (SRW) and EDG 0C. This leads to a Unit 2 Seismic core damage frequency (CDF) of 1.52E-5 which is approximately 23% higher than the Unit 1 Seismic CDF of 1.29E-5. There are no other significant differences between Unit 1 and Unit 2. The Unit 2 Seismic CDF was evaluated using a modified version of the Unit 1 Seismic Model.

2.0 General Considerations

CCNPP was a focused scope plant for the IPEEE evaluation. The SPID Section 3.3.1 – General Considerations requires that focused scope plants perform full scope relay chatter reviews and a soil failures evaluation.

2.1 Relay Chatter

CCNPP is a focused scope review IPEEE plant and therefore must perform full scope relay chatter reviews. The Nuclear Energy Institute (NEI) letter "Relay Chatter Reviews for Seismic Hazard Screening" dated October 3, 2013 (Ref. 15) states that full scope relay chatter reviews will be performed on a schedule consistent with high frequency evaluations. Thus, this report does not address relay chatter, but CCNPP intends to perform relay chatter reviews on the same schedule as the high frequency confirmations for the plant.

2.2 Soil Failure Evaluation

For the IPEEE adequacy review the following soil failures need to be addressed for full-scope plant sites (Section 3.2.4.3 of NUREG-1407):

- Soil Liquefaction
- Foundation Settlement
- Slope Instability (failure)

Per Section 3.1.3 of Ref. 1 the following buildings are evaluated for IPEEE adequacy of Calvert Cliffs Units 1 and 2.

- Reactor Buildings
- Auxiliary Buildings
- Intake Structure
- New Emergency Diesel Generator Building
- Turbine Building

Per Section 7 of EPRI NP-6041-SL, the soil failure evaluation of above listed buildings has been performed using the following steps:

- Collection and review of pertinent documents: The documents containing static and seismic geotechnical data available in Calvert Cliffs UFSAR Section 2.7 and available calculations/documents related to the soil failures have been collected, reviewed and used in performing the soil failures evaluation.
- Identifying the soil-related issues affecting the success path: Soil liquefaction, foundation settlements and slope instability affecting the buildings identified in Section 3.1.3 of the IPEEE (listed above) are evaluated.

- Screening of selected soil-related issues through a review: Soil-related issues (soil liquefaction, foundation settlement, slope failure) are screened to eliminate the issues that do not require further detailed evaluation.
- In case any soil related issue could not be screened out, calculation is performed to evaluate the particular issue.

Geological and Geotechnical Information

Sections 2.4 and 2.7 of CCNPP UFSAR describe the supporting media for CCNPP structures. The stratigraphy consists of a sequence of Quaternary, Tertiary and Cretaceous sand, silt, and gravel deposits, which are about 2,500 feet thick at the site. Generalized soil profiles for the site are shown in CCNPP UFSAR Figures 2.7-27 and 2.7-28. Existing available soil boring data include those obtained from borings made in the 1967 & 1968 time frame prior to the initial excavation work as well as more recent borings made for the New Diesel Generator Project. In addition, laboratory testing provided the soil physical characteristics for foundation design. The laboratory tests program included grain size and specific gravity tests to determine the particle size and its distribution; Atterberg limits tests to determine soil plasticity characteristics; consolidation tests, to determine the soil settlement characteristics; unconfined compression and static triaxial shear tests to aid in the evaluation of the foundation bearing capacity and slope stability analyses; dynamic triaxial shear tests to determine the dynamic properties used in the evaluation of liquefaction potential of foundation materials; compaction tests; and moisture – density , void ratio and relative density determination.

The foundations for the Containment Buildings, Auxiliary Building, and Turbine Building, are mat foundations on Miocene soils. The Miocene soil is exceptionally dense and will support heavy foundation loads on the order of 15,000 to 20,000 psf. Section 2.7.5 of the UFSAR provides the foundation design contact pressures for the Containment Structure mat, Auxiliary Building mat, Turbine Pedestal mat, Turbine Building Column footings, Intake and Discharge Structure mat. For all buildings the allowable bearing capacity substantially exceeds the design contact pressures (by a factor of about two).

The original groundwater surface was between +15 feet and +20 feet above Mean Sea Level (MSL) in the plant area; however, a permanent pipe drain system and subsurface drain system surrounding the plant will maintain the groundwater below Elevation +16 feet above MSL.

Soil Liquefaction

The potential for soil to liquefy depends on soil classification (grain size distribution), its compaction level (relative density), groundwater level and the intensity of the earthquake. EPRI NP-6041-SL provides the following general definition of soil liquefaction: The pore water pressures in a saturated soil can increase under earthquake loading conditions. This increase in pore water pressure can lead to a condition of liquefaction, whereby the excess pore water pressure becomes equal to the effective confining pressure. Even if the excess pore water pressure is less than the effective confining pressure, the shear strength of the soil can reduce to a value which could result in soil failure.

A detailed probabilistic soil-structure interaction analysis and fragility calculation was performed by Stevenson & Associates (Ref. 8) for use in the CCNPP IPEEE. The report provided estimates of potential for soil failures (liquefaction potential, seismically induced permanent and transient displacements) of the grounds at CCNPP. All evaluations were performed using previously existing soils exploration data for the site. Slope stability results are mentioned in Section 2.7.6 of the UFSAR.

Bechtel Associates conducted initial field explorations and the laboratory testing of the subsurface and foundations at CCNPP. These studies included site and area reconnaissance, field supervision of the boring operations, a review of pertinent literature, and the foundation analysis and evaluation. The initial graphic boring logs and laboratory test data cited were presented in the CCNPP Preliminary Safety Analysis Report (Ref. 20).

The results of liquefaction evaluation conducted for the Power Block (Reactor Buildings, Auxiliary Building) and the Intake Structure using empirical correlations indicate that the likelihood of 100 percent pore pressure buildup is negligible for a peak ground acceleration of 0.4 g. At and below the foundation levels main structures (except the New Diesel Generator Building and the Turbine Building), the soil boring logs indicate blowcounts generally in excess of 80, relative density of 80 to 100 percent, and fine contents (passing 200 sieve size) of about 15% or larger.

For the New Diesel Generator Building, which is founded considerably higher (elevation of about +30 feet above MSL) than the Reactor Buildings, Auxiliary Building and Turbine Building, and above the groundwater table, initial liquefaction is indicated at a median peak ground acceleration of 0.27 g, with a range of 0.2 g to 0.36 g, based on the observed data from 6 test borings (Ref. 21). From the triaxial tests referred in Ref. 21, initial liquefaction is indicated for the New Diesel Generator Building to potentially occur at peak ground accelerations of 0.24 g to 0.3 g. There is stratum of an average of 9 feet thick and about 16 feet below the foundation of the New Diesel Generator Building, which is susceptible to liquefaction. Due to this plus the fact the that differential contact pressure is small compared to the existing overburden pressure prior to excavation (an increase of 1.2 ksf), stability failure is not considered a realistic hazard. Hence, the only remaining issue for the New Diesel Generator Building is liquefaction induced settlement, which is addressed in the settlement evaluation section below.

For the Turbine Building, data from one boring taken under the foundation area near the South-West corner at a depth of -25 feet to -30 feet, indicate the potential for initial liquefaction at about the reference level earthquake peak ground acceleration of 0.4 g. Other borings taken in the area beneath the Turbine Building foundation indicate liquefaction to be unlikely similar to borings taken in the areas beneath the Reactor Buildings and Auxiliary Building.

Seismically Induced Displacement and Foundation Settlement

A conservative review of the differential displacements between buildings (performed for the Reactor and Auxiliary Buildings) indicates a probability of 0.1 that the buildings will impact if subjected to the Review Level Earthquake (RLE). The RLE was defined by the median shape Uniform Hazard Spectrum for the Calvert Cliffs site from NUREG-1488 for a 10,000-year return period, at a peak ground acceleration (PGA) of 0.4 g set at 50 Hz. The PGA of 0.4 g is 4 times the associated median PGA of 0.1 g from NUREG-1488 and is 2.67 times the Design Basis Earthquake of 0.15 g.

Seismic induced settlements were estimated using empirical correlations. The settlements were computed using calculated shear stresses from the free field response analyses, as well as the Standard Penetration Test (SPT) blow counts from the test borings, suitably modified for pressure effects. In each case, the final pressure existing under the structure (gross overburden pressure minus excavation stress plus building weight pressures) were found to be almost the same as the initial overburden pressures; that is, the structures are essentially fully floating. The pressure differences were not considered significant, so that free-field induced pressures and shear stresses could be used in the settlement calculations.

For the Reactor Buildings, Auxiliary Building, and Intake Structure essentially no seismic induced settlements are indicated. For the ground without any structures, a total settlement of 0.25 inches was determined. For the Turbine Building, based on the data from one boring (area near the South-West corner of Turbine Building), a total seismic settlement of 0.5 inches is indicated. For the New Diesel Generator Building, a total seismic settlement of 0.3 – 0.75 inches is estimated. Using the correlation of shear strain and volumetric strain, a settlement of about 0.25 inches is estimated.

Slope Instability

Section 2.7.6.1 of CCNPP UFSAR discusses the slope stability evaluation of the slopes at the plant site. The slope stability evaluations of various slopes show that the factors of safety were calculated to be in the range of about 1.7 to 6.5 for static conditions and in the range of about 1.2 to 1.6 for the SSE dynamic condition.

Figure 2.7-30 of CCNPP UFSAR shows the locations and slope cross-sections around the Reactor Buildings, Auxiliary Building and the Turbine Building. The following provides the distances of the slopes from the buildings:

- Reactor Building: The ground is flat (grade elevation 45 ft) up to at least 120 ft from the building and then there is a slope.
- Auxiliary Building: The ground is flat (grade elevation 45 ft) up to at least 110 ft from the building and then there is a slope.
- Turbine Building: The ground is flat (grade elevation 45 ft) up to at least 80 ft from the building and then there are slopes towards the Chesapeake Bay.

Since the maximum factor of safety for the slopes is 1.6 for the SSE condition (0.15 g), the factor of safety for the Review Level Earthquake (RLE of 0.3 g) will be less than 1.0.

Thus some of the slopes are susceptible to failure during RLE. However, the distances of these slopes are far away from the buildings (at least 80 ft), hence the failure of these slopes will not adversely affect the Reactor Buildings, the Auxiliary Building and the Turbine Building.

The Intake Structure is located between the Turbine building and the Chesapeake Bay shoreline. Per Section 2.7.5.1 of the CCNPP UFSAR, a 300 ft segment of anchored sheet piling extends from the intake structure to the inside intake channel at the shoreline. The approximate slope of the excavation from the intake inlet to the channel junction is approximately 10 horizontal to 1 vertical. The minimum factors of safety for the intake structure were computed to be 2.7 and 1.6 for the static and dynamic (SSE) conditions, respectively. Since the minimum factor of safety is 1.6 for the SSE (0.15 g) condition, the median factor of safety for the Review Level Earthquake (RLE of 0.3 g) is expected to be at least equal to 1.0. It is judged that even if some failure of the slope occurs (movement of the soil), it will not adversely affect the Intake Structure.

The slopes near the New EDG Building were evaluated for slope stability. The factors of safety of the slope were computed to be about 1.7 and 1.2 for the static and dynamic (SSE) conditions, respectively. Since the minimum factor of safety of the slope is 1.2 for the SSE (0.15 g) condition, the factor of safety could be less than 1.0 for the Review Level Earthquake (RLE of 0.3 g) and the slope is susceptible to having some movement during the Review Level Earthquake event. However by engineering judgment the soil movement is not expected to be of such a magnitude that it will affect the New EDG Building.

3.0 Prerequisites

In accordance with the requirements noted in Section 3.3.1 of the SPID, the following prerequisites must be addressed in order to use the IPEEE analysis for seismic hazard screening purposes and to demonstrate that the IPEEE results can be used for comparison with the ground motion response spectra (GMRS):

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

Prerequisite 1:

There were no specific seismic commitments outlined in the CCNPP IPEEE. The PRA has been independently evaluated which included the treatment of seismic response.

Prerequisite 2:

As stated in the NTTF Recommendation 2.3 Seismic Walkdown Reports (Refs. 12 & 13), there were no specific seismic modifications made as a result of the IPEEE. The IPEEE and supporting documentation and calculations did take credit for modifications made as result of the USI A-46 program. A review of these modifications indicates all have been completed and remain in place.

Prerequisite 3:

No specific weaknesses were identified in the NRC Staff Evaluation Report (SER) (Ref. 16) related to the SPRA.

Prerequisite 4:

A review of plant modifications installed since the completion of the IPEEE was conducted. The review looked at major, minor, equivalency and retired in place type modifications. No modifications were identified that degraded or impacted the conclusions reached in the IPEEE. The source of the modification information was the data set used by Design Engineering to track modification status. Additionally, a review of the engineering procedures was conducted to determine the engineering control processes in place to insure the seismic qualifications of equipment was maintained thereby preventing a negative impact on the seismic portion of the IPEEE. This review identified processes/procedures in place since 1994 that require seismic reviews of modifications as well as engagement with PRA engineers if needed, via the specialty review process. The various procedures were consolidated into a two procedures CNG-CM-1.01-1003 (Ref. 25), Design Engineering and Configuration Management, and FES-007 (Ref. 26), Preparation of Design Impacts and Change Impact Screen. These procedures provide direction to engage the PRA engineers (owners of the SPRA model) during the modification process to insure the SPRA model is not degraded.

An interview with the PRA engineer also determined that no major modifications have been performed that have degraded/impacted the SPRA model. Therefore it is concluded that the IPEEE results have not been impacted by major modifications.

Based on the above, all four prerequisite from EPRI SPID have been met for Calvert Cliffs Units 1 and 2.

4.0 Adequacy Demonstration

In accordance with the guidance provided in Section 3.3.1 of the SPID, the following nine (9) Adequacy Demonstration items should be addressed:

1. Structural models and structural response analysis (use of existing or new models, how soil conditions including variability were accounted for)
2. In-structure demands and ISRS (scaling approach or new analysis)
3. Selection of seismic equipment list or safe shutdown equipment list
4. Screening of components
5. Walkdowns
6. Fragility evaluations (generic, plant-specific analysis, testing, documentation of results)
7. System modeling (diversity of success paths, development of event and fault trees, treatment of non-seismic failures, human actions)
8. Containment performance
9. Peer review (how peer review conducted, conformance to guidance, peer review membership, peer review findings and their disposition)

Each of the above items is addressed below.

4.1 Structural Models and Structural Response Analysis

Stevenson & Associates (S&A) performed the probabilistic seismic soil-structure interaction (SSI) analyses in Ref. 8 for the horizontal earthquake components using the original two-dimensional (2D) stick models developed for the Reactor Building, Auxiliary Building, and Intake Structure. The original models were developed by Bechtel using the Atomic Energy Commission publication TID-7024 (Ref. 14) as a basic design guide. For the Turbine Building a simplified model was developed by S&A, since no structural model was available. The New Emergency Diesel Generator (EDG) building used the model developed for the Diesel Generator Building seismic analysis. The global stick models are considered to adequately model the global dynamic behavior of the structures up to a frequency of 33 Hz.

The original Single Degree of Freedom (SDOF) models were used for the vertical SSI analyses. The soils stiffnesses and soil dampings of these models used the same effective soil shear modulus values and damping computed for the horizontal SSI analyses. Vertical floor response spectra (FRS) were computed using a time history analysis. The radiation damping for vertical translation was limited to 60% for the Auxiliary Building and Turbine Building.

Structural damping was taken as 7% for both steel and concrete elements of the structural model, and 5% for equipment. The cutoff frequency for the SSI analyses was 30 Hz. For the Reactor Building, Auxiliary Building, Intake Structure, and Turbine Building seismic torsional and three-dimensional (3D) effects were incorporated in the probabilistic analyses by introducing additional randomness and uncertainty factors.

EDG Building Floor Response Spectra (FRS) were obtained directly from the 3D stick model using modal time history analysis with composite modal damping limited to 20%.

The results of the SSI analyses included development of probabilistic in-structure response spectra for use in calculating equipment fragilities, and probabilistic modal forces and moments in the structure models for calculating the structural fragilities. The Review Level Earthquake (RLE) used to determine the structural response was defined by the median shape Uniform Hazard Spectrum (UHS) for CCNPP from NUREG-1488 for a 10,000-year return period at a PGA of 0.4 g set at 50 Hz.

For the SSI analyses the use of a cutoff frequency of 30 Hz drops the Zero Period Ground Acceleration (ZPGA) to lower values. This does not affect the amplitude of the FRS at least up to 30 Hz. The 30 Hz cutoff frequency is considered to be realistic for the actual site soil conditions, including the nonlinear hysteretic soil behavior under a severe earthquake. The significant SSI modes have lower frequencies, and are practically insensitive to the frequencies higher than 30 Hz. Moreover, the stick models are considered to be less representative for the actual dynamic behavior of structures for frequencies higher than 30 Hz.

For the characterization of the soil deposit randomness two basic variabilities were considered for each soil layer: (i) the value of the shear modulus at low strains, and (ii) the shape of the normalized shear modulus-shear strain curve. The random variability of the soil shear modulus was considered based on experimental data used for the original Calvert Cliffs design calculations. In SSI models, the effect of the randomness in soil hysteretic damping is less significant than in the soil stiffness (the radiation damping overshadows the soil material hysteretic damping) and therefore was not considered. The random variations of shear modulus values with depth were considered to be perfectly correlated, i.e. the soil layers are considered uniformly soft or uniformly hard.

EPRI NP-6041-SL requires that the seismic models adequately account for mass, stiffness, damping, co-directional responses, three-dimensional (3D) model vs. two-dimensional (2D) model, and torsion effects. To account for these items, in addition to soil deposit randomness, the analysis included variabilities for motion incoherence, 3D effect, torsion effect, earthquake component combination, structural system, mass, damping, and stiffness.

Structural Model and Structural Response Analysis Review Conclusion

Based on the above summary of the methodology used for the structural models and structural response analysis, the structural seismic models used for generation of ISRS for IPEEE evaluations meet the requirements of NUREG-1407 and EPRI NP-6041 and are adequate for screening purposes.

4.2 In-Structure Demands and ISRS

Per the *Probabilistic Soil Structure Interaction Analysis*, the Monte Carlo simulation technique was used to determine the probabilistic seismic response of the CCNPP buildings. The Monte Carlo simulation appropriately represents the nonlinearities of the structural dynamic response from both the material behavior and the frequency dependent nature of the structural response. This is particularly important when seismic

SSI effects are significant. For the in-structure response spectra (ISRS), the relationship between spectral amplitudes and the input soil or structure stiffness parameters is heavily nonlinear especially near the resonant frequencies which are of primary interest.

The use of deterministic SSI models to evaluate the median ISRS, may give unrealistic spectral shapes with sharper peaks and valleys than the statistically estimated median ISRS, as illustrated in the ASCE publication on "Uncertainty and Conservatism in the Seismic Analysis in the Design of Nuclear Facilities", 1986 (Ref. 17). Thus, by using the Monte Carlo sampling technique, appropriate statistical estimations of ISRS (including median ISRS) and structural forces and moments were computed.

For the probabilistic ISRS development, simulated ISRS for randomly sampled soil stiffnesses at selected elevations in different buildings were developed. Simulated ISRS for the Reactor Building, Auxiliary Building, and Intake Structure showed that the random variation of soil stiffness is more significant for the Reactor Building. The ISRS standard deviation is considerably larger for the Reactor Building which indicates that the randomness in SSI is more important for the ISRS in this building. The ISRS standard deviation has spectral peaks with large amplitude at the upper elevations of the Reactor Building, but at lower elevations the standard deviation drastically drops. This indicates that the SSI effects are less significant for the evaluation of horizontal ISRS near the basemat and that random SSI effects are manifested primarily through rocking motion rather than horizontal translation.

The Median and 84 Percentile ISRS including all randomness variabilities and composite variability for all buildings and elevations computed using Monte Carlo sampling were developed. For the development of ISRS, in addition to soil randomness, the effects of seismic excitation and modeling uncertainties were also included.

In-Structure Demand and ISRS Review Conclusion

Based on the material presented above and adequacy of the structural models discussed in Section 4.1, the in-structure demands and ISRS meet the requirements of NUREG-1407 and EPRI NP-6041 and are adequate for screening purposes.

4.3 Selection of SSEL

The component walkdown list is compiled from the components in the CCPRA component database, Safe Shutdown Equipment Lists (SSEL) from USI A-46 program, NUCLEIS information management system and the Q-list. The CCPRA component database was used for the IPE Internal Flooding Analysis. This database lists level 1 PRA system components and their locations. The SSEL was developed for USI A-46, "Verification of Seismic Adequacy of Mechanical and Electrical Equipment in Operating Reactors." The Q-list contains safety related components. Some Q-list components are duplicated in the CCPRA component database. All the non-duplicated components were retained for initial screening in the seismic walkdown component list. NUCLEIS is Calvert Cliffs' information management system. The NUCLEIS Equipment module contains the locations for most plant components. It is used to identify component locations not available in either the CCPRA component database or in the Q-list.

Initial screening is performed on these components as described in IPEEE Seismic Component Walkdown List Development (Ref. 5). For example, check valves are inherently rugged and are screened from the walkdown list. In addition, systems such as Main Feedwater, Condensate, and Circulating Water are not walked down because of their low importance in the PRA. Components that passed this initial screening comprise the walkdown list and were walked down.

The Seismic IPEEE is closely coordinated with USI A-46. Most of the components included in the Seismic IPEEE walkdown are also on the USI A-46 Safe Shutdown Equipment List. The scope of the Seismic IPEEE walkdown includes the additional components necessary to supplement the work done during the USI A-46 walkdown.

The scope of the walkdown also includes the identification of higher capacity equipment and components which may be screened out from explicit consideration in the CCNPP SPRA.

Attachment A, in the IPEEE Seismic Component Walkdown List Development contains a complete component walkdown list. For each component the following information is provided:

- If the component is also on the USI A-46 SSEL
- The building and room where the component is located
- If the component meats the caveats of EPRI NP-6041-SL
- If the equipment anchorage has a safety factor greater than 2 with respect to USI A-46 criteria
- If the component has seismic interaction, fire or flood concerns
- The component screening level (0.3 g or 0.5 g)
- If the Unit 2 component is similar to the Unit 1 component

A review of the screening results from Attachment A to the IPEEE Seismic Component Walkdown List Development indicates that all of the SSEL components from USI A-46 as well as the additional CCNPP SPRA components were selected and screened appropriately. The additional components not screened out by EQE were added to the walkdown list for review and resolution.

Selection of SSEL Review Conclusion

Based on the above, the methodology used to develop the SSEL is in compliance with NUREG-1407 and results are adequate for screening purposes.

4.4 Screening of Components

For screening, the guidelines of Table 2-4 of EPRI NP-6041 are used. The guidelines are set to screen earthquakes of about 0.3g and 0.5g peak ground acceleration (0.8 g and 1.2 g peak spectral acceleration). For USI A-46, the equipment is evaluated for the plant's safe shutdown during an earthquake with a peak ground acceleration of 0.15 g. Therefore, equipment and components are screened as follows:

- At 0.3 g peak ground acceleration, if caveats of column 1 of EPRI NP-6041 are met and the USI A-46 anchorage calculation showed a factor of safety greater than two.
- At 0.5 g peak ground acceleration, if caveats of column 2 of EPRI NP-6041 are met and the USI A-46 anchorage calculation showed a factor of safety greater than four.

One exception to the above has to do with seismic interactions, particularly block walls. For seismic interactions, the walkdown must note potential seismic interactions for earthquake levels above the Safe Shutdown Earthquake (SSE). These are not documented in the USI A-46 walkdowns. For block walls, nearby equipment and components could be screened at 0.3 g if the wall is qualified by elastic analysis for IEB 80-11 (Ref. 18). It is assumed that all walls are reinforced and anchored in accordance with BGE Drawing 62-128-E, "Masonry Details." Equipment near walls qualified by inelastic analysis could be screened at 0.3 g if the Seismic Review Team (SRT) judged that there is sufficient margin based on the wall height and thickness. In no case is equipment near masonry walls screened at the 0.5 g level.

For seismic-fire and seismic-flood interaction, potential fire and flood sources are identified by BGE and then evaluated per EPRI NP-6041 guidelines at a review level of 0.3 g. Those sources not screened out are identified for further review and possible fragility analysis.

The walkdown lists and screening results are contained in Attachment A to the IPEEE Seismic Component Walkdown List Development. Attachment A is a complete walkdown and screening report prepared by EQE. It contains sections on methodology, the seismic review team, screening, seismic-fire interaction and seismic-flood interaction.

A review of the screening results from Attachment A to the IPEEE Seismic Component Walkdown List Development indicates that all of the SSEL components from USI A-46 as well as the additional CCNPP SPRA components were screened appropriately. The additional components not screened out by EQE were added to the walkdown list for review and resolution. Walkdown results captured resolution of the specified issues.

Screening of Components Review Conclusion

NUREG-1407 directs that the walkdown follow the guidelines of EPRI NP-6041-SL, Rev. 1. The guidelines contained therein are used for both the walkdown and the screening out of higher capacity components.

Based on the material presented above, the component screening meets the requirements of NUREG-1407 and EPRI NP-6041 and the results are adequate for screening purposes.

4.5 Walkdowns

NUREG-1407 directs that the walkdown follow the guidelines of EPRI NP-6041-SL. The guidelines contained therein are used for the walkdown and the screening out of higher capacity components. The walkdowns were conducted by a Seismic Review Team (SRT) consisting of personnel from EQE and CCNPP.

The purposes of the walkdown were to: 1) visually inspect and screen-out inherently rugged components from further review, 2) define the failure modes (such as anchorage failure) and elements which are not screened, and 3) add to the analysis any seismic interaction items judged to be a potentially serious problem. The scope of the walkdowns and screening covered only equipment and components. Structure and soil analyses were performed separately by S&A and are discussed in Section 4.1.

The component walkdown list is compiled from the components in the PRA database, Safe Shutdown Equipment Lists (SSEL) from USI A-46 program, NUCLEIS information management system and the Q-list. Initial screening is performed on these components as described in IPEEE Seismic Component Walkdown List. For example, check valves are inherently rugged and are screened from the walkdown list. In addition, systems such as Main Feedwater, Condensate, and Circulating Water are not walked down because of their low importance in the PRA. Components that passed this initial screening comprise the walkdown list and were walked down.

The Seismic IPEEE is closely coordinated with USI A-46: Most of the components included in the Seismic IPEEE walkdown are also on the USI A-46 Safe Shutdown Equipment List. The scope of the Seismic IPEEE walkdown includes the additional work necessary to supplement the work done during the USI A-46 walkdown.

The scope of the walkdown also includes the identification of higher capacity equipment and components which may be screened out from explicit consideration in the CCNPP SPRA. As specified in NUREG-1407, guidelines of EPRI NP-6041 are used for both the walkdowns and the screening out of the higher capacity components.

A review of the walkdown results from the Ref. 5 indicates that all of the outliers from USI A-46 and the remaining CCNPP SPRA components were walked down and dispositioned appropriately. The additional components reviewed by EQE had HCLPF calculations performed as required

Walkdowns Review Conclusion

Based on the material presented above, the seismic walkdowns meet the requirements of NUREG-1407 and EPRI NP-6041 and results are adequate for screening purposes.

4.6 Fragility Evaluations

EPRI TR-103959 (Ref. 4) and EPRI NP-6041-SL were used as design guides for developing Structural Fragilities. They are in compliance with NUREG-1407 and are deemed adequate for screening.

Structural fragilities for CCNPP structures and buildings identified above were calculated for the use in CCNPP IPEEE SPRA. Fragilities were calculated using state of the art and current standard industry SPRA methods of the day. The seismic hazard curves for the CCNPP site were developed by EPRI and Lawrence Livermore National Laboratory (LLNL) (Ref. 9). In accordance with NUREG-1407 the higher LLNL curves were used for the analysis. Thus, for CCNPP buildings the Review Level Earthquake (RLE) is defined by the median shape Uniform Hazard Spectrum (UHS) from NUREG-1488 for a 10,000-year return period. Fragilities are expressed in terms of peak ground accelerations, and HCLPF values are provided as well.

EQE developed seismic fragilities of selected equipment for the Calvert Cliffs Nuclear Power Plant (Ref. 6). These fragilities were used in the seismic probabilistic risk analysis (SPRA) as part of the IPEEE. EQE used a screening approach to select the most critical components for fragility development. It was deemed impractical to perform fragility analysis for every SSC listed in the plant system fault trees; the components that control the frequency of core damage were evaluated. The rest of the components are either relatively strong or have been screened out by system considerations.

Fragility evaluations for buildings are performed by the firm of Stevenson and Associates, and they are documented in Ref. 8. Fragility evaluations for components are performed by the firm of EQE International, and they are documented in Ref. 6. An assessment of the methodology used for building and component fragility calculations is provided below.

Building Fragilities

Per Section 3.1.3.1 of Ref. 1, the following buildings required fragility evaluation:

- Containment
- Auxiliary Building
- Intake Structure
- Turbine Building
- New Emergency Diesel Generator Building
- Fire Pump House
- Condensate Storage Tank and Fuel Storage Tank Enclosures

Section 3.1 of Ref. 8 describes the approach used for building fragility evaluations. This consists of:

1. Linear response spectrum seismic analysis of dynamic model of each building that considers SSI. The input spectrum for each horizontal direction was median UHS with peak ground acceleration of 0.4g. Vertical direction spectrum was 2/3 of horizontal direction spectrum. As a result of these analyses, median forces applicable to each potential critical failure mode were determined.
2. Utilize median capacity equation for each failure mode to calculate median safety factor for the mode of failure under consideration.
3. Use modification for inelastic energy absorption, if applicable, to modify the median safety factor.

4. Calculate median g-rating of the mode of failure by multiplying the median safety factor time 0.4g in Item 1 above.
5. Estimate logarithmic standard deviation values for randomness β_r and uncertainty β_u for the mode of failure under consideration
6. Determine high confidence low probability failure (HCLPF) capacity of the failure mode using Items 4 and beta values of Item 5 in logarithmic distribution formulation typically used in SPRA work.

The application of above steps to the various buildings is documented in Appendix F of Ref. 8. The approach used to obtain main results for two buildings is reviewed below.

For Auxiliary Building main results are provided in Tables 3.3 through 3.6 of Ref. 8. The calculated median capacity is 1.19g in Table 3.6. This results from flexural failure of Reinforced Concrete (RC) wall at elevation 69'-0" of the Auxiliary Building. A review of calculation on Page F2-18 of Appendix F in Ref. 8 shows reasonable application of required steps for median capacity evaluation. Also based on estimated beta values in Table 3.6 of Ref. 8, the HCLPF capacity estimated is 0.45g for the Auxiliary Building.

For Turbine Building the main result is provided in Table 3.11 of Ref. 8. The calculated minimum capacity is 2.88g. This results from bending failure of structural column at its base at elevation 45'. A review of calculation on Page F5.1-15 of Appendix F of Ref. 8 shows reasonable application of required steps for median capacity evaluation. Also discussions on Pages F5.1-17 through F5.1-21 of Ref. 8 show reasonable selections for the beta values used. The HCLPF capacity for the Turbine Building is calculated to be 1.02g.

Based on the above, the methodologies used for the calculation of HCLPF capacity of the Auxiliary Building and Turbine Building are considered acceptable and the HCLPF capacities obtained for these two structures are reasonable.

Component Fragilities

EQE International used a screening approach to select the most critical components for detailed fragility evaluation. Section 2 of Ref. 6 describes the details of this screening process. A summary of this screening is provided below:

1. All USI A-46 components that had a USI A-46 factor of safety of 2 or greater relative to 0.15g SSE peak ground acceleration were screened out from detailed fragility evaluation, because this will enable them to be screened out in IPEEE program utilizing the first column in Table 2-4 of EPRI NP-6041-SL.
2. The items not screened out under Item 1 above, including the USI A-46 outliers, and the IPEEE components not included in USI A-46 program were subjected to a screening using the following equation for estimation of their HCLPF

$$HCLPF = FS * \frac{Sa_{A-46}}{Sa_{84}} * 0.4g$$

In this equation,

FS = factor of safety under USI A-46 evaluation

$Sa_{A\text{-}46}$ = spectral acceleration from USI A-46 spectrum at equipment frequency

Sa_{84} = 84^{th} percentile spectral acceleration from Ref. 8 at equipment frequency

Derivation of the above equation is shown in Section 2 of Ref. 6. Table 1 of Ref. 6 lists the results obtained through using the above HCLPF value for the equipment evaluated. The equipment with HCLPF less than 0.3g in Table 1 are listed in Table 2 of Ref. 6. These equipment items, if could not be ruled out either because of planned modification or replacement, were subjected to detailed fragility calculation. Results are summarized in Table 3 of Ref. 6.

Supporting calculations for fragility information provided in Table 3 of Ref. 6 are provided in Ref. 7. As examples, the approach used for two equipment items from Ref. 7 follows;

Item 1 in Table 3 of Ref. 6 includes the 120V Distribution Panels. The concern for fragility is for the evaluation of anchorage of these panels. This anchorage evaluation is documented in Section 4.2 of Ref. 7. There are 10 different cabinets in this group for the units. The cabinets are each 14.5" deep x 32" wide x 90" high. They are in one bay or two bay configurations. The critical case considered is the cabinet in one bay configuration with only two bolts, one in each opposite corner (Page 4.2-1 of Ref. 7). The median factor of safety is reasonably calculated as 1.9 (Page 4.2-3 of Ref. 7). This leads to median g-rating of 0.76g, shown on Page 4.2-7 and reported in Table 3 of Ref. 6. The beta values for the various considerations are reasonably discussed and listed on Pages 4.2-3 through 4.2-7 of Ref. 7. These beta values are also listed in Table 3 of Ref. 6. The calculated HCLPF for this item based on median capacity of 0.76g and the listed beta values is 0.15g, as indicated in Table 3 of Ref. 6 and documented on Page 4.2-7 of Ref. 7.

Item 4 in Table 3 of Ref. 6 includes results for SG Blowdown and Waste Sampling Hoods for Units 1 and 2. Section 4.6 of Ref. 7 covers the anchorage fragility evaluation for this group of hoods, Page 4.6-1 lists the hoods and the configuration of anchorage. The Unit 2 anchors are ¼" shell anchors. Unit 1 anchors are 3/8" shell anchors. Total weight of each cabinet is assumed to be 3 times the cabinet weight (Page 4.6-2), which is a reasonable assumption. Because center of gravity is not known, it is conservatively assumed to be at ¾ time the cabinet height of 96".

Calculation of safety factor for each unit and each cabinet type is shown on Pages 4.6-2 through 4.6-5 of Ref. 7. These calculations are reasonably performed. Results for median safety factors are tabulated on Page 4.6-5 for both units and anchor types. The use of these factors with the 0.4g peak ground acceleration leads to median g-ratings that are listed in Table 3 of Ref. 6. The beta values for the various factors are discussed and quantified on Pages 4.6-6 through 4.6-9 of Ref. 7. The combined beta values are computed and listed on Page 4.6-9 of Ref. 7, and they are entered in Table 3 of Ref. 6.

The calculated HCLPF values are also listed in Table 3 of Ref. 6 with a note that Unit 2 values are low because of ¼" anchors.

Based on the above, the methodologies used for the calculation of HCLPF capacity of the 120V Distribution Panels and SG Blowdown and Waste Sampling Hoods are considered acceptable and the HCLPF capacities obtained for these components are reasonable.

<u>Fragility Evaluations Review Conclusion</u>

Based on the material presented above, it is concluded that the methodology used to perform fragility/HCLPF calculations is in compliance with NUREG-1407 and EPRI NP-6041-SL and the results are adequate for screening purposes.

4.7 System Modeling

The Calvert Cliffs PRA (CCPRA) model used for the IPEEE was a linked event tree model. The general transient event tree was used to quantify seismic core damage frequency. A seismic event tree was added in front of the model to assess the probability of system failures due to the seismic event. The impact of the seismic event tree top event failures are then cascaded into the general transient event trees to reflect the dependencies of the seismic failures on the downstream top events. For example, the Service Water System is susceptible to seismic induced failure and is represented by Seismic Top Event LG in the Seismic Event Tree. When Top Event LG fails, the corresponding Service Water Top Event in the General Transient Event Tree is failed. This in turn fails the service water cooled Emergency Diesel Generators and other Service Water dependent components. A similar linkage exists for all of the Seismic Top Events and their dependent components.

Plant equipment on the equipment list that did not screen out based on equipment walkdowns had fragilities performed. Each susceptible system is modeled as a top event in the Seismic Event Tree. Each Seismic Top Event has a probability of seismic induced failure corresponding to the level of the seismic initiating event, as defined by its fragility. The seismic top events modeled are: Surrogate (represents total plant failure and results in core damage), Refueling Water Storage Tank, 0C Diesel Generator, Service Water, Control Room HVAC, 500 kV Switchyard, Secondary Systems, and Containment Isolation.

Each seismic initiating event is quantified one at a time. Seismic induced failures may occur, but note also that each component and system is also subject to random failure or unavailability due to maintenance. So the seismic event is just one possible cause of failure. A seismic core damage sequence may consist of both seismic and non-seismic induced failures.

Since we expect a seismic event to adversely affect operator performance, human action failure rates used in the Internal Events CCPRA are adjusted for seismic scenarios. Human action failure rates are calculated using performance shaping factors

(PSFs). Those PSFs that were judged to be affected by a seismic event are multiplied by influence factors to account for the lower success rate following a seismic event.

Different influence factors are used for human actions depending on whether it is a short term or long term action. Short term actions are those that must be completed within 15 minutes and all others are considered long term actions. A seismic event is considered to have a greater effect on short-term actions because of the initial shock imposed by the seismic event and the more limited amount of time to complete the action.

Specific adjustments are made to performance shaping factors depending on whether the PSF is judged to be time dependent, time independent or g-level dependent. Some PSFs were judged to be unchanged, for example, procedure quality and adequacy. The influence factors are scaled according to the strength of the seismic event. They range from no impact at 0 g (no seismic event) to guaranteed failure at 1.0 g for short term actions and 2.0 g for long-term actions.

System Modeling Review Conclusion

The CCNPP seismic PRA was performed using the approaches identified in NUREG-1407. For example, Section 3.1.1.1 states that a mean point estimation using a single hazard curve (rather than a family of hazard curves) and a single fragility curve (rather than a family of fragility curves) for each component is sufficient to get insights into potential seismic vulnerabilities. The CCNPP SPRA uses this methodology. Each Structure/System/Component (SSC) in the SPRA has a mean fragility curve based on a median capacity and composite uncertainty.

The methodology and results of the system modeling aspect of the CCNPP SPRA are adequate for screening purposes. The CCPRA Internal Events model, upon which the SPRA is built, was peer reviewed by an industry review team following the Combustion Engineering Owners Group (CEOG) industry peer review process described in WCAP-15801 (Ref. 19). The peer review took place in November 2001. The review team concluded that the Calvert Cliffs PRA Systems Analysis met all High Level Requirements.

In summary, the methodology used to develop the system modeling is in compliance with NUREG-1407, and the methodology and results are adequate for screening purposes.

4.8 Containment Performance

Containment penetrations and containment isolation valves are screened at 0.5g. Seismic-induced failure of containment penetrations or containment isolation valves result in a failure of the containment isolation function. This function is modeled by surrogate Top Event LL in the Seismic Event Tree. For CCNPP, a large release is defined as a break greater than a 4-inch diameter hole. Top Event LL is conservatively mapped to large release for all failures by making Top Event SI (containment penetrations greater than four-inch function) dependent on Top Event LL success.

The fragility assigned to the containment isolation function is based on the screening level used by the Seismic Review Team during the component walkdowns and the guidance provided by EQE in the fragility report (Ref. 6). This guidance states that for the purposes of defining surrogate element fragilities, components mounted in structures should conservatively have a median HCLPF ratio of about four. Containment electrical penetrations and piping penetrations are screened at HCLPF of 0.5 g. Containment isolation valves are also walked down and all those whose failure could lead to containment bypass or release are also screened at 0.5g. Therefore, Top Event LL is assigned fragility with a HCLPF of 0.5g. Using the recommended median/HCLPF ratio of four yields a median acceleration capacity of 2 g.

The randomness, β_r, and uncertainty, β_u, are assigned as 0.40 and 0.44, respectively, based on the guidance given for establishing surrogate fragilities in Ref. 6.

The seismic structure fragility analysis (Ref. 8) determined a median acceleration capacity and HCLPF value for the containment shell, and the reinforced-concrete base slab. These values are shown in Table 3-2 of this report. The base slab is the most limiting and has a median acceleration capacity of 2.31 g and HCLPF of 0.70 g. This is bounded by the surrogate Top Event LA, which leads directly to core damage and the Containment Isolation Top Event LL, which is mapped to a large (greater than four-inch diameter) leak in containment. Therefore, containment structural failures are not modeled in this evaluation.

Containment Performance Review Conclusion

The containment performance methodology described above complies with Section 3.2.6 of NUREG-1407. Component screening was performed at 0.5 g and containment isolation valves were walked down for interactions. Top Event LL addresses seismic impacts on containment isolation functions and, as stated above, Top Event LA bounds the limiting structural component.

Based on compliance with NUREG-1407 and no peer review comments indicating issues with the treatment of containment performance, the results presented in the IPEEE remain adequate for screening purposes.

4.9 Peer Review

Peer review guidelines are provided in Section 7 of NUREG-1407. In accordance with these guidelines, the peer review should be conducted by individuals who are not associated with the initial evaluation. In addition, the peer review team should include plant staff and the IPEEE submittal should include a description of the review performed, the results of the review team's evaluation, and a list of the review team members.

IPEEE Program Organization

Calvert Cliffs Nuclear Power Plant's IPEEE was developed by a project team consisting primarily of a project manager and eight BGE PRA engineers. For each major task (seismic, fire, and other external events) there was a lead project engineer. Several of the project team members were actively involved in developing the CCNPP IPE and therefore had previous knowledge of PRA methodology.

Composition of Independent Review Teams

The IPEEE analysis received four levels of review:

1. Selective independent peer reviews of assumptions, methodology and technical calculations by other IPEEE team members who were not involved in the original analysis.
2. All technical work performed by outside contractors was reviewed by the IPEEE team members to ensure technical knowledge transfer and accuracy of results.
3. High level peer reviews by recognized experts.
4. Selective independent peer reviews by discipline engineers who are familiar with the subject.

The following discussion provides a general description of how these four levels of review were applied for seismic external event initiator.

The component fragility calculations were reviewed by EQE and were owner acceptance reviewed by a BGE IPEEE project team member. EQE's internal review was performed in accordance with their QA procedures in order to meet BGE's safety-related purchase order requirements. Similarly, Stevenson & Associates conducted an internal review of soil liquefaction, SSI, and structural fragility analyses. A BGE IPEEE team member provided an owner acceptance review on these structure related analyses. A BGE senior civil-structural engineer who was involved in the seismic USI A-46 project also provided a limited review on the analyses performed by EQE and Stevenson & Associates. In addition, ERIN Engineering and Research provided an overall peer review on all seismic related analyses. The peer review comments and their resolutions are included in Table B1 (Section 6.3 of Ref. 1).

Areas of Review and Major Comments

Table B1 provides a matrix of subject, comments and resolutions that resulted from the internal and external IPEEE analysis peer review either by the project team or contractors. Only seismic comments and resolutions are included in Table B1.

Peer Review Conclusion

Based on Section 7 of NUREG-1407 which outlines the peer review requirements, as well as Section 3.1.1.1 General Considerations, the peer review conducted for the IPEEE was performed in accordance with the NUREG-1407 requirements. The team consisted of both in house engineers supplemented by consultants with specific seismic and PRA knowledge. Additionally, an independent contractor, with seismic expertise, conducted a review of the seismic sections of the IPEEE.

Based on the above, the peer review was conducted by qualified personnel who were not associated with the initial evaluations and their findings/concerns were tracked and addressed by initially screening in relevant components. Thus, the peer review meets the NUREG-1407 requirements and the results are adequate for screening purposes.

5.0 Conclusions

The NRC 50.54(f) letter has requested all nuclear power plant licensees to conduct seismic hazard reevaluations using updated seismic hazard information and present-day methods. CCNPP is performing the seismic hazard and screening per the EPRI SPID guidance. A new GMRS has been developed for Calvert Cliffs using the SPID, RG 1.208, and NUREG-2115 (Ref. 27) guidelines. Using the SPID guidelines, the GMRS can be compared to the IHS to screen out of future seismic risk assessments. In order to perform the GMRS to IHS screening, the Calvert Cliffs IPEEE is subject to an adequacy review to ensure that the IPEEE is of sufficient quality. This report documents the adequacy review performed following the guidance provided in Section 3.3.1 of the SPID.

The SPID defines four categories which must be addressed in order to use the IHS for seismic hazard screening. The four categories are:

- General Considerations
- Prerequisites
- Adequacy Demonstration
- Documentation

Calvert Cliffs is a focused scope plants binned to 0.3 g PGA per NUREG-1407. The IPEEE seismic assessment was performed using a SPRA per the EPRI NP-6041-SL methodology. The SPID IPEEE adequacy "General Considerations" requires that focused scope plants perform full scope evaluations of soil failure modes and relay chatter. NEI Letter "Relay Chatter Reviews for Seismic Hazard Screening" dated October 3, 2013 states that full scope relay chatter reviews will be performed later on a schedule consistent with high frequency evaluations. Therefore, relay chatter is not addressed in this report and will be evaluated later.

Soil failure modes were evaluated in Section 4. The results of this evaluation conclude that liquefaction, slope stability, and settlement are not a concern.

The four IPEEE adequacy Prerequisites were reviewed. Prerequisites 1 to 3 were confirmed to be acceptable. Prerequisite 4 reviewed major plant modifications which could degrade / impact the conclusions reached in the seismic IPEEE. A review of the major modifications indicates that none of the major modifications has an adverse impact on the IPEEE conclusions. Prerequisite 4 is acceptable.

The nine Adequacy Demonstration items defined in the SPID were reviewed based on available information from the IPEEE submittal and additional available backup reference information. All nine items were found to be adequate for seismic hazard screening purposes.

This report documents the IPEEE Adequacy review for Calvert Cliffs Units 1 and 2 which was performed following the guidelines of the SPID, section 3.3.1.

Therefore, the Calvert Cliffs seismic IPEEE is adequate for seismic hazard screening and the IHS can be used for screening of the new GMRS.

6.0 References

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Table B1 –SEISMIC IPEEE Peer Review Comment and Resolutions (Section 6.3 of Ref. 1)

	Comment	Resolution
Seismic	Top Event LK is a surrogate top event included as the last top event in the seismic event tree to model seismic failure of the secondary systems. The intent of Top Event LK is to screen/eliminate sequences from the model that are low intensity, have no associated seismic failures, and are therefore not initiating events (i.e., do not cause a plant trip). The current use of Top Event LK to achieve this will be difficult to defend. Top Event LK should be eliminated from the model and top events that model the secondary systems should be assumed failed for all seismic events.	The Top Event LK fragility is based on the judgment that the seismic capacity of the modeled systems is likely to be as good as or comparable to the relatively low-capacity switchyard fragility. The switchyard (offsite power) fragility is the most limiting seismic top event (Top Event LJ). The median capacity for LK is set to one half of that for LJ. The randomness (Br) and uncertainty (Bu) are conservatively assigned a value of twice that for LJ. In addition, LK is only used for seismic initiating events at or below the Operational Basis Earthquake (OBE = 0.08G). Top Event LK is set to guaranteed fail for all seismic initiators above this level.
Seismic	The split fraction value for Top Event LK is based on a fragility family with a median acceleration of approximately 0.1. There is no basis for this median acceleration.	The intent of Top Event LK could have been achieved by truncating the low end of the fragility curves at a higher level, such as the HCLPF value. This has been done in some PRA's and was suggested for Calvert Cliffs. The HCLPF corresponds to about 1% failure fraction. The fragility curves in the Calvert Cliffs PRA are truncated 0.005%. Truncation at the HCLPF was considered but no basis could be found. Top Event LK achieves a result similar to truncation at the HCLPF, although not as dramatic. Truncation at the HCLPF would mean that all seismic top events would be guaranteed success for seismic initiators at or below 0.08pga. We believe that the use of Top Event LK with the current fragility curve truncation provides more realistic modeling than truncation at the HCLPF. Also, most of the functions LK represents are already modeled in the internal events model by initiating events such as Loss of Main Feedwater, Loss of Condenser Vacuum, or Loss of Instrument Air. These initiating events have comparable or greater frequencies than the three lowest-g seismic initiating events used with LK. See Section 3.1.5.2 for additional details on the use of LK.
Seismic	Equipment judged to have a HCLPF peak ground acceleration of 0.3g or greater were screened from the analysis. Surrogate Top Event LA was used to consider all components screened out based on this criteria. Top Event LA represents five different systems, each with a HCLPF of 0.3g, and the condensate storage tank. The failure of Top Event LA is calculated as an "OR" gate with inputs from each of the six system failures. Failure of Top Event LA is mapped directly to core damage. There are non-conservative issues associated with this treatment such as seismic induced fires and floods, relay chatter, LOCA initiators, and containment bypass sequences. Options to consider: 1) remove the surrogate top event	The suggested options were incorporated into the analyses. The Seismic event trees were quantified both with and without surrogate Top Event LA. The quantification without LA is used for evaluating sequences and may be used for relative risk contributions. Also, the containment isolation function was broken out of Top Event LA and added back in as a second surrogate top event.

	from the presentation of realistic risk contributions of seismic events and add it back in as a sensitivity analysis but use two top events instead of one, 2) one for equipment needed to protect the core, and another to prevent a large early release.	
Seismic	It is recommended that the final report sections on both seismic induced fires and floods draw specific conclusions on the existence or non-existence of unscreened components whose failure has the potential to create fires or floods that could cause additional damage and dependent failure effects beyond the direct consequences of the earthquake.	This recommendation was incorporated. Seismic-fire interaction, Section 3.1.3.3, was expanded considerably and describes the risk associated with the unscreened components.
Seismic	The generic dismissal of RCS components and piping and associated penetrations may have bypassed adequate consideration of the potential for large early containment isolation failures and bypass events.	See response below
Seismic	Piping and penetration induced bypasses appear to have been generically dismissed. Hence, the potential for plant specific vulnerabilities has not been addressed. The NUREG-CR-4551 analyses for Surry is one example where plant specific vulnerabilities were identified.	See response below
Seismic	The exclusion of seismic induced LOCAs is based on a generic methodology document provided by EQE and not a plant specific evaluation. These components were excluded from the IPEEE walkdown list (see page A67 of the walkdown evaluation sheets; i.e., Ref.3-3. Is this justified to dismiss these events generically?	Most of the piping penetrations were walked down by the SRT and are screened at 0.5 g. However, the large NSSS components (Rx vessel, pressurizer, RCP's, steam generators) are generically screened based on the screening criteria in Table 2-4 of EPRI NP 6041-SL (Ref. 3-4). These guidelines are based on NUREG/CR-4334. Reference 3-4 states that they are based on "a general industry and regulatory consensus that, in fact, there are wide classes of elements in nuclear power plants which have demonstrated a substantial seismic ruggedness either because of their performance in past earthquakes, available generic ruggedness or fragility data, or because generally accepted seismic margin capacity evaluations have been performed on like elements in previous seismic margin or SPRA elements". Appendix A of Reference 3-4 outlines the basis for the screening guidelines. The criteria in Table 2-4 of EPRI NP 6041-SL is that evaluation of NSSS supports is not required if supports are designed for combined loading determined by dynamic SSE and pipe break analysis. Calvert Cliffs meets this requirement. Per the Calvert Cliffs Updated Final Safety Analysis Report, Reference 3-23, the RCS is designated a seismic Class 1 system for seismic design and is designed for three categories of load combinations and stress. Category three is Normal Operating loadings + pipe rupture + Safe Shutdown earthquake. Under this loading condition, deflection of the NSSS supports is limited to maintain supported equipment within limits (given in Reference 3-23). One of the peer reviewer concerns was that the potential for large-early-release failures caused by failure of the NSSS components may not have been adequately addressed. The current modeling uses containment isolation top event LK which is mapped to a large break in containment. Results show about 81 % of seismic CDF results in late containment failure, and about 14 % of seismic CDF is shared between large

		and small early release (see section 3.1.6.1). So although NSSS component failure is not explicitly modeled, Top Event LK serves to represent the impact of this failure type. This peer review comment was also discussed with the lead SRT expert. He feels that the screening was appropriate. In addition, he pointed out that the equipment arrangement inside containment prevents all but a very limited view of the supports of these large components. A more detailed screening would be done by calculations based on plant drawings. Although this would provide greater assurance, he felt that this was not an area of potential vulnerability and that additional analysis is probably not warranted. Discussion with Surry's seismic personnel revealed more information. The Surry vulnerabilities discussed in NUREG-4551 and cited in the peer review comment appear inconsistent with other analyses performed on Surry's RCP and Steam Generator supports. A Westinghouse analysis was performed using the WESTDYNE computer code (reviewed and found acceptable by the NRC for use on Surry Units). Elastic analysis using 90% of yield allowable showed that the RCP and steam generator supports had a factor of safety of at least 2.5 for the DBE. Based on this analysis, Surry is also screening NSSS components in their seismic PRA. In light of the above, BGE's position is that there is not enough information available to indicate that the screening of the CCNPP NSSS components is inappropriate. We feel that further analysis is not warranted.
Seismic	The documents reviewed in the course of this review provided no information on the treatment of passive structures in the screening process. The disposition of these items should be documented.	This comment was incorporated and is explained in the second paragraph of Section 3.1.2.
Seismic	The hazard curve used in the CCSPRA is based on the mean LLNL curve presented in NUREG-1488, and is CCNPP site specific. Exceedance frequencies within the 50 to 1000 cm/s² not explicitly provided are calculated by linear interpolation. In the reviewers judgment based on a plot of the mean hazard curve, logarithmic interpolation, rather than linear, would have been a more appropriate method for calculating the exceedance frequencies.	In hindsight, a logarithmic interpolation would have been more appropriate. The linear interpolation used tends to overestimate the hazard frequencies of the interpolated points. A sensitivity analysis showed that the linear interpolation may cause us to overestimate (more conservative) the seismic CDF by around 5% compared to results obtained using a logarithmic interpolation.
Seismic	The CCSPRA defined thirty seismic initiating events. These initiating events span seismic events from approximately 0.01 g to 6.12 g. The partitioning into thirty initiating events introduces a level of complexity into the model that the reviewers believe is not required. The thirty bins were selected with the intent of keeping the contribution of any given initiator to less than five percent of the total seismic core damage frequency. RISKMAN's fragility calculation option uses a piecewise integration scheme which effectively normalizes the calculated failure fraction to generate the best estimate for the failure fraction, regardless of the bin width. This feature combined with the fact that most of the seismic core damage frequency comes from the guaranteed failure bin, and from sequences involving the failure of a single fragility item (e.g., the	Some of the bins could probably have been combined without a significant increase in core damage frequency. This will be considered in future improvements to the Seismic model.

	surrogate Top Event ZA) lead to the observation that many fewer bins could be used to obtain the same degree of accuracy.	
Seismic	Top Event LE (OC EDG Sustains a Seismic Event) models the failure of item 13 from Table 3 of Ref. 3-2. Items 14 and 15 from Table 3 are also related to the diesel generator HVAC and have HCLPF values of less than O.3g. Why have the items been screened from the model?	These items were missed initially but are now included in the fragility of the 0C-EDG. Top event LE now includes items 13, 14 and 15. Refer to Table 3-3 and Section 3.1.5.2 of this report for a more detailed explanation of Top Event LE.
Seismic	Top Event LG includes the "U1 Turbine Lube Oil Cooler." This item has a HCLPF greater than 0.3g, and therefore is in theory, included in the surrogate Top Event LA and can be excluded from Top Event LG.	This item could have been excluded. It is not wrong to include it because its failure would still lead to SRW failure. It does not have a significant impact on the fragility for top event LG because it is only one of a composite of seven lower capacity component fragilities.
Seismic	The EXCEL file indicates that item 10 from Table 3 of the EQE report (Ref. 3-2) is included in Top Event LG. An e-mail received from John Koelbel on 6/12/97 indicated that item 10 is not included. The disposition of item 10 should be verified.	Item 10 is included and should be included in Top Event LG.
Seismic	A review of the split fraction assignment rules for the Seismic Event Tree shows that split fraction values for the seismic top events are set equal to 1.0 when the calculated split fraction value is greater than some cutoff value. This is certainly conservative and simplifies the event tree quantification, but this process should be documented. Most top events (e.g., LA, LE, LG, LJ) are guaranteed failed when the calculated split fraction is greater than 0.95, while others (e.g., LB, LH) are guaranteed failed when the split fraction is greater than 0.5.	This comment has been incorporated. The truncations are explained in Sections 3.1.5.3 and 3.1.5.5.
Seismic	SMC GTI Event Tree. According to the table from Ref. 6. Top Event SV should be guaranteed failed when Top Event LA is failed. It appears that the rules as written accomplish this, but one has to trace back through the rules for several other top events to verify that SV will indeed be failed when LA is failed. It would be much clearer, and easier for someone less familiar with the model to understand, if the split fraction SVF given LA=F was entered as the first SV split fraction, and split fraction SVI was left as the default. A similar comment applies to many top events in several event trees. The model currently modifies all rules for a top event, by adding the condition that the related seismic top event must be successful, rather than add a single guaranteed failed split fraction to the top of the list that is dependent on the failure of the related seismic top event. The latter method requires many fewer changes to the base general transient model, and is easier to review.	The intent of the rule changes in question were not intended to guarantee failure of the top event when LA failed, but to speed the model by forcing use of either the guaranteed success or guaranteed failure split fractions for sequences which were guaranteed to go to core damage (sequences where LA is failed). For example, before the change to top event SV rules, split fraction SV1 would be evaluated in both the failed and success states. After the rule change, when LA is failed, either SVS will be used, which has only one path (SV success), or SVF will be used, which also has only one path (SV failure). SV1 which has an intermediate failure probability and would normally be evaluated in both success and failed states, will not be used when LA is failed. The same type of rule change was made for several other top events.
Seismic	SMCGT2 Event Tree. It appears that Top Event PT should be guaranteed failed if Top Event LA is failed. But unless the analyst or reviewer is very familiar with the dependencies modeled for a number of previous top events, it is very difficult to determine whether PT will be guaranteed failed when Top Event LA is failed. This may not be strictly a question of style. A subtle change in the modeling assumptions related to the "previous" top events, may have a sneaky impact on the current rules for Top Event PT. It would be simpler and safer to add a rule at the beginning of the split fractions for Top Event PT that fails PT given LA=F.	Top Event PT is not meant to be guaranteed failed if top event LA is failed. A guaranteed failure split fraction, PTF, was added to speed the model. The explanation for Top Event SV, above, applies here as well.

Seismic	A mission time of 24 hours is assumed for the diesel generators following a seismically induced loss of offsite power. As evidenced during the hurricane event at Turkey Point when the plant was dependent on diesel generators for seven days or more, an external event induced loss of offsite power may last much longer than a "typical" loss of offsite power event. Part of the argument for using 24 hours in the internal events analysis (which has been way overused for a long time) is the high probability of recovery of the internal event initiator. While the 24 hour success criterion is typical of previous seismic PRAs, we believe it is difficult to defend especially for the high intensity seismic events. Something like 72 hours might be more appropriate for some of the larger seismic events. The use of 24 hours for the remaining success criteria other than diesel generators IS probably acceptable because you are only crediting such system success when there is no station blackout, and it appears that you have been careful not to inadvertently recover seismically damaged equipment	For internal events, the 24 hour mission time is used in part to account for the repairable nature of redundant equipment. In the seismic analyses, the same logic is applied. So even though a seismic induced loss of off-site power is likely to last more than 24 hours, we use a 24 hour mission time to account for the likely repairs that could be performed on diesels that failed in the 24 hour mission time. At CCNPP, we have five emergency diesel generators: two are self-cooled and three are cooled by Service Water. For long term AFW flow control indication, only one EDG is required. For EDG failures that occur after the 24 hr mission time, it is likely that one of the initial EDG failures would be recovered, and at any time, at least one EDG would be available. For example, one of the likely causes of losing the three SRW dependent EDG's is a loss of SRW cooling. EDG's 2A, 2B and IB fail at relatively low g levels due to failure of low-seismic-capacity SRW coolers in the Turbine Building. Recovery efforts would be to restore SRW. Turbine Building SRW could be isolated and the more rugged Auxiliary Building portion refilled and restarted to restore SRW to one of the SRW-dependent EDG's. Let us consider extending the mission time for the EDG's. This would require extending the mission times for EDG support systems such as SRW, SW, and 4KV Busses. AFW and other equipment should really be extended as well, for this option. The longer mission time tends to increase CDF. However, a longer mission time also allows more recovery options, which tends to decrease CDF. We currently do not model EDG recovery in either the internal events or external events models. Recovery scenarios would have to be modeled to avoid getting an overly-conservative increase in CDF. However, these are likely to be very complex. We believe that the current modeling yields realistic failure probabilities. In addition, the rest of the industry uses 24 hours also, and this allows for fairer comparisons. Therefore, we do not currently plan on extending the mission time beyond 24 hours for external events.

ATTACHMENT (2)

SEISMIC HAZARD AND SCREENING REPORT IN RESPONSE TO THE 50.54(F) INFORMATION REQUEST REGARDING FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: SEISMIC FOR R.E. GINNA NUCLEAR POWER PLANT

Constellation Energy Nuclear Group, LLC
March 31, 2014

Executive Summary

Purpose

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the Nuclear Regulatory Commission (NRC) issued a 50.54(f) letter (Reference 1) requesting information in response to NRC Near-Term Task Force (NTTF) recommendations intended to clarify and strengthen the regulatory framework for protection against natural phenomena. Reference 1 requests that licensees and holders of construction permits under Title 10 Code of Federal Regulations Part 50 (Reference 2) reevaluate the seismic hazards at their sites using updated seismic hazard information and present-day regulatory guidance and methodologies. This report provides the information requested in items (1) through (7) of the "Requested Information" in Enclosure 1 of Reference 1, pertaining to NTTF Recommendation 2.1 for R. E. Ginna Nuclear Power Plant, LLC (Ginna) in accordance with the documented intention of Constellation Energy Nuclear Group, LLC (CENG) transmitted to the NRC via letter dated April 26, 2013 (Reference 3).

Scope

In response to the 50.54(f) letter (Reference 1) and following the Screening, Prioritization, and Implementation Details (SPID) industry guidance document (Reference 4), a seismic hazard reevaluation for Ginna was performed to develop a Ground Motion Response Spectrum (GMRS) for comparison with the Safe Shutdown Earthquake (SSE). The new GMRS represents a beyond-design-basis alternative seismic demand developed by more modern techniques than were used for plant licensing. It does not constitute a change in the plant design or licensing basis.

Section 2 of this report provides a summary of the Ginna regional and local geology and seismicity, other major inputs to the seismic hazard reevaluation, and detailed seismic hazard results including definition of the GMRS. Seismic hazard analysis for Ginna, including site response evaluation and GMRS development (Sections 2.2, 2.3, and 2.4 of this report) was performed by Lettis Consultants International, Inc. (LCI) for Electric Power Research Institute (EPRI) (Reference 5). Section 3 describes the characteristics of the appropriate plant-level SSE for Ginna. Section 4 provides a comparison of the GMRS to the SSE. Sections 5 and 6 discuss interim actions and conclusions, respectively, for Ginna.

Conclusions

For Ginna, the SSE envelopes the GMRS in the frequency range of 1 to 10 Hz. Therefore per the SPID, Sections 3.2 and 7 (Reference 4), Ginna screens out of further seismic risk assessments in response to NTTF 2.1: Seismic, including seismic probabilistic risk assessment (SPRA) or seismic margin assessment (SMA), as well as spent fuel pool integrity evaluations. Additionally, Ginna screens out of the Expedited Seismic Evaluation Process (ESEP) interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 6).

Due to the GMRS exceeding the SSE in the frequency range above 10 Hz, high frequency confirmation will be performed in accordance with the SPID Sections 3.2 and 3.4 (Reference 4) based upon the schedule for central and eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013 (Reference 7). As mentioned in Section 3.4 (Reference 4), high frequency vibratory motions above about 10 Hz are not damaging to the large majority of nuclear plant structures, components, and equipment. However, those components determined to be potentially vulnerable to high frequency vibration, such as relays, contactors, and switches, will be evaluated as part of the high frequency confirmation in order to ensure such components maintain their functions important to safety.

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1 Introduction

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the Nuclear Regulatory Commission (NRC) established a Near Term Task Force (NTTF) to conduct a systematic review of NRC processes and regulations 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 requesting information to assure these recommendations would be addressed by all U.S. nuclear power plants (Reference 1). Reference 1 requests that licensees and holders of construction permits under Title 10 Code of Federal Regulations Part 50 (10CFR50) (Reference 2) reevaluate the seismic hazards at their sites using updated seismic hazard information and present-day regulatory guidance and methodologies. Depending on the outcome of the comparison between the reevaluated seismic hazard and the current site-specific design basis, performance of a seismic risk assessment may be necessary. Risk assessment approaches acceptable to the NRC staff include a seismic probabilistic risk assessment (SPRA), or a seismic margin assessment (SMA). Based upon the risk assessment results, 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" in Enclosure 1 of Reference 1, pertaining to NTTF Recommendation 2.1 for Ginna, located in Wayne County, New York, in accordance with the documented intention of Constellation Energy Nuclear Group, LLC (CENG) transmitted to the NRC via letter dated April 26, 2013 (Reference 3). In providing this information, Ginna followed the Screening, Prioritization, and Implementation Details (SPID) industry guidance document (Reference 4). The "Augmented Approach" guidance document (Reference 6) has been developed as the process for evaluating critical plant equipment as an interim action to demonstrate additional plant safety margin, prior to performing the complete plant seismic risk evaluations.

The original geological program involving a regional geological survey, borings, and other tests at the site was conducted to provide information needed to assess foundation conditions, seismic activity, and ground-water conditions. Additional studies were performed in 1973 as part of the Sterling alternative site evaluation. These results indicate that the rock and compact granular soil on the site provide a suitable foundation for plant structures (Reference 8, Section 2.5.1).

Structures, systems, equipment, and components related to plant safety are required to withstand the design-basis earthquake. These structures, systems, and components are placed in the applicable seismic category depending on their function. The original classifications of all components, systems, and structures of Ginna Station for the purpose of seismic design were Class I, Class II, or Class III as recommended in (Reference 8, Section 3.7.1.1.1)

1. TID 7024, Nuclear Reactors and Earthquakes, August 1963.
2. G. W. Housner, "Design of Nuclear Power Reactors Against Earthquakes," Proceedings of the Second World Conference on Earthquake Engineering, Volume I, Japan, 1960, pages 133, 134, and 137.

Class I

Those structures and components including instruments and controls whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity. Also, those structures and components vital to safe shutdown and isolation of the reactor. All components, systems, and structures classified as Class I were designed such that primary steady-state stresses when combined with the seismic stress resulting from the response to a ground acceleration of 0.20g acting in the vertical and horizontal planes simultaneously, are limited so that the function of the component, system, or structure shall not be impaired as to prevent a safe and orderly shutdown of the plant.

Class II

For Ginna Station, there were no Class II structures.

Class III

Those structures and components which are not related to reactor operation or containment. The structural design of all Class III structures met the requirements of the applicable building code which was the State Building Construction Code of the State of New York, 1961. This code did not reference the Uniform Building Code.

All systems and components designated Seismic Category I are designed so that there is no loss of function in the event of the safe shutdown earthquake. On May 22, 1992, Generic Letter 87-02, Supplement 1, transmitted Supplemental Safety Evaluation Report No. 2 (SSER No. 2) on the Seismic Qualification Utility Group (SQUG) Generic Implementation Procedure, Revision 2, dated February 14, 1992 (GIP-2). SSER No. 2 approved the methodology in the GIP-2 for use in verification of equipment seismic adequacy including equipment involved in future modifications and replacement equipment. In letters dated November 30, 1992, and June 8, 1993, the NRC accepted RG&E"s response to Generic Letter 87-02, Supplement 1. (Reference 8, Section 3.1.2.1.2)

Plant structures, systems, and components designated as Seismic Category I are designed to remain within applicable stress limits for the safe shutdown earthquake (0.20g) (Reference 8, Section 2.5.2.2).

In response to the 50.54(f) letter (Reference 1) and following the guidance provided in the SPID (Reference 4), a seismic hazard reevaluation for Ginna was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed.

Based on the results of the screening evaluation for Ginna, the SSE exceeds the GMRS in the frequency range of 1 to 10 Hz. Therefore per the SPID, Sections 3.2 and 7 (Reference 4), Ginna screens out of further seismic risk assessments in response to NTTF 2.1: Seismic, including seismic probabilistic risk assessment (SPRA) or seismic margin assessment (SMA), as well as spent fuel pool integrity evaluations. Additionally, Ginna screens out of the Expedited Seismic Evaluation Process (ESEP) interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 6).

Due to the GMRS exceeding the SSE in the frequency range above 10 Hz, high frequency confirmation will be performed in accordance with the SPID Sections 3.2 and 3.4 (Reference 4) based upon the schedule for central and eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013 (Reference 7).

2 Seismic Hazard Reevaluation

Ginna is situated on the south shore of Lake Ontario in Wayne County, New York, and is approximately 16 miles east of Rochester, NY (Reference 8, Section 1.2.1). A geological program involving a regional geological survey, borings, and other tests at the site was conducted to provide information needed to assess foundation conditions, seismic activity, and ground-water conditions. (Reference 8, Section 2.5.1)

The site is within 150 miles of the St. Lawrence valley area where earthquakes of Richter magnitude 7.0 have been experienced (Reference 8, Section 1.2.1). It is within 50 miles of the area around Buffalo which has experienced moderate earthquake activity of a smaller magnitude, and within 35 miles of the fault system near Attica. Historical and physical evidence indicates that the site is seismologically quiet (Reference 8, Section 2.5.2).

The northeastern United States and eastern Canada are moderately active earthquake areas. However, there is no instrumental or verifiable record of extremely large magnitude shocks (above Richter 8) and there is no record of damaging earthquakes with epicenters within 50 miles of the site. (Reference 8, Section 2.5.2.1). The maximum expected earthquakes would not result in significant ground motion at the site. Ground acceleration at the site is estimated to be less than 1% of gravity. It is judged that the maximum credible earthquake would be one of Richter magnitude 6.0 with an epicenter 60 miles from the site or one of magnitude 7.0 at a 90-mile epicentral distance (Reference 8, Section 2.5.2.2).

2.1 Regional and Local Geology

2.1.1 Regional Geology (Reference 8, S ection 2.5.1.1)

The site is located on the southern shore of Lake Ontario in the eastern portion of the Erie-Ontario Lowlands Physiographic Province (Fenneman, 1938). The regional topography is of low relief and rises gradually from an elevation of +250 mean sea level (msl) at the lake to +500 ft msl at the Portage Escarpment, which is the northern boundary of the Appalachian Plateau Province to the south. A beach ridge 10- to 25-ft high parallels the shoreline of Lake Ontario 4 miles to the south. North of the ridge is the lake plain of former glacial Lake Iroquois. The site lies on this plain.

The southern margin of Lake Ontario is characterized by many promontories which seem to reflect prominent joint directions in bedrock. The site is located near one such promontory called Smokey Point. Major joint directions are north 75° to 85° east and north 10° east to 30° west. Erosional bluffs along the lake range from 15- to 30-ft high. Smokey Point is located at the eastern end of a 5-mile-long ridge, the crest of which is about +310 ft. Relief in the site area is low, with elevations ranging from +350 to +300 ft. The site is underlain by 20 to 60 ft of glacial deposits and approximately 2700 ft of Paleozoic (570 million years to 225 million years before present) sedimentary rocks over crystalline basement. The uppermost Paleozoic unit is sandstone of Upper Ordovician (455 to 430 million years before present) Queenston formation. The Queenston is roughly 1000-ft thick in this area and overlays approximately 80 ft of Oswego sandstone, approximately 600 ft of Lorraine shales, and probably less than 30 ft of Potsdam sandstone. The pre-Cambrian surface is roughly 2600- to 2700-ft deep at the site.

The glacial deposits include at least two till horizons. The lower unit overlies bedrock and varies in thickness from 6 to 25 ft. This unit consists of grayish-red, calcareous, silty clay. The unit is poorly sorted and contains numerous striated and faceted pebbles, cobbles, and boulders. The upper till unit is at or near the ground surface and ranges from 7 to 30 ft in thickness. This unit is composed of relatively uniform olive-gray to yellow-brown silty, sandy clay, with large boulders several feet in diameter. Between the two till horizons is a zone of lakebed deposits consisting of gray, very plastic clay.

Rochester Gas and Electric Corporation (RG&E) determined by regional correlation that the lower till unit is associated with the Woodfordian glacial advance, a substage of the Wisconsinan Stage, which took place about 22,000 years ago. The lakebed deposit is believed to have been deposited in the bed of Lake Iroquois. The upper till is related to a minor glacial re-advancement that occurred about 12,000 years ago.

2.1.2 Local Geology (Reference 8, Section 2.5.1.2)

The major Ginna Station structures are supported in the Queenston formation or atop a thin layer of natural or compacted granular soils immediately above the bedrock. The Queenston formation, which is generally found at depths of 30 to 40 ft, is composed of alternating strata of thinly to thickly bedded, dense, fine-grained sandstone, silty and sandy siltstone, with occasional thin beds of fissile shale. Bedding is essentially horizontal with occasional crossbedding and shaly partings. The color is predominately red, but random green blotches and layers occur throughout the depths explored. Occasional continuous vertical joints were noted in the borings and during site inspections.

Prior to construction of the plant foundations, the soil overburden (30 to 40 ft of glacial drift) was removed. The exposed rock surface was observed to be similar to that examined in nearby outcrops. Bedding was horizontal and occasional cross-bedding and shaly partings were evident. A pattern of vertical joints of limited vertical extent was evident in the outcropping rock, particularly along the lakeshore side of the excavation. The observed joints continued to depths of from 20 to 30 ft from the top of the rock, but no evidence of movement along the joints was found. Some minor exfoliation noted in the bottom of the excavation is believed to have been caused primarily by the heavy-equipment traffic on the excavation floor and the drying effects of exposure to air.

The onshore shaft and tunnels were inspected during construction as well as after completion of the tunneling. Examination of the exposed rock revealed conditions consistent with those encountered during the previous studies. No zones of defective rock were found and no weathered rock was evident in the tunnels. The rock in both tunnels is sound. Water flow was practically nonexistent, being essentially limited to scattered areas of minor moisture infiltration. The actual conditions found in the tunnel excavations were in agreement with those encountered in all previous borings drilled during the initial subsurface investigation and the other supplementary investigations.

2.2 PROBABILISTIC SEISMIC HAZARD ANALYSIS

2.2.1 Probabilistic Seismic Hazard Analysis Results

In accordance with the 50.54(f) letter (Reference 1) and following the guidance in the SPID (Reference 4), 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 (Reference 9) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (Reference 10). For the PSHA, a lower-bound moment magnitude of 5.0 was used, as specified in Reference 1.

For the PSHA, the CEUS-SSC background seismic sources out to a distance of 400 miles (640 km) around Ginna were included. This distance exceeds the 200-mile (320 km) recommendation contained in NRC Reg. Guide 1.208 (Reference 11) 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 (Reference 9), the following sources lie within 1,000 km of the site and were included in the analysis:

1. Charlevoix
2. 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 (Reference 4), 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 2.3.7 at the SSE control point elevation.

2.3 SITE RESPONSE EVALUATION

Following the guidance contained in Enclosure 1 of the 50.54(f) letter (Reference 1) and the SPID (Reference 4) 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 Ginna.

2.3.1 Description of Subsurface Material

Ginna is located in the Erie-Ontario Physiographic Province of New York on the south shore of Lake Ontario. The general site conditions consist of about 40 ft (12.2m) of soil and till overlying Upper Ordovician Queenston Formation (siltstone, sandstone, and shale). Older Paleozoic sedimentary rock (Oswego, Lorraine, Trenton and Potsdam) extend from a depth of about 1000 ft (305 m) to Precambrian basement at a depth of about 2,700 ft (823m). (Reference 8, Section 2.5.1.1)

Ginna consists of a single unit, with the deepest structure foundations at elevation 231'8". This elevation was determined to be the SSE control point (See Section 3.2 of this Report), and the profile was modeled up to this elevation. Table 2.3.1-1 shows the geotechnical profile data for the site (Reference 12).

Table 2.3.1-1 Summary of Geotechnical Profile Data for Ginna

SOIL/ROCK DESCRIPTION	ELEV. [ft]	DEPTH [1] [ft]	Density γ [pcf]	Compressional Wave Velocity V_P [fps]	Shear Wave Velocity V_S [fps]	Poisson's Ratio ν
Ground Surface Elevation During Initial Site Borings	275'	0	---	---	---	---
Current Ground Surface Elevation	270'	5	---	---	---	---
Deepest Structure Foundation Elevation: 231'-8"	231'-8"	---	---	---	---	---
Soil	---	0-40	130 [6]	4400 [2]	2000 [5]	0.4 [7]
Queenston Siltstone, Sandstones and Shales	---	40-1040	158 [6]	12800 [2]	8200 [5][8]	0.15 [7]
Oswego Sandstone	---	1040-1120	165 [6]	14000 [3]	9000 [5]	0.15 [7]
Lorraine Siltstone and Shales	---	1120-1720	165 [7]	16000 [3]	9300 [5]	0.25 [7]
Trenton Limestone	---	1720-2670	160 [7]	16000 [7] 15000 [4]	9300 [5]	0.25 [7]
Potsdam Sandstone	---	2670-2700	165 [7]	16000 [7]	10250 [5]	0.15 [7]
Pre-Cambrian Basement	---	2700+	---	---	---	---

NOTES:
(1) Depth as measured from pre-construction, sub-surface investigations, with ground surface elevation of approximately 275' MSL
(2) Measured at site
(3) Measured at outcrop at Oswego, NY
(4) Measured at outcrop near Prospect, NY
(5) Calculated
(6) Measured
(7) Assumed
(8) Previously identified as 7200 fps. This value was determined to have been computed incorrectly during seismic hazard report preparation (Reference 13).

2.3.2 *Development of Base Case Profiles and Nonlinear Material Properties*

Table 2.3.1-1 shows the recommended shear-wave velocity and density versus depth for the best estimate profile (P1). Based on Table 2.3.1-1 and the specified location of the SSE, the base-case profile P1 consists of about 1000 ft (305m) of firm rock overlying Oswego and deeper formations with shear-wave velocity assumed to represent hard reference rock conditions.

Shear-wave velocities (Vs) in Table 2.3.1-1 are estimates based on refraction data using compression-wave data and estimates of Poisson's ratio. Upon review, it was determined that the Queenston layer should have an estimated Vs of 8200 fps (Reference 13) and that value of Vs was used for analysis of site response. For the deeper strata (Oswego, Lorraine, Trenton and Potsdam), estimates of shear-wave velocity are roughly equal to or greater than 9,300 ft /sec (2,830m/sec), the velocity of hard basement rock. All shear-wave velocity values were derived from compressional-wave measurements and an assumed Poisson ratio except for the Potsdam sandstone where the compressional wave velocity was also assumed (Table 2.3.1-1).

Based on the high shear-wave velocity estimates for the Queenstown Formation of 8,200 ft/s (2,499m/s) a factor of 1.25 was adopted to reflect upper-and lower-range base-cases. The scale factor of 1.25 reflects a $\sigma_{\mu ln}$ of about 0.2 based on the SPID (Reference 4) 10^{th} and 90^{th} 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-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-cases profiles P2 and P3 respectively. In this case the upper range base-case shear-wave velocity exceeds that of hard reference rock which was taken to represent profile P3. The base-case profile P1 was taken to have a mean depth below the SSE of 997 ft (304m) to hard reference rock, based on the high shear-wave velocity estimates (Table 2.3.1-1). Below 1,000 ft (305m) the velocity was set equal to the hard reference rock value of 9285 ft/sec (2,830m/sec) to a depth of 2,674 ft (815m) and randomized ± 802 ft (± 245m). The lower range base-case profile P2 was taken to extend to a depth below the SSE of 2,674 ft (815m) to hard reference rock, randomized ± 802 ft (± 245m). The base-case profiles (P1, P2, and P3) are shown in Figure 2.3.2-1 and listed in Table 2.3.2-1. The depth randomization reflects ± 30% of the depth and was included to provide a realistic broadening of the fundamental resonance rather than reflect actual random variations to basement shear-wave velocities across a footprint.

Figure 2.3.2-1 Shear-wave velocity profiles for the Ginna site

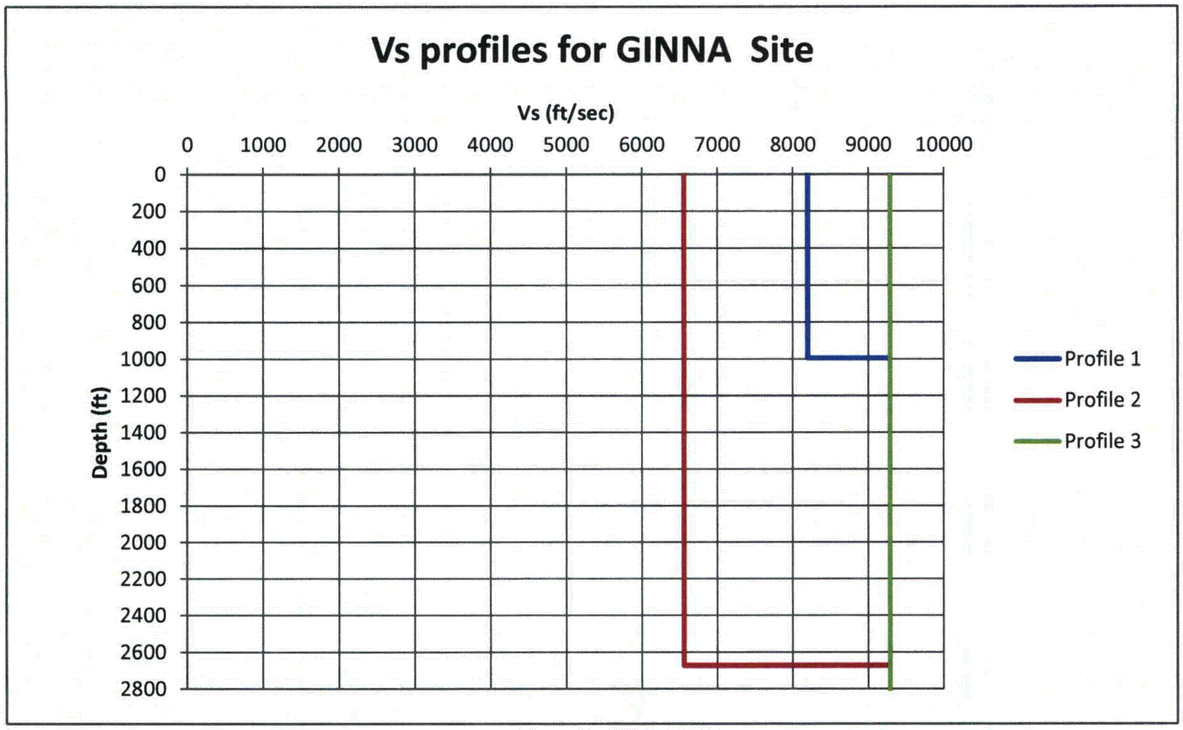


Table 2.3.2-1 Layer thicknesses, depths, and shear-wave velocities (V_s) for three profiles, Ginna site

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	8200		0	6560		0	9285
5.5	5.5	8200	5.5	5.5	6560	5.5	5.5	9285
8.5	14.0	8200	8.5	14.0	6560	8.5	14.0	9285
6.0	20.0	8200	6.0	20.0	6560	6.0	20.0	9285
9.0	29.0	8200	9.0	29.0	6560	9.0	29.0	9285
15.0	44.0	8200	15.0	44.0	6560	15.0	44.0	9285
6.0	50.0	8200	6.0	50.0	6560	6.0	50.0	9285
8.0	58.0	8200	8.0	58.0	6560	8.0	58.0	9285
14.0	72.0	8200	14.0	72.0	6560	14.0	72.0	9285
14.0	86.1	8200	14.0	86.1	6560	14.0	86.1	9285
14.0	100.1	8200	14.0	100.1	6560	14.0	100.1	9285
14.0	114.1	8200	14.0	114.1	6560	14.0	114.1	9285
6.0	120.1	8200	6.0	120.1	6560	6.0	120.1	9285
10.2	130.3	8200	10.2	130.3	6560	10.2	130.3	9285
16.2	146.6	8200	16.2	146.6	6560	16.2	146.6	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)
16.2	162.8	8200	16.2	162.8	6560	16.2	162.8	9285
16.2	179.0	8200	16.2	179.0	6560	16.2	179.0	9285
16.2	195.3	8200	16.2	195.3	6560	16.2	195.3	9285
16.2	211.5	8200	16.2	211.5	6560	16.2	211.5	9285
16.2	227.8	8200	16.2	227.8	6560	16.2	227.8	9285
16.2	244.0	8200	16.2	244.0	6560	16.2	244.0	9285
6.0	250.0	8200	6.0	250.0	6560	6.0	250.0	9285
10.7	260.7	8200	10.7	260.7	6560	10.7	260.7	9285
16.7	277.3	8200	16.7	277.3	6560	16.7	277.3	9285
16.7	294.0	8200	16.7	294.0	6560	16.7	294.0	9285
16.7	310.7	8200	16.7	310.7	6560	16.7	310.7	9285
16.7	327.3	8200	16.7	327.3	6560	16.7	327.3	9285
16.7	344.0	8200	16.7	344.0	6560	16.7	344.0	9285
16.7	360.7	8200	16.7	360.7	6560	16.7	360.7	9285
16.7	377.3	8200	16.7	377.3	6560	16.7	377.3	9285
16.7	394.0	8200	16.7	394.0	6560	16.7	394.0	9285
16.7	410.7	8200	16.7	410.7	6560	16.7	410.7	9285
16.7	427.3	8200	16.7	427.3	6560	16.7	427.3	9285
16.7	444.0	8200	16.7	444.0	6560	16.7	444.0	9285
16.7	460.7	8200	16.7	460.7	6560	16.7	460.7	9285
16.7	477.3	8200	16.7	477.3	6560	16.7	477.3	9285
16.7	494.0	8200	16.7	494.0	6560	16.7	494.0	9285
6.0	500.0	8200	6.0	500.0	6560	6.0	500.0	9285
44.3	544.3	8200	44.3	544.3	6560	44.3	544.3	9285
50.3	594.6	8200	50.3	594.6	6560	50.3	594.6	9285
50.3	644.9	8200	50.3	644.9	6560	50.3	644.9	9285
50.3	695.2	8200	50.3	695.2	6560	50.3	695.2	9285
50.3	745.5	8200	50.3	745.5	6560	50.3	745.5	9285
50.3	795.8	8200	50.3	795.8	6560	50.3	795.8	9285
50.3	846.1	8200	50.3	846.1	6560	50.3	846.1	9285
50.3	896.4	8200	50.3	896.4	6560	50.3	896.4	9285
50.3	946.7	8200	50.3	946.7	6560	50.3	946.7	9285
50.3	996.9	8200	50.3	996.9	6560	50.3	996.9	9285
167.1	1164.1	9285	167.1	1164.1	6560	167.1	1164.1	9285
167.1	1331.2	9285	167.1	1331.2	6560	167.1	1331.2	9285
167.1	1498.3	9285	167.1	1498.3	6560	167.1	1498.3	9285
167.1	1665.4	9285	167.1	1665.4	6560	167.1	1665.4	9285
167.1	1832.6	9285	167.1	1832.6	6560	167.1	1832.6	9285
167.1	1999.7	9285	167.1	1999.7	6560	167.1	1999.7	9285
167.1	2166.8	9285	167.1	2166.8	6560	167.1	2166.8	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)
167.1	2334.0	9285	167.1	2334.0	6560	167.1	2334.0	9285
167.1	2501.1	9285	167.1	2501.1	6560	167.1	2501.1	9285
173.1	2674.2	9285	173.1	2674.2	6560	173.1	2674.2	9285
3280.8	5955.0	9285	3280.8	5955.0	9285	3280.8	5955.0	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 Ginna. 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. (152 m) of firm rock at the Ginna site, two sets of shear modulus reduction and hysteretic damping curves were used. Consistent with the SPID (Reference 4), 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) were assumed to represent an equally plausible alternative rock response across loading levels. 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. (152 m).

2.3.2.2 Kappa

For the Ginna profile of either 997 ft. (304m) or 2,674 ft. (815m) of firm rock over hard reference rock, the kappa value of 0.006s for hard rock (Reference 4) was combined with the low strain damping in the hysteretic damping curves to give the values listed in Table 2.3.2-2. The low strain kappa values range from 0.006s for the stiffest profile (P3), taken as reference rock, to 0.019s for the softest and deepest profile (P2) combined with EPRI rock curves (Table 2.3.2-2). The mean base case profile P1 has a total kappa of 0.012s.

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

Velocity Profile	Kappa (s)	Weights
P1	0.012	0.4
P2	0.019	0.3
P3	0.006	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 Ginna 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 (Reference 4), 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 Reference 14 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. (15.2 m) and 0.15 below that depth. As specified in Reference 4, 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 (Reference 4), 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 1.5 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 Ginna site were the same as those identified in Tables B-4, B-5, B-6 and B-7 of Reference 4 as appropriate for typical CEUS sites.

2.3.5 Methodology

To perform the site response analyses for the Ginna 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 (Reference 4). The guidance contained in Appendix B of the Reference 4 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 Ginna site.

2.3.6 Amplification Functions

The results of the site response analysis consist of amplification factors (5% of critical damping 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 (Reference 4), 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 Reference 4 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 Ginna firm rock site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2). Between the nonlinear and linear (equivalent-linear) analyses, Figure 2.3.6-1 and Figure 2.3.6-2 respectively show only a minor difference across structural frequency as well as loading level. Tabulated values of the amplification factors are provided in Appendix A.

Figure 2.3.6-1

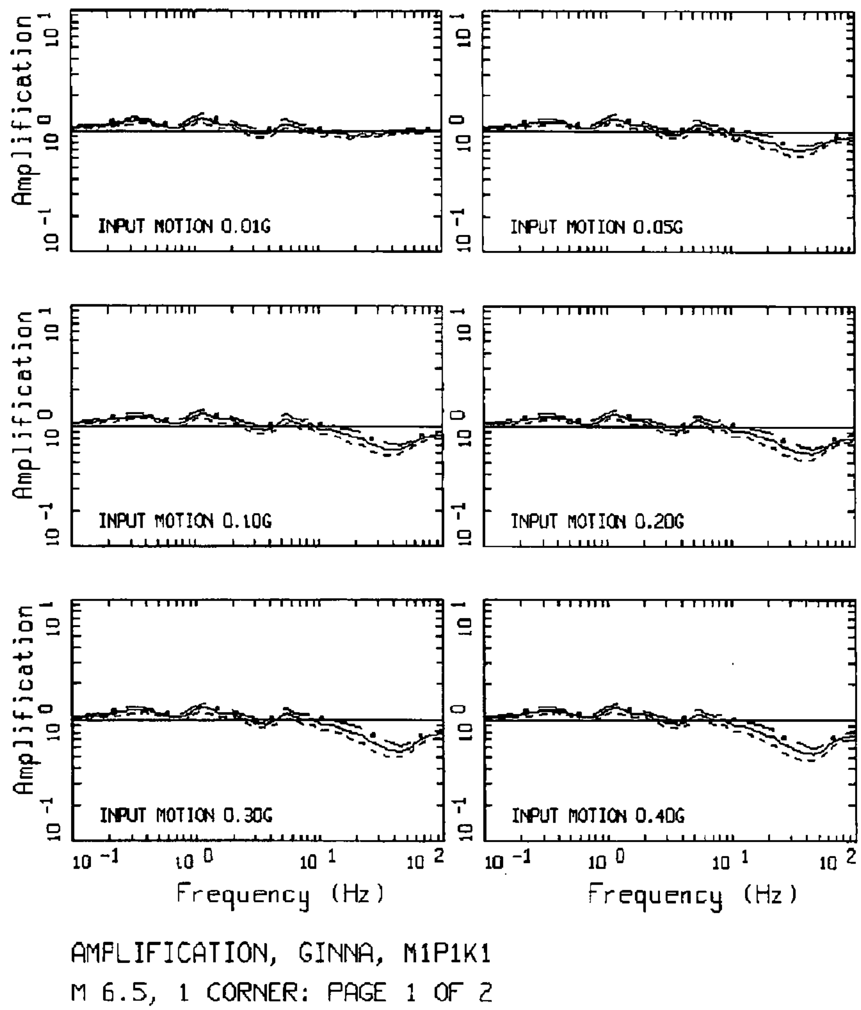


Figure 2.3.6-1 Example suite of amplification factors (5% of critical damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), EPRI rock modulus reduction and hysteretic damping curves (model M1), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g; **M** 6.5 and single-corner source model (Reference 4)

Figure 2.3.6-1 (cont'd)

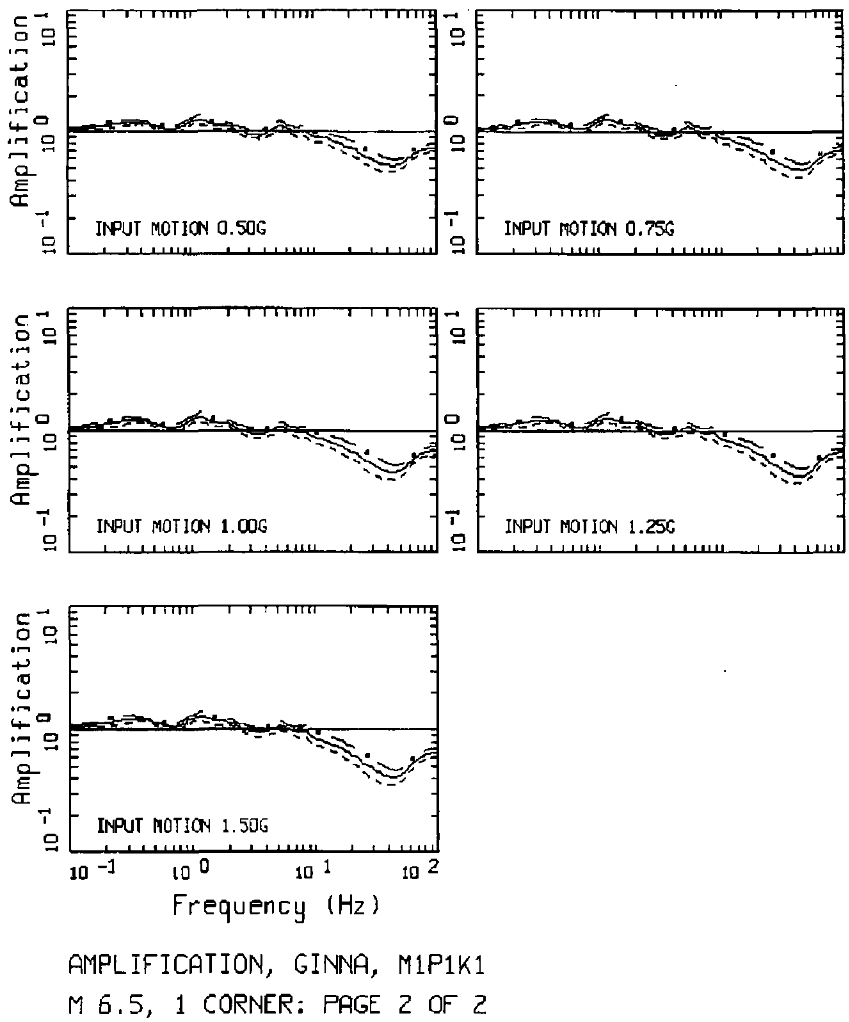


Figure 2.3.6-2

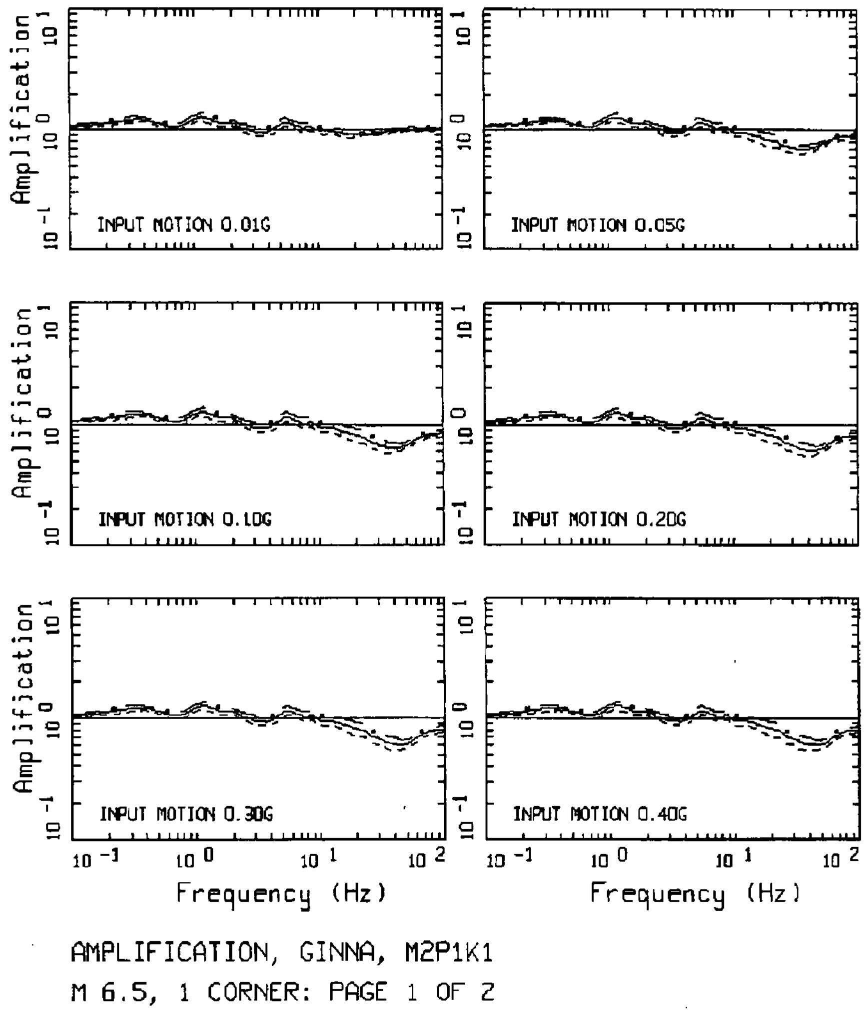
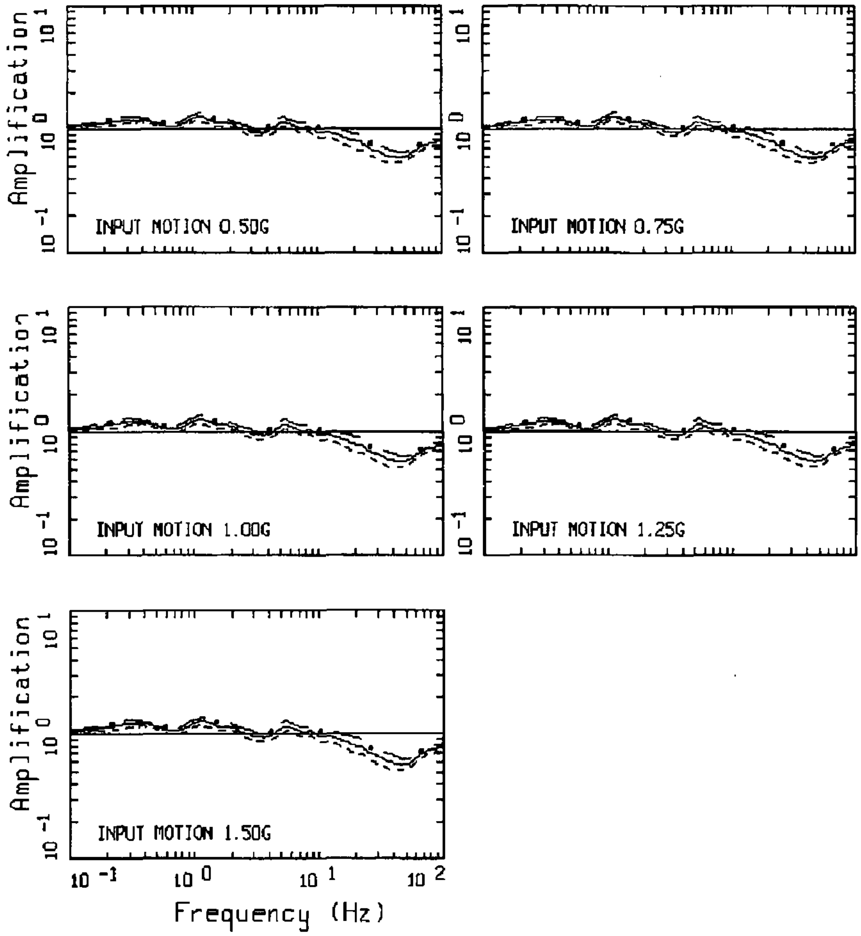


Figure 2.3.6-2 Example suite of amplification factors (5% of critical damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), linear site response (model M2), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g; M 6.5 and single-corner source model (Reference 4)

Figure 2.3.6-2 (cont'd)

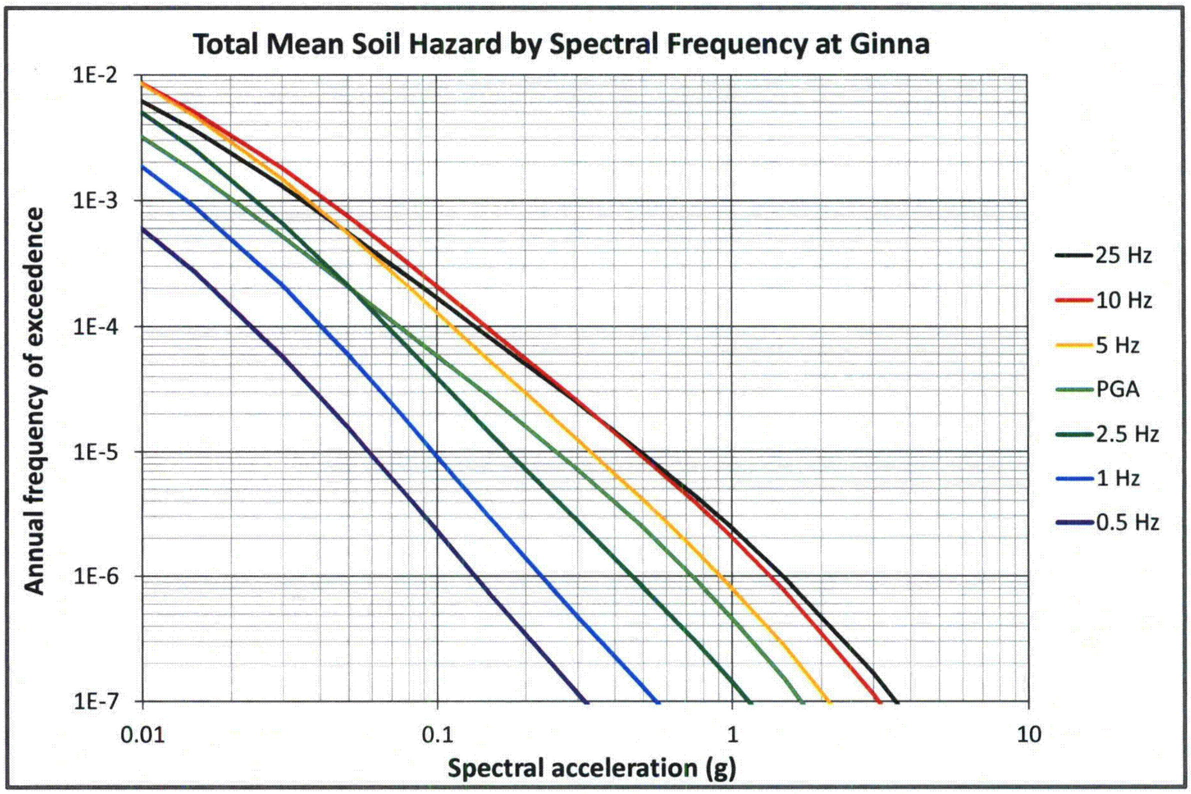


AMPLIFICATION, GINNA, M2P1K1
M 6.5, 1 CORNER: PAGE 2 OF 2

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 (Reference 4). 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 Ginna 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 and site response amplification functions 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 (PGA) at Ginna



2.4 GROUND MOTION RESPONSE SPECTRUM

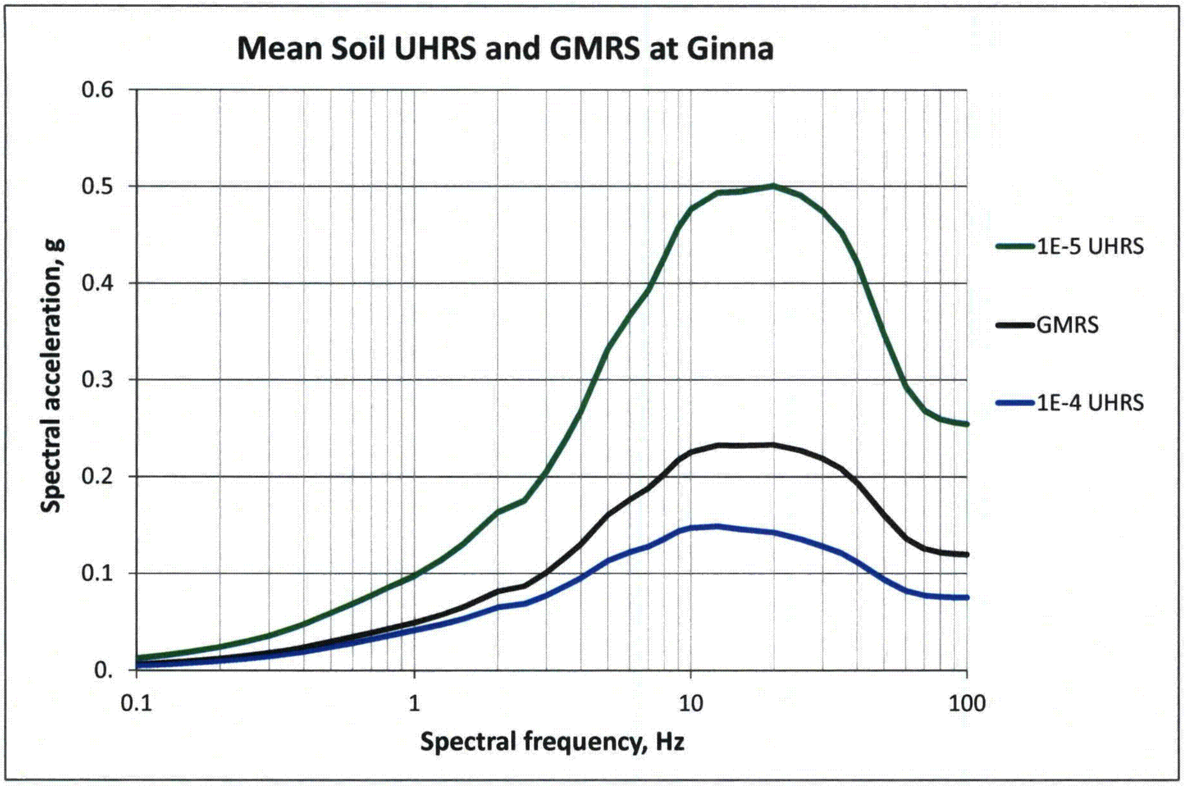
The control point hazard curves described above have been used to develop uniform hazard response spectra (UHRS) and the ground motion response spectrum (GMRS). The UHRS were obtained through linear interpolation in log-log space to estimate the spectral acceleration at each spectral frequency for the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

Table 2.4-1 UHRS and GMRS Ginna

Freq. (Hz)	10^{-4} UHRS (g)	10^{-5} UHRS (g)	GMRS (g)
100	7.46E-02	2.54E-01	1.19E-01
90	7.48E-02	2.56E-01	1.20E-01
80	7.53E-02	2.59E-01	1.21E-01
70	7.69E-02	2.68E-01	1.25E-01
60	8.17E-02	2.93E-01	1.36E-01
50	9.34E-02	3.48E-01	1.60E-01
40	1.11E-01	4.21E-01	1.93E-01
35	1.20E-01	4.52E-01	2.08E-01
30	1.28E-01	4.73E-01	2.19E-01
25	1.35E-01	4.90E-01	2.27E-01
20	1.42E-01	5.00E-01	2.33E-01
15	1.45E-01	4.94E-01	2.32E-01
12.5	1.48E-01	4.93E-01	2.33E-01
10	1.46E-01	4.76E-01	2.26E-01
9	1.43E-01	4.57E-01	2.17E-01
8	1.35E-01	4.25E-01	2.03E-01
7	1.27E-01	3.92E-01	1.88E-01
6	1.21E-01	3.66E-01	1.76E-01
5	1.13E-01	3.31E-01	1.60E-01
4	9.47E-02	2.67E-01	1.30E-01
3.5	8.62E-02	2.37E-01	1.16E-01
3	7.68E-02	2.04E-01	1.01E-01
2.5	6.80E-02	1.75E-01	8.68E-02
2	6.44E-02	1.63E-01	8.10E-02
1.5	5.24E-02	1.29E-01	6.48E-02
1.25	4.66E-02	1.13E-01	5.70E-02
1	4.07E-02	9.69E-02	4.89E-02
0.9	3.79E-02	9.10E-02	4.58E-02
0.8	3.50E-02	8.47E-02	4.26E-02
0.7	3.14E-02	7.69E-02	3.86E-02
0.6	2.75E-02	6.80E-02	3.40E-02
0.5	2.35E-02	5.87E-02	2.93E-02
0.4	1.88E-02	4.69E-02	2.34E-02
0.35	1.64E-02	4.11E-02	2.05E-02
0.3	1.41E-02	3.52E-02	1.76E-02
0.25	1.17E-02	2.93E-02	1.47E-02
0.2	9.39E-03	2.35E-02	1.17E-02
0.15	7.04E-03	1.76E-02	8.79E-03
0.125	5.87E-03	1.47E-02	7.33E-03
0.1	4.69E-03	1.17E-02	5.86E-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 the 1E-4 and 1E-5 uniform hazard spectra and GMRS at control point for Ginna (5%-damped response spectra)



3

Plant Design Basis [and Beyond Design Basis Evaluation Ground Motion]

The design basis for Ginna is identified in the Updated Final Safety Evaluation Report (UFSAR) Sections 2.5.2, 2.5.2.2 and 3.7.1.2 (Reference 8), and in the Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: "SEP Safety Topics III-6, Seismic Design Consideration and III-11, Component Integrity," dated January 29, 1982 (Reference 17)

3.1 SSE Description of Spectral Shape

Ginna was originally designed for an operating-basis earthquake characterized by a peak horizontal ground acceleration of 0.08g and for a safe shutdown earthquake with a peak horizontal ground motion of 0.2g. Peak horizontal and vertical accelerations were assumed to be the same. The response spectra used were those developed by Housner (Reference 8, Section 3.7.1.2).

The NRC conducted a seismic reevaluation of Ginna Station commencing in 1979 as part of the Systematic Evaluation Program (SEP). For the SEP reevaluation a safe shutdown earthquake with a peak horizontal ground motion of 0.2g was used. Two-thirds of that value was used for the vertical component. The response spectra used was that given in Regulatory Guide 1.60. It is noted that the site specific ground response spectra recommended by the NRC (Reference 15) for SEP evaluation of the seismic design adequacy of Ginna Station, indicates a peak horizontal ground motion acceleration of 0.17g, less than the 0.2g value used (Reference 8, Section 3.7.1.2).

The NRC initiated a generic program to develop criteria for the seismic qualification of equipment in operating plants as an Unresolved Safety Issue (USI A-46). Under this program, an explicit set of guidelines (or criteria) to be used to judge the adequacy of the seismic qualifications (both functional capability and structural integrity) of safety-related mechanical and electrical equipment at all operating plants was developed.

The NRC Staff as a result of the seismic review of the R. E. Ginna Nuclear Power Plant concluded that, since the ground response spectrum (0.2g Regulatory Guide 1.60 spectrum (Reference 16)) used for Ginna seismic reevaluation envelops the Ginna site-specific ground response spectrum, additional safety margins in the structures, systems, and components do exist for resisting seismic loadings. The staff also concluded that Ginna Station has an adequate seismic capacity to resist a postulated safe shutdown earthquake, and there is reasonable assurance that the operation of the facility will not endanger the health and safety of the public (Reference 17).

The SSE is defined in terms of the peak ground acceleration (PGA) and a design response spectrum. The design response spectrum shape is scaled to the PGA (anchor point) to develop the design response spectrum. The current design response spectrum shape for the Ginna SSE is a Reg. Guide 1.60 spectral shape (Reference 8, Section 3.7.1.2). This was upgraded from original Housner spectral shape during the SEP process. The peak horizontal ground acceleration for the SSE is noted to be 20% of gravity (0.20g), (Reference 8, Section 3.7.1.2). The design response spectrum shape is scaled in accordance with the PGA to produce a SSE ground response spectrum for the Ginna site.

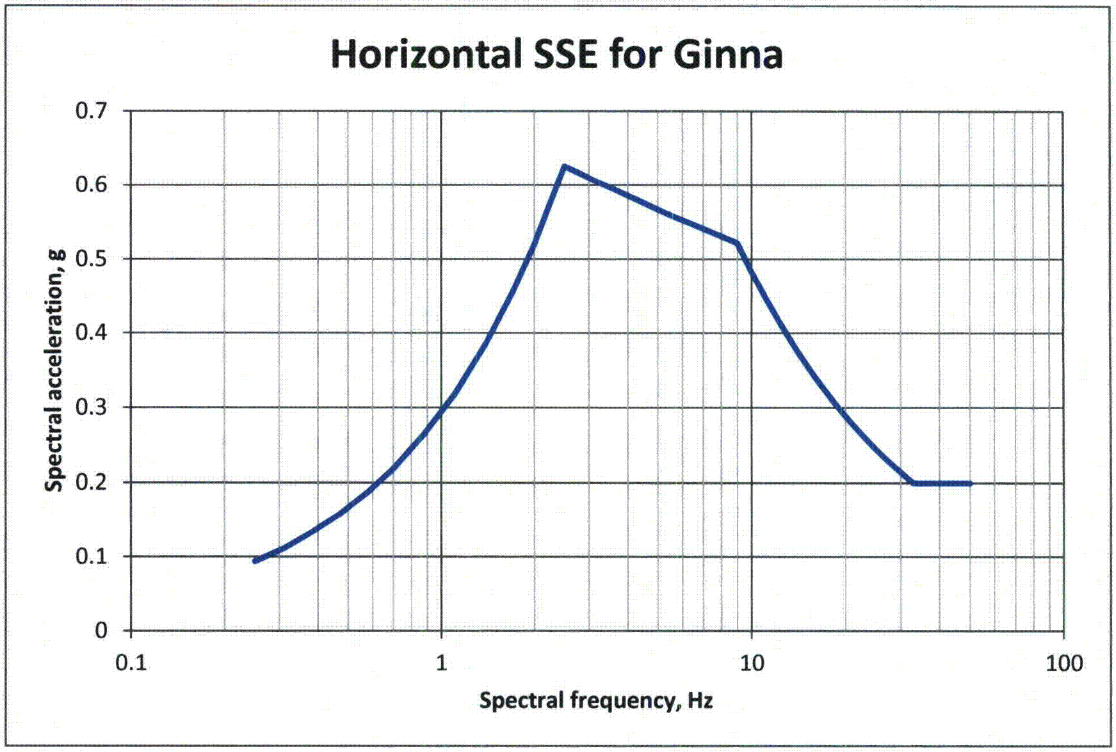
Table 3.1-1 shows the spectral acceleration values as a function of frequency for the 5% of critical damping horizontal SSE for Ginna (Reference 18). The 5% of critical damping horizontal SSE for Ginna is shown in Figure 3.1-1.

Table 3.1-1 Horizontal SSE for Ginna (5% of critical damping response spectrum)

Freq (Hz)	Spectral Acceleration (g)
50	0.2
33	0.2
29	0.22
26	0.238
23	0.261
20	0.289
18	0.313
16	0.341
14.5	0.367
13	0.398
11.5	0.436
10	0.483
9	0.522
8	0.531
7	0.541
6.4	0.548
5.6	0.558
5	0.567
4.5	0.576
4	0.586
3.5	0.597
3.2	0.604
2.8	0.616
2.5	0.626
2	0.521
1.7	0.456
1.4	0.388

Freq (Hz)	Spectral Acceleration (g)
1.1	0.318
.88	0.265
0.7	0.219
0.58	0.188
0.47	0.158
0.38	0.133
0.31	0.112
0.25	0.094

Figure 3.1-1 Horizontal SSE for Ginna (5% of critical damping response spectrum)



3.2 Control Point Elevation

As indicated in Reference 19, the Ginna site has multiple SSE inputs, and the containment building rests on rock at elevation 231'-8". This elevation was taken as the SSE Control Point, and the profile was modeled up to that elevation.

Ginna is a SEP plant, which has been subjected to a seismic re-evaluation in accordance with the NRC SEP process. Ginna does not have a formal control point defined in the UFSAR. Reference 4 notes that if the control point is not defined in the FSAR "For sites classified as a rock site or where the key safety-related structures are rock-founded then the control point is located at the top of the rock." Reference 4 also notes that deviations from the recommendations described above should also be documented. Ginna has quantified the existing seismic design basis, and will deviate from recommendations set forth in Reference 4 to ensure that the SSE-to-GMRS comparison as noted in Section 3 of Reference 4 is properly completed.

The Ginna powerblock complex is a compilation of structures with some founded in soil, and some founded in rock. The elevation of rock at the site varies from 235' to 240'. The base of the containment structure is founded on competent rock at 231'-8". The adjoining Intermediate Building and Auxiliary Building are founded on rock within two feet of the Containment (233'-6" and 233'-8" respectively). The Control Building and Diesel Generator Building are Seismic Category 1 (SC-1) structures which are founded on engineered backfill above the rock. The Intermediate Building basement floor slab (poured on rock at 233'-6") has the site's seismic instrument affixed to the slab.

For the SPID re-evaluation and SSE-to-GMRS comparison, the control point was selected to be 231-8", the elevation of the containment basemat and deepest Safety-Related powerblock structure. This location was chosen for the following reasons:

1. Ginna UFSAR (Reference 8) Section 3.7.1.3 states: "The input base excitation was a synthetic time-history acceleration record for which the corresponding response spectra were compatible with the 0.2g Regulatory Guide 1.60 spectra." This statement indicates that base input excitation, meeting the Regulatory Guide 1.60 spectra, was input directly to the base of the structure.

2. Ginna Reference 8 Section 3.7.2.4 states: "Soil-structure interaction was not considered in the design of Ginna Station. Sophisticated methods of treating soil-structure interaction exist; however, for structures that are founded on competent rock, as is Ginna Station, the effects of soil-structure interaction are considered relatively small. There is little radiation damping, and consideration of rock foundation compliance results in only slight increases in the periods of response of a structure when compared with the fixed-base case. It was expected that any variation in load that results from neglecting soil-structure interaction would be well within the accuracy of the calculations. This would be especially true for the containment structure, in which the walls are attached to the foundation rock by rock anchors. Therefore, soil-structure interaction was not taken into account in the seismic reevaluation".

3. The site's structural models, created to assist in the development of the Design Basis floor response spectra were reviewed. Structures founded on rock were modeled as rigidly connected to rock with the input motion (time history) scaled to a Regulatory Guide 1.60 spectra. The effects of soil backfill were not modeled in the analysis for these structures. For structures founded in soil, equivalent soil springs were developed to model the soil between the bottom of the structure and the top of the competent rock. The input motion was fed into the model at the competent rock with the soil-springs accounting for attenuation between the competent rock and the structure. Portions of structures excavated into competent (rigid) rock were excluded from the structural models (Containment Sump, Auxiliary Building Sub-Basement) as the rigid foundation precluded any spectral amplification for these portions of the structure. These portions of structures excavated into competent rock were discounted from determining the control point elevation.

4. The site's seismic instrumentation is affixed to a slab-on-rock with input motion fed from the competent rock at 233'-6". Reference 8 Section 3.7.4 notes "The response of the accelerograph located in the basement of the intermediate building will be virtually the same as one located in the basement of the containment. The elevations of the basement floors of both the containment and intermediate building are within 2 ft of one another and both basement mats are supported upon the underlying Queenston formation". The site seismic instrumentation acceptance criteria is a Regulatory Guide 1.60 spectra anchored at 0.20g's and 0.08g's for the site SSE and OBE respectively. This criteria is set forth in Ginna procedure TEG-2.1, "Safe Shutdown Earthquake (SSE) & Operating Basis Earthquake (OBE) Exceedance Determination," (Reference 20) and Ginna design basis calculation EWR5217 "Ground Response Spectra" (Reference 18). The seismic instrument, and supporting calculations have no adjustment for SSI effects or free-field considerations (not located on grade), and assume a rigid-base connection to the competent rock.

Given the reasons presented above, the GMRS to SSE comparison, as noted in Reference 4, is performed at the containment basemat to ensure an accurate comparison with existing site-specific design-basis seismic analysis and seismic event evaluation procedures.

4 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 SSE exceeds the GMRS. Therefore, a risk evaluation will not be performed.

4.2 High Frequency Screening (> 10Hz)

In the frequency range above 10 Hz, the GMRS exceeds the SSE slightly at spectral frequencies between approximately 30 to 38 Hz. Therefore, the plant screens in for a high frequency confirmation.

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

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 Interim Actions

Based on the screening evaluation outcome described in Section 4 of this report, existing Ginna seismic capacity is sufficient to demonstrate and ensure continued seismic safety relative to the reevaluated seismic hazard. Although a comprehensive seismic risk assessment (SPRA or SMA) is not required, high frequency confirmations will be performed in response to the section of the 50.54(f) letter (Reference 1) request for information pertaining to seismic hazard reevaluation in response to NTTF Recommendation 2.1.

5.1 Expedited Seismic Evaluation Process

The nuclear power industry has proposed, and CENG has committed to follow the "Augmented Approach" guidance document (Reference 6) to fulfill Enclosure 1: Seismic of Reference 1 request for information, regarding seismic aspects of NTTF Recommendation 2.1. The ESEP, contained within Reference 6, adds the additional short term aspect to the overall response to NTTF Recommendation 2.1. The ESEP addresses the part of Reference 1 that requests "interim evaluations and actions taken or planned to address the higher seismic hazard relative to the design basis, as appropriate, prior to completion of the risk evaluation." Specifically, the ESEP focuses initial industry efforts on short term evaluations that will lead to prompt modifications to some of the most important components that could improve plant seismic safety.

As described in Section 4 of this report, the SSE envelopes the GMRS between 1 and 10 Hz. Therefore, Ginna screens out of the ESEP based on Section 2.2 of Reference 6.

5.2 Interim Evaluation of Seismic Hazard

Consistent with NRC letter dated February 20, 2014, (Reference 21) the seismic hazard reevaluations presented herein are distinct from the current design and licensing bases of Plant. 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, Nuclear Energy Institute (NEI) letter dated March 12, 2014 (Reference 22) 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 (Reference 23):

> 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.

Ginna is included in the March 12, 2014 risk estimates (Reference 22). Using the methodology described in the NEI letter, all CEUS plants were shown to be below 10-4/year; thus, the above conclusions apply.

5.3 SEISMIC WALKDOWN INSIGHTS

Seismic walkdowns were performed at Ginna in accordance with the NRC-endorsed walkdown methodology (Reference 24). All potentially degraded, nonconforming, or unanalyzed conditions identified as a result of the seismic walkdowns were entered into the corrective action program to be addressed. The seismic walkdown of two items is deferred for completion in the spring 2014 Refueling Outage.

There was one issue identified during the seismic walkdowns that was judged to be a "Potentially Adverse Seismic Condition" and a prompt operability assessment was performed. However, following a detailed assessment, it was concluded that the issue would not prevent the equipment from performing its safety-related function during or after a seismic event.

None of the remaining issues identified during the seismic walkdowns and area walk-bys were determined to be "Potentially Adverse Seismic Conditions" in that the issues that were identified would not prevent the equipment from performing its safety-related function or the plant from achieving safe shutdown. There was one general concern identified for seismic housekeeping issues that were not in conformance with Station Procedures.

Seismic walkdown results indicate that the seismic capability of systems, structures and components is being maintained.

5.4 BEYOND DESIGN BASIS SEISMIC INSIGHTS

For the seismic IPEEE analysis (Reference 25), Ginna is categorized as a 0.3g focused-scope plant (per NUREG-1407; Reference 26). The licensing seismic design basis earthquake was based on Housner spectrum anchored to 0.2g PGA. Ginna listed all safety related structures, components, and performed the seismic evaluations based on the Screening Tables 2-3 and 2-4 of EPRI NP-6041, "A Methodology for Assessment of Nuclear Power Plant Seismic Margin," (Reference 27) for focused-scope plants and the 0.2g RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," (Reference 28) spectrum. The submittal addressed all the Generic Safety Issues (GSIs) as requested in NUREG-1407.

The screening of equipment seismic capacity for high-confidence-of-low-probability-of-failure (HCLPF) at Ginna is based on Table 2-4 of EPRI NP-6041 (Reference 27) for focused-scope plants. If a component did not satisfy the screening requirements, its seismic evaluation was then performed based on the upgraded design basis. The exceptions are certain anchorage evaluations where the anchored equipment is rigid. There the calculated factor of safety was reduced by 1.5 to meet the 0.3g screening requirement. To substantiate the component capacity evaluation Ginna provided lists of components that were evaluated either as rigid or in the frequency range over 10Hz, showing that these components have a factor of safety of 1.5 or greater and, therefore, meet the IPEEE screening criteria (Reference 25).

However, focused-scope IPEEE submittals may only be used for screening in the SPID after having been enhanced to bring the assessment in line with full scope assessments. These enhancements are defined in Section 3.3.1 of the SPID (Reference 4). Ginna is not demonstrating plant seismic adequacy based on screening from further review by comparing the GMRS to the IPEEE HCLPF spectrum.

Based on Ginna's IPEEE review, a number of seismic "vulnerabilities" were identified. Vulnerabilities associated with seismic/fire interactions were addressed in the IPEEE submittal (fire) as well as the IPEEE Seismic Evaluation Report, and indicated that all of these issues were resolved. In addition, the seismic margin analysis indicated that there were a number of equipment components that could not be screened out using the high confidence of low probability of failure (HCLPF) value for focused review level plants at 0.3g (Reference 25).

The Reactor Makeup Water Tank and the Monitor Tanks were considered seismic outliers and were examined to determine the correct course of action to reduce the core damage risks associated with a seismic event. Modifications to tanks and anchorages were made to address these seismic vulnerabilities (Reference 24).

Block wall outliers were resolved through a combination of structural modifications and operational procedure enhancements. Plant operations staff performed a series of walkdowns and field exercises to determine the viability of achieving equipment local operations in a timely fashion given the hazards and distractions block wall failures would present (Reference 29).

The evaluation determined that failures in the main steam header area of the Intermediate Building represented the most significant impediment to achieving event control. To offset the consequences associated with block wall failures in this area, a structural modification was performed. The objective of this modification was to minimize the possibility of blocks causing small leaks in small bore piping and instrument lines. With this modification in place, plant operators maintain the ability to perform their necessary duties (Reference 29).

Additionally, emergency response procedures were modified to warn operators of the block wall interaction possibilities and to provide guidance in mitigating the equipment mal-operations (Reference 29).

For safety-related equipment that could be affected by block wall interactions and is not credited for seismic safe shutdown at Ginna, no action was specifically taken to address these outliers (Reference 24).

6 Conclusions

In accordance with the 50.54(f) letter (Reference 1), a seismic hazard and screening evaluation was performed for Ginna. A GMRS was developed solely for purpose of screening for additional evaluations in accordance with the SPID (Reference 4).

Based on the results of the screening evaluation, the SSE envelopes the GMRS in the frequency range of 1 to 10 Hz, therefore Ginna is not required to perform a risk evaluation or spent fuel pool integrity evaluation. Additionally, Ginna screens out of the ESEP interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 6).

Based on the results of the screening evaluation, the GMRS exceeds the SSE in the frequency range above 10 Hz, high frequency confirmation will be performed in accordance with SPID, Section 3.4 (Reference 4). This evaluation will be completed based upon the schedule for CEUS nuclear plants provided in the April 9, 2013 letter from industry to the NRC (Reference 7).

7 References

1. Letter from E. J. Leeds (NRC) and M. R. Johnson (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *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 (ML12053A340)

2. Title 10 Code of Federal Regulations Part 50

3. Letter from M. G. Korsnick (CENG) to NRC Document Control Desk, *Response to Request for Information: Near-Term Task Force Recommendation 2.1, Seismic Reevaluation*, dated April 26, 2013 (ML13120A105)

4. EPRI 1025287, *Seismic Evaluation Guidance: Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, November 2012

5. Lettis Consultants International, Inc, *Ginna Seismic Hazard and Screening Report*, dated January 14, 2014 (Ginna Document 1041)

6. EPRI 3002000704, *Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, Palo Alto, CA, May 2013

7. Letter from A. R. Pietrangelo (NEI) to D. L. Skeen (NRC), *Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations,* dated April 9, 2013 (ML13101A379)

8. R.E. Ginna Nuclear Power Plant Updated Final Safety Analysis Report (UFSAR), Revision 24

9. EPRI 1021097 (NUREG-2115), *Central and Eastern United States Seismic Source Characterization for Nuclear Facilities*, Palo Alto, CA, January 2012

10. EPRI 3002000717, *EPRI (2004, 2006) Ground-Motion Model (GMM) Review Project*, Palo Alto, CA, June 2013

11. NRC Regulatory Guide 1.208, *A performance-based approach to define the site-specific earthquake ground motion*, 2007

12. Site Evaluation Study, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation, dated June 14, 1965

References (cont'd)

13. Ginna Condition Report CR-2013-006524, *Potential Error in Site Preliminary Geotechnical Report (Shear Wave Velocity)*

14. Silva, W.J., N. Abrahamson, G. Toro and C. Costantino, "*Description and validation of the stochastic ground motion model*," Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573, 1997

15. Letter from D. M. Crutchfield, NRC, to SEP Owners, Subject: *Site Specific Ground Response Spectra for SEP Plants Located in the Eastern U.S.*, dated June 17, 1981

16. NRC Regulatory Guide 1.60, *Design Response Spectra for Seismic Design of Nuclear Power Plants*, Revision 1

17. Letter from D. M. Crutchfield, NRC, to J. E. Maier, RG&E, Subject: *SEP Safety Topics III-6, Seismic Design Consideration and III-11, Component Integrity*, dated January 29, 1982

18. Engineering Work Request (EWR) 5217, *Design Analysis, Ground Response Spectra*, Revision 1

19. REPORTS-2013-0054, *R.E. Ginna Nuclear Power Plant Response to EPRI Request for Fukushima 2.1 Site Amplification Calculations*, Revision 000

20. TEG-2.1, *Safe Shutdown Earthquake (SSE) & Operating Basis Earthquake (OBE) Exceedance Determination*, Revision 00300

21. Letter from E. J. Leeds (NRC) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status on the Enclosed List, *Supplemental Information Related to Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* dated February 20, 2014 (ML14030A046)

22. Letter from A. R. Pietrangelo (NEI) to E. J. Leeds (NRC), *Seismic Risk Evaluations for Plants in the Central and Eastern United States*, dated March 12, 2014

23. Supplement 34 to NUREG-0933, *Resolution of Generic Safety Issues*, dated September 2011

24. Letter from M. G. Korsnick (CENG) to Document Control Desk (NRC*), R. E. Ginna Nuclear Power Plant, Response to 10 CFR 50.54(f) Request for Information, Recommendation 2.3, Seismic*, dated November 27, 2012 (ML12347A104)

25. Letter from G. S. Vissing (NRC) to Dr. R.C. Mecredy (RG&E), Subject: *Review of Ginna Individual Plant Examination of External Events (IPEEE) Submittal (TAC NO. M83624)*, dated December 21, 2000

References (cont'd)

26. NUREG-1407, *Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities*, dated June 1991

27. EPRI NP-6041-SL, *A Methodology for Assessment of Nuclear Power plant Seismic Margin*, Revision 1

28. Regulatory Guide 1.60, *Design Response Spectra for Seismic Design of Nuclear Power Plants*, dated December 1973

29. Letter from R. C. Mecredy (RG&E) to Document Control Desk (NRC), *Response to Request for Information on IP EEE*, dated July 30, 1999

A

Additional Tables

The following additional tables are included in Appendix A:

Table A-1a Mean and Fractile Seismic Hazard Curves for PGA (100Hz) at Ginna

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.05E-02	2.46E-02	3.95E-02	5.05E-02	6.26E-02	7.13E-02
0.001	3.62E-02	1.38E-02	2.60E-02	3.57E-02	4.77E-02	5.66E-02
0.005	8.15E-03	2.25E-03	4.25E-03	6.93E-03	1.10E-02	1.98E-02
0.01	3.20E-03	8.60E-04	1.36E-03	2.49E-03	4.19E-03	1.01E-02
0.015	1.71E-03	4.37E-04	6.45E-04	1.21E-03	2.19E-03	6.09E-03
0.03	5.19E-04	1.02E-04	1.55E-04	3.01E-04	6.73E-04	2.19E-03
0.05	2.07E-04	3.28E-05	5.27E-05	1.10E-04	2.80E-04	8.85E-04
0.075	9.91E-05	1.44E-05	2.42E-05	5.20E-05	1.40E-04	4.01E-04
0.1	5.87E-05	8.47E-06	1.44E-05	3.14E-05	8.35E-05	2.22E-04
0.15	2.78E-05	3.95E-06	7.03E-06	1.60E-05	4.01E-05	9.51E-05
0.3	7.24E-06	8.72E-07	1.77E-06	4.50E-06	1.10E-05	2.25E-05
0.5	2.45E-06	2.22E-07	5.05E-07	1.51E-06	3.95E-06	7.55E-06
0.75	9.52E-07	5.75E-08	1.53E-07	5.42E-07	1.60E-06	3.09E-06
1.	4.61E-07	1.84E-08	5.75E-08	2.42E-07	7.89E-07	1.60E-06
1.5	1.51E-07	2.96E-09	1.16E-08	6.54E-08	2.60E-07	5.75E-07
3.	1.67E-08	1.20E-10	4.56E-10	4.25E-09	2.68E-08	7.34E-08
5.	2.44E-09	5.05E-11	9.11E-11	4.07E-10	3.37E-09	1.15E-08
7.5	4.30E-10	3.47E-11	5.35E-11	9.79E-11	5.35E-10	2.07E-09
10.	1.12E-10	3.01E-11	4.01E-11	9.11E-11	1.62E-10	5.75E-10

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.54E-02	3.42E-02	4.56E-02	5.50E-02	6.64E-02	7.45E-02
0.001	4.31E-02	2.19E-02	3.33E-02	4.25E-02	5.42E-02	6.26E-02
0.005	1.33E-02	4.56E-03	7.66E-03	1.18E-02	1.82E-02	2.84E-02
0.01	6.18E-03	1.90E-03	3.09E-03	5.12E-03	8.35E-03	1.60E-02
0.015	3.65E-03	1.07E-03	1.64E-03	2.88E-03	4.98E-03	1.04E-02
0.03	1.29E-03	3.14E-04	4.90E-04	9.37E-04	1.79E-03	4.01E-03
0.05	5.55E-04	1.07E-04	1.74E-04	3.73E-04	8.00E-04	1.72E-03
0.075	2.76E-04	4.63E-05	7.77E-05	1.77E-04	4.13E-04	8.47E-04
0.1	1.68E-04	2.64E-05	4.56E-05	1.07E-04	2.53E-04	5.12E-04
0.15	8.28E-05	1.31E-05	2.25E-05	5.20E-05	1.27E-04	2.46E-04
0.3	2.46E-05	3.95E-06	7.13E-06	1.64E-05	3.79E-05	7.03E-05
0.5	9.65E-06	1.51E-06	2.84E-06	6.73E-06	1.53E-05	2.68E-05
0.75	4.39E-06	6.09E-07	1.23E-06	3.09E-06	7.23E-06	1.23E-05
1.	2.42E-06	3.01E-07	6.26E-07	1.67E-06	4.07E-06	6.93E-06
1.5	9.79E-07	9.65E-08	2.13E-07	6.36E-07	1.72E-06	2.96E-06
3.	1.67E-07	8.72E-09	2.29E-08	8.85E-08	3.05E-07	5.83E-07
5.	3.61E-08	1.04E-09	3.05E-09	1.46E-08	6.64E-08	1.38E-07
7.5	9.20E-09	1.90E-10	5.12E-10	2.80E-09	1.60E-08	3.84E-08
10.	3.19E-09	9.24E-11	1.69E-10	8.00E-10	5.20E-09	1.38E-08

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.14E-02	4.63E-02	5.20E-02	6.09E-02	7.13E-02	7.89E-02
0.001	5.13E-02	3.37E-02	4.13E-02	5.05E-02	6.17E-02	6.93E-02
0.005	1.83E-02	7.66E-03	1.16E-02	1.72E-02	2.49E-02	3.19E-02
0.01	8.57E-03	3.14E-03	4.77E-03	7.66E-03	1.20E-02	1.69E-02
0.015	5.05E-03	1.77E-03	2.60E-03	4.43E-03	7.13E-03	1.08E-02
0.03	1.78E-03	5.83E-04	8.47E-04	1.49E-03	2.49E-03	4.37E-03
0.05	7.47E-04	2.29E-04	3.33E-04	5.91E-04	1.04E-03	1.98E-03
0.075	3.56E-04	9.93E-05	1.49E-04	2.72E-04	5.05E-04	9.93E-04
0.1	2.07E-04	5.35E-05	8.23E-05	1.55E-04	2.96E-04	5.91E-04
0.15	9.54E-05	2.22E-05	3.57E-05	7.03E-05	1.40E-04	2.72E-04
0.3	2.50E-05	5.20E-06	8.98E-06	1.87E-05	3.84E-05	6.83E-05
0.5	9.07E-06	1.77E-06	3.23E-06	7.03E-06	1.42E-05	2.39E-05
0.75	3.87E-06	6.93E-07	1.32E-06	3.01E-06	6.17E-06	1.01E-05
1.	2.04E-06	3.33E-07	6.54E-07	1.55E-06	3.33E-06	5.42E-06
1.5	7.67E-07	1.05E-07	2.13E-07	5.66E-07	1.29E-06	2.16E-06
3.	1.13E-07	8.85E-09	2.13E-08	7.13E-08	1.95E-07	3.68E-07
5.	2.19E-08	9.65E-10	2.60E-09	1.11E-08	3.79E-08	8.12E-08
7.5	5.04E-09	1.72E-10	4.25E-10	2.01E-09	8.47E-09	2.10E-08
10.	1.62E-09	9.11E-11	1.42E-10	5.58E-10	2.64E-09	7.13E-09

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.28E-02	4.77E-02	5.27E-02	6.17E-02	7.34E-02	8.12E-02
0.001	5.35E-02	3.47E-02	4.19E-02	5.27E-02	6.54E-02	7.34E-02
0.005	1.91E-02	7.45E-03	1.18E-02	1.87E-02	2.68E-02	3.23E-02
0.01	8.48E-03	2.88E-03	4.77E-03	8.00E-03	1.23E-02	1.57E-02
0.015	4.73E-03	1.53E-03	2.53E-03	4.31E-03	7.03E-03	9.24E-03
0.03	1.46E-03	4.77E-04	7.34E-04	1.27E-03	2.22E-03	3.14E-03
0.05	5.45E-04	1.74E-04	2.57E-04	4.56E-04	8.12E-04	1.25E-03
0.075	2.36E-04	7.03E-05	1.05E-04	1.95E-04	3.52E-04	5.66E-04
0.1	1.29E-04	3.57E-05	5.50E-05	1.05E-04	1.90E-04	3.14E-04
0.15	5.41E-05	1.34E-05	2.19E-05	4.31E-05	8.12E-05	1.36E-04
0.3	1.24E-05	2.68E-06	4.63E-06	9.79E-06	1.95E-05	3.19E-05
0.5	4.09E-06	7.66E-07	1.42E-06	3.23E-06	6.54E-06	1.05E-05
0.75	1.62E-06	2.57E-07	5.05E-07	1.25E-06	2.64E-06	4.31E-06
1.	8.03E-07	1.10E-07	2.29E-07	6.00E-07	1.32E-06	2.22E-06
1.5	2.78E-07	2.88E-08	6.36E-08	1.92E-07	4.70E-07	8.35E-07
3.	3.53E-08	1.74E-09	4.70E-09	1.95E-08	6.09E-08	1.27E-07
5.	6.09E-09	2.01E-10	4.90E-10	2.49E-09	1.01E-08	2.49E-08
7.5	1.28E-09	9.11E-11	1.18E-10	4.25E-10	1.98E-09	5.66E-09
10.	3.88E-10	4.37E-11	9.11E-11	1.42E-10	5.83E-10	1.79E-09

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.89E-02	4.25E-02	4.77E-02	5.83E-02	7.03E-02	7.77E-02
0.001	4.66E-02	2.80E-02	3.42E-02	4.63E-02	5.91E-02	6.73E-02
0.005	1.29E-02	5.05E-03	7.66E-03	1.23E-02	1.87E-02	2.32E-02
0.01	5.03E-03	1.69E-03	2.76E-03	4.63E-03	7.55E-03	9.79E-03
0.015	2.57E-03	8.12E-04	1.32E-03	2.29E-03	3.90E-03	5.27E-03
0.03	6.61E-04	1.92E-04	3.05E-04	5.50E-04	1.04E-03	1.51E-03
0.05	2.10E-04	5.58E-05	8.85E-05	1.69E-04	3.33E-04	5.05E-04
0.075	7.90E-05	1.92E-05	3.09E-05	6.26E-05	1.25E-04	1.98E-04
0.1	3.89E-05	8.72E-06	1.44E-05	3.05E-05	6.17E-05	1.01E-04
0.15	1.44E-05	2.76E-06	4.90E-06	1.08E-05	2.35E-05	3.79E-05
0.3	2.75E-06	3.73E-07	7.55E-07	1.95E-06	4.63E-06	7.89E-06
0.5	8.21E-07	7.55E-08	1.74E-07	5.35E-07	1.40E-06	2.57E-06
0.75	3.03E-07	1.82E-08	4.77E-08	1.77E-07	5.35E-07	1.02E-06
1.	1.44E-07	6.00E-09	1.74E-08	7.66E-08	2.53E-07	5.12E-07
1.5	4.67E-08	1.13E-09	3.68E-09	2.04E-08	8.12E-08	1.84E-07
3.	5.30E-09	1.04E-10	2.19E-10	1.42E-09	8.47E-09	2.35E-08
5.	8.29E-10	5.05E-11	9.11E-11	1.84E-10	1.18E-09	3.95E-09
7.5	1.61E-10	3.23E-11	4.50E-11	9.11E-11	2.25E-10	8.00E-10
10.	4.57E-11	3.01E-11	4.01E-11	9.11E-11	1.01E-10	2.53E-10

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	4.02E-02	1.87E-02	2.68E-02	4.01E-02	5.27E-02	6.09E-02
0.001	2.59E-02	9.93E-03	1.60E-02	2.53E-02	3.57E-02	4.31E-02
0.005	5.12E-03	1.31E-03	2.53E-03	4.56E-03	7.77E-03	1.07E-02
0.01	1.86E-03	3.68E-04	7.45E-04	1.55E-03	3.01E-03	4.37E-03
0.015	9.09E-04	1.51E-04	3.14E-04	7.13E-04	1.51E-03	2.32E-03
0.03	2.11E-04	2.60E-05	5.58E-05	1.46E-04	3.63E-04	6.26E-04
0.05	6.00E-05	5.91E-06	1.31E-05	3.73E-05	1.02E-04	1.92E-04
0.075	2.03E-05	1.74E-06	3.79E-06	1.15E-05	3.47E-05	6.73E-05
0.1	9.17E-06	6.93E-07	1.55E-06	4.83E-06	1.55E-05	3.09E-05
0.15	2.98E-06	1.79E-07	4.37E-07	1.46E-06	4.98E-06	1.07E-05
0.3	4.73E-07	1.36E-08	4.37E-08	1.95E-07	7.66E-07	1.90E-06
0.5	1.30E-07	1.60E-09	6.83E-09	4.19E-08	2.01E-07	5.58E-07
0.75	4.57E-08	2.96E-10	1.38E-09	1.11E-08	6.73E-08	2.07E-07
1.	2.12E-08	1.23E-10	4.31E-10	4.01E-09	2.92E-08	9.93E-08
1.5	6.73E-09	9.11E-11	1.18E-10	8.72E-10	8.12E-09	3.14E-08
3.	7.53E-10	4.01E-11	7.34E-11	9.79E-11	6.54E-10	3.19E-09
5.	1.20E-10	3.01E-11	4.01E-11	9.11E-11	1.18E-10	4.77E-10
7.5	2.42E-11	3.01E-11	4.01E-11	9.11E-11	9.11E-11	1.25E-10
10.	7.11E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	1.97E-02	8.72E-03	1.34E-02	1.90E-02	2.60E-02	3.23E-02
0.001	1.10E-02	4.25E-03	6.93E-03	1.02E-02	1.53E-02	2.01E-02
0.005	1.82E-03	3.42E-04	7.34E-04	1.49E-03	2.92E-03	4.43E-03
0.01	5.99E-04	6.93E-05	1.69E-04	4.31E-04	1.04E-03	1.72E-03
0.015	2.74E-04	2.32E-05	6.00E-05	1.74E-04	4.90E-04	8.60E-04
0.03	5.75E-05	2.92E-06	7.89E-06	2.84E-05	1.04E-04	2.10E-04
0.05	1.55E-05	5.66E-07	1.51E-06	6.00E-06	2.72E-05	6.09E-05
0.075	5.10E-06	1.40E-07	3.90E-07	1.64E-06	8.47E-06	2.13E-05
0.1	2.28E-06	4.90E-08	1.44E-07	6.45E-07	3.57E-06	9.93E-06
0.15	7.36E-07	1.01E-08	3.42E-08	1.74E-07	1.05E-06	3.42E-06
0.3	1.15E-07	4.90E-10	2.25E-09	1.84E-08	1.34E-07	5.75E-07
0.5	3.09E-08	9.93E-11	2.80E-10	3.09E-09	2.92E-08	1.57E-07
0.75	1.08E-08	9.11E-11	9.79E-11	6.83E-10	8.23E-09	5.27E-08
1.	5.05E-09	5.50E-11	9.11E-11	2.39E-10	3.14E-09	2.32E-08
1.5	1.62E-09	4.01E-11	6.54E-11	9.11E-11	7.23E-10	6.64E-09
3.	1.90E-10	3.01E-11	4.01E-11	9.11E-11	9.93E-11	5.66E-10
5.	3.18E-11	3.01E-11	4.01E-11	9.11E-11	9.11E-11	1.20E-10
7.5	6.62E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11
10.	2.00E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11

Table A-2 Amplification Functions for Ginna

PGA (100 Hz)	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	9.89E-01	3.71E-02	1.30E-02	9.01E-01	4.54E-02	1.90E-02	9.56E-01	8.18E-02	2.09E-02	1.06E+00	7.82E-02
4.95E-02	8.79E-01	4.97E-02	1.02E-01	7.57E-01	8.28E-02	9.99E-02	9.35E-01	9.73E-02	8.24E-02	1.05E+00	7.98E-02
9.64E-02	8.42E-01	5.37E-02	2.13E-01	7.35E-01	9.08E-02	1.85E-01	9.30E-01	9.93E-02	1.44E-01	1.05E+00	7.96E-02
1.94E-01	8.13E-01	5.64E-02	4.43E-01	7.19E-01	9.51E-02	3.56E-01	9.23E-01	1.01E-01	2.65E-01	1.05E+00	7.92E-02
2.92E-01	7.98E-01	5.77E-02	6.76E-01	7.11E-01	9.70E-02	5.23E-01	9.18E-01	1.02E-01	3.84E-01	1.04E+00	7.90E-02
3.91E-01	7.88E-01	5.85E-02	9.09E-01	7.04E-01	9.84E-02	6.90E-01	9.14E-01	1.02E-01	5.02E-01	1.04E+00	7.89E-02
4.93E-01	7.81E-01	5.91E-02	1.15E+00	6.98E-01	9.94E-02	8.61E-01	9.10E-01	1.03E-01	6.22E-01	1.04E+00	7.89E-02
7.41E-01	7.68E-01	6.01E-02	1.73E+00	6.87E-01	1.01E-01	1.27E+00	9.03E-01	1.04E-01	9.13E-01	1.03E+00	7.89E-02
1.01E+00	7.59E-01	6.07E-02	2.36E+00	6.77E-01	1.03E-01	1.72E+00	8.96E-01	1.05E-01	1.22E+00	1.03E+00	7.88E-02
1.28E+00	7.51E-01	6.08E-02	3.01E+00	6.69E-01	1.04E-01	2.17E+00	8.89E-01	1.05E-01	1.54E+00	1.03E+00	7.87E-02
1.55E+00	7.45E-01	6.08E-02	3.63E+00	6.63E-01	1.04E-01	2.61E+00	8.83E-01	1.05E-01	1.85E+00	1.02E+00	7.86E-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	1.02E+00	7.76E-02	1.27E-02	1.16E+00	8.07E-02	8.25E-03	1.14E+00	7.97E-02
7.05E-02	1.01E+00	7.75E-02	3.43E-02	1.15E+00	7.87E-02	1.96E-02	1.14E+00	7.69E-02
1.18E-01	1.01E+00	7.72E-02	5.51E-02	1.15E+00	7.80E-02	3.02E-02	1.13E+00	7.60E-02
2.12E-01	1.01E+00	7.67E-02	9.63E-02	1.15E+00	7.74E-02	5.11E-02	1.13E+00	7.53E-02
3.04E-01	1.01E+00	7.65E-02	1.36E-01	1.15E+00	7.70E-02	7.10E-02	1.13E+00	7.51E-02
3.94E-01	1.01E+00	7.63E-02	1.75E-01	1.15E+00	7.68E-02	9.06E-02	1.13E+00	7.50E-02
4.86E-01	1.01E+00	7.61E-02	2.14E-01	1.15E+00	7.67E-02	1.10E-01	1.13E+00	7.49E-02
7.09E-01	1.01E+00	7.58E-02	3.10E-01	1.15E+00	7.65E-02	1.58E-01	1.13E+00	7.49E-02
9.47E-01	1.01E+00	7.56E-02	4.12E-01	1.15E+00	7.64E-02	2.09E-01	1.13E+00	7.50E-02
1.19E+00	1.01E+00	7.50E-02	5.18E-01	1.15E+00	7.63E-02	2.62E-01	1.13E+00	7.50E-02
1.43E+00	1.01E+00	7.48E-02	6.19E-01	1.15E+00	7.63E-02	3.12E-01	1.13E+00	7.51E-02

Tables A2-b1 and A2-b2 are tabular versions of the typical amplification factors provided in Figures 2.3.6-1 and 2.3.6-2. Values are provided for two input motion levels at approximately 10-4 and 10-5 mean annual frequency of exceedance. These factors are unverified and are provided for information only. The figures should be considered the governing information.

Table A2-b1 Median AFs and Sigmas for Model 1, Profile 1, for 2 PGA Levels

M1P1K1	Rock PGA=0.0964			M1P1K1	PGA=0.292		
Freq. (Hz)	Soil_SA	med. AF	sigma ln(AF)	Freq. (Hz)	Soil_SA	med. AF	sigma ln(AF)
100.0	0.082	0.847	0.060	100.0	0.227	0.777	0.069
87.1	0.082	0.837	0.060	87.1	0.230	0.764	0.070
75.9	0.084	0.820	0.061	75.9	0.234	0.740	0.072
66.1	0.086	0.786	0.063	66.1	0.242	0.696	0.074
57.5	0.090	0.728	0.067	57.5	0.258	0.627	0.081
50.1	0.099	0.675	0.073	50.1	0.288	0.579	0.093
43.7	0.110	0.643	0.090	43.7	0.328	0.557	0.111
38.0	0.122	0.640	0.118	38.0	0.366	0.568	0.140
33.1	0.134	0.654	0.130	33.1	0.402	0.593	0.152
28.8	0.144	0.692	0.142	28.8	0.430	0.637	0.163
25.1	0.153	0.720	0.124	25.1	0.452	0.668	0.139
21.9	0.163	0.793	0.132	21.9	0.478	0.747	0.140
19.1	0.169	0.818	0.132	19.1	0.489	0.779	0.138
16.6	0.173	0.859	0.121	16.6	0.494	0.824	0.126
14.5	0.175	0.899	0.105	14.5	0.493	0.865	0.112
12.6	0.180	0.940	0.091	12.6	0.501	0.906	0.097
11.0	0.177	0.940	0.093	11.0	0.491	0.915	0.100
9.5	0.180	0.990	0.106	9.5	0.493	0.964	0.113
8.3	0.178	1.051	0.091	8.3	0.483	1.028	0.093
7.2	0.164	1.027	0.084	7.2	0.442	1.007	0.085
6.3	0.165	1.090	0.070	6.3	0.438	1.067	0.068
5.5	0.166	1.142	0.087	5.5	0.439	1.121	0.084
4.8	0.153	1.070	0.098	4.8	0.405	1.060	0.097
4.2	0.138	0.987	0.080	4.2	0.363	0.983	0.080
3.6	0.128	0.938	0.075	3.6	0.336	0.935	0.076
3.2	0.123	0.951	0.086	3.2	0.320	0.948	0.085
2.8	0.122	0.990	0.079	2.8	0.316	0.988	0.079
2.4	0.118	1.030	0.067	2.4	0.303	1.029	0.067
2.1	0.115	1.101	0.060	2.1	0.294	1.099	0.059
1.8	0.106	1.133	0.060	1.8	0.270	1.132	0.059
1.6	0.093	1.141	0.075	1.6	0.235	1.141	0.073
1.4	0.084	1.189	0.071	1.4	0.211	1.188	0.071
1.2	0.079	1.256	0.091	1.2	0.195	1.253	0.090
1.0	0.070	1.241	0.065	1.0	0.174	1.238	0.064
0.91	0.060	1.156	0.067	0.91	0.147	1.155	0.066
0.79	0.051	1.078	0.060	0.79	0.124	1.078	0.059
0.69	0.045	1.051	0.058	0.69	0.107	1.051	0.058
0.60	0.040	1.072	0.060	0.60	0.095	1.071	0.059
0.52	0.036	1.117	0.051	0.52	0.084	1.116	0.051
0.46	0.031	1.164	0.037	0.46	0.073	1.162	0.037
0.10	0.001	1.069	0.020	0.10	0.003	1.064	0.020

Table A2-b2 Median AFs and Sigmas for Model 2, Profile 1, for 2 PGA Levels

M2P1K1	PGA=0.0964			M2P1K1	PGA=0.292		
Freq. (Hz)	Soil_SA	med. AF	sigma ln(AF)	Freq. (Hz)	Soil_SA	med. AF	sigma ln(AF)
100.0	0.082	0.852	0.055	100.0	0.236	0.807	0.061
87.1	0.083	0.842	0.055	87.1	0.239	0.794	0.061
75.9	0.084	0.825	0.056	75.9	0.244	0.771	0.062
66.1	0.086	0.792	0.056	66.1	0.253	0.728	0.062
57.5	0.091	0.734	0.058	57.5	0.272	0.662	0.065
50.1	0.100	0.681	0.062	50.1	0.308	0.619	0.072
43.7	0.112	0.650	0.081	43.7	0.354	0.601	0.095
38.0	0.123	0.647	0.106	38.0	0.395	0.613	0.122
33.1	0.136	0.661	0.111	33.1	0.432	0.638	0.124
28.8	0.145	0.699	0.117	28.8	0.459	0.680	0.128
25.1	0.155	0.728	0.109	25.1	0.482	0.713	0.117
21.9	0.165	0.801	0.128	21.9	0.506	0.791	0.134
19.1	0.170	0.826	0.130	19.1	0.514	0.819	0.135
16.6	0.174	0.868	0.123	16.6	0.518	0.863	0.127
14.5	0.176	0.908	0.105	14.5	0.516	0.904	0.108
12.6	0.181	0.949	0.088	12.6	0.523	0.946	0.090
11.0	0.178	0.946	0.084	11.0	0.506	0.943	0.085
9.5	0.181	0.995	0.097	9.5	0.507	0.992	0.098
8.3	0.179	1.057	0.091	8.3	0.496	1.055	0.091
7.2	0.165	1.032	0.088	7.2	0.452	1.030	0.089
6.3	0.166	1.096	0.077	6.3	0.449	1.094	0.078
5.5	0.167	1.147	0.086	5.5	0.448	1.145	0.086
4.8	0.154	1.073	0.098	4.8	0.410	1.072	0.098
4.2	0.138	0.989	0.081	4.2	0.365	0.989	0.081
3.6	0.128	0.939	0.074	3.6	0.337	0.938	0.074
3.2	0.123	0.951	0.085	3.2	0.321	0.950	0.085
2.8	0.122	0.991	0.080	2.8	0.316	0.989	0.080
2.4	0.118	1.031	0.067	2.4	0.303	1.029	0.067
2.1	0.115	1.101	0.062	2.1	0.294	1.099	0.062
1.8	0.107	1.133	0.061	1.8	0.270	1.131	0.060
1.6	0.093	1.142	0.076	1.6	0.235	1.139	0.075
1.4	0.084	1.189	0.071	1.4	0.210	1.186	0.070
1.2	0.079	1.256	0.091	1.2	0.195	1.252	0.090
1.0	0.070	1.241	0.065	1.0	0.174	1.237	0.064
0.91	0.060	1.156	0.068	0.91	0.147	1.154	0.067
0.79	0.051	1.078	0.060	0.79	0.124	1.077	0.059
0.69	0.045	1.052	0.058	0.69	0.107	1.051	0.057
0.60	0.040	1.072	0.060	0.60	0.095	1.071	0.059
0.52	0.036	1.118	0.051	0.52	0.084	1.116	0.050
0.46	0.031	1.164	0.037	0.46	0.073	1.162	0.036
0.10	0.001	1.069	0.020	0.10	0.003	1.064	0.020

ATTACHMENT (3)

SEISMIC HAZARD AND SCREENING REPORT IN RESPONSE TO THE 50.54(F) INFORMATION REQUEST REGARDING FUKUSHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: SEISMIC FOR NINE MILE POINT NUCLEAR STATION, UNITS 1 & 2

Constellation Energy Nuclear Group, LLC
March 31, 2014

Executive Summary

Purpose

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the Nuclear Regulatory Commission (NRC) issued a 50.54(f) letter (Reference 1) requesting information in response to NRC Near-Term Task Force (NTTF) recommendations intended to clarify and strengthen the regulatory framework for protection against natural phenomena. The 50.54(f) letter (Reference 1) requests that licensees and holders of construction permits, under Title 10 Code of Federal Regulations Part 50 (Reference 2), reevaluate the seismic hazards at their sites using updated seismic hazard information and present-day regulatory guidance and methodologies. This report provides the information requested in items (1) through (7) of the "Requested for Information" in Enclosure 1 of the 50.54(f) letter (Reference 1), pertaining to NTTF Recommendation 2.1 for Nine Mile Point Nuclear Station Units 1 (NMP1) and 2 (NMP2), in accordance with the documented intention of Constellation Energy Nuclear Group, LLC (CENG) transmitted to the NRC via letter dated April 26, 2013 (Reference 20).

Scope

In response to the 50.54(f) letter (Reference 1) and following the Screening, Prioritization and Implementation Details (SPID) industry guidance document (Reference 3), a seismic hazard reevaluation for NMP1 and NMP2 was performed to develop a Ground Motion Response Spectrum (GMRS) for comparison with the Safe Shutdown Earthquake (SSE) for each plant. The new GMRS represents a beyond-design-basis alternative seismic demand developed by more modern techniques than were used for plant licensing. It does not constitute a change in the plant design or licensing basis.

Section 2 provides a summary of the NMP regional and local geology and seismicity, other major inputs to the seismic hazard reevaluation and detailed seismic hazard results including definition of the GMRS. Seismic hazard analysis for Nine Mile Point, including site response evaluation and GMRS development (Sections 2.2, 2.3 and 2.4 of this report) was performed by the Lettis Consultants International (LCI) (Reference 27). A more in-depth discussion of the calculation methods used in the seismic hazard reevaluation is not included in this report but can be found in References 3, 7, 8, 9 and 10. Section 3 describes the characteristics of the appropriate plant-level SSE for both NMP1 and NMP2. Section 4 provides a comparison of the GMRS to the SSE for both NMP1 and NMP2. Sections 5 and 6 discuss interim actions and conclusions, respectively, for both NMP1 and NMP2.

CONCLUSIONS

For both NMP1 and NMP2, the SSE envelopes the GMRS in the frequency range of 1 to 10 Hz. Therefore per the SPID, Sections 3.2 and 7 (Reference 3), NMP1 and NMP2 screen out of further seismic risk assessments in response to NTTF 2.1: Seismic, including seismic probabilistic risk assessment (SPRA) or seismic margin assessment (SMA), as well as spent fuel pool integrity evaluations. Additionally, NMP1 and NMP2 screen out of the Expedited Seismic Evaluation Process (ESEP) interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 4).

Due to the GMRS exceeding both SSEs in the frequency range above 10 Hz, high frequency confirmations will be performed for both units in accordance with the SPID Sections 3.2 and 3.4 (Reference 3), based upon the schedule for central and eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013 (Reference 6). As mentioned in Section 3.4 (Reference 4), high frequency vibratory motions above 10 Hz are not damaging to the large majority of nuclear plant structures, components and equipment. However, those components determined to be potentially vulnerable to high frequency vibration, such as relays, contactors and switches, will be evaluated as part of the high frequency confirmation in order to ensure such components maintain their functions important to safety.

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1 Introduction

Following the accident at the Fukushima Daiichi nuclear power plant resulting from the March 11, 2011, Great Tohoku Earthquake and subsequent tsunami, the Nuclear Regulatory Commission (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 50.54(f) letter that requests information to assure these recommendations are addressed by all U.S. nuclear power plants (Reference 1). The 50.54(f) letter (Reference 1) requests that licensees and holders of construction permits under Title 10 Code of Federal Regulations Part 50 (10CFR50) (Reference 2) reevaluate the seismic hazards at their sites against present-day NRC requirements. Depending on the comparison between the reevaluated seismic hazard and the current site-specific 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 risk assessment results, 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" in Enclosure 1 of the 50.54(f) letter (Reference 1), pertaining to NTTF Recommendation 2.1 for Nine Mile Point Nuclear Station Units 1 (NMP1) and 2 (NMP2), located in Oswego County, New York, in accordance with the documented intention of Constellation Energy Nuclear Group, LLC (CENG) transmitted to the NRC via letter dated April 26, 2013 (Reference 20). In providing this information, Nine Mile Point Nuclear Station, LLC followed the Screening, Prioritization, and Implementation Details (SPID) industry guidance document (Reference 3). The "Augmented Approach" guidance document (Reference 4) has been developed as the process for evaluating critical plant equipment as an interim action to demonstrate additional plant safety margin, prior to performing the complete plant seismic risk evaluations.

NMP1 was licensed prior to Appendix A to 10 CFR Part 100, and the original geological and seismic siting investigations were conducted by Dames & Moore Engineering Geologists and Soil Mechanics Engineers. The site evaluation study explored the surface and subsurface geologic features of the site; analyzed the nature of flow of surface and ground water; developed seismological criteria for use in the design of structures to resist earthquake ground motion; and provided recommendations for site preparation, developed criteria for foundation design and discussed foundation installation. The summary of the report states that the power plant site is in a seismically quiet area within a moderately active region. There is no historical basis which would indicate that accelerations greater than about one percent of gravity would be experienced in the foundation rock at the plant site (Reference 29, Appendix C).

At NMP1, principal structures and equipment, which may serve either to prevent accidents or to mitigate their consequences, are designed, fabricated and erected to withstand the most severe earthquake, flooding condition, wind, ice and other natural phenomena expected to occur at the site, in accordance with Criterion 1 of the "General Design Criteria for Nuclear Power Plant Construction Permits" proposed by the U.S. Atomic Energy Commission in 1965 (Reference 11, Section I). The structural design of buildings and components was based on the maximum credible earthquake motion. Class I structures and components, whose failure could cause significant release of radioactivity, or which are vital to safe shutdown and isolation of the reactor, were designed so that the probability of failure would approach zero when subjected to the maximum credible earthquake motion (Reference 11, Section III).

For NMP2, the original geological and seismic siting investigations were performed in accordance with Appendix A to 10 CFR Part 100 and meet General Design Criterion 2 in Appendix A to 10CFR50 (Reference 2 and Reference 12, Section 3.1). Investigations of the origin and history of movement of both small displacement faults on the site proper and the large structural zone in close proximity to the site have been performed for NMP2 within the context of Appendix A to 10CFR100 (Reference 5, and Reference 12, Section 2.5) and is used for the design of seismic Category I systems structures and components.

In response to the 50.54(f) letter (Reference 1) and following the SPID guidance (Reference 3), a seismic hazard reevaluation for NMP1 and NMP2 was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed. Based on the results of the screening evaluation for both NMP1 and NMP2, the SSE exceeds the GMRS in the frequency range of 1 to 10 Hz. Therefore, per the SPID Sections 3.2 and 7 (Reference 3), NMP1 and NMP2 screen out of further seismic risk assessments in response to NTTF 2.1: Seismic, including seismic probabilistic risk assessment (SPRA) or seismic margin assessment (SMA), as well as spent fuel pool integrity evaluations. Additionally, NMP1 and NMP2 screen out of the ESEP interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 4).

Due to the GMRS exceeding both SSEs in the frequency range above 10 Hz, high frequency confirmations will be performed for both units in accordance with the SPID Sections 3.2 and 3.4 (Reference 3) based upon the schedule for central and eastern United States (CEUS) nuclear plants provided via letter from the industry to the NRC dated April 9, 2013 (Reference 6).

2 Seismic Hazard Reevaluation

NMP1 and NMP2 are situated on the southeast shore of Lake Ontario approximately six to seven miles northeast of the city of Oswego, New York (Reference 11, Section I and Reference 12, Section 1.1). The Nine Mile Point site is located within the Erie-Ontario Lowlands subdivision of the Central Lowlands physiographic province (Reference 12, Section 2.5.1.1.1). The area is situated within the northern part of the Appalachian Basin geologic province characterized by few deformation features despite the long history of site bedrock. Bedrock at the site consists of early Paleozoic marine sediment underlain by Precambrian crystalline rock (Reference 12, Section 2.5.1.1.1). All major structures are founded on sound rock (Reference 11, Section III and Reference 12, Section 3.7A.1.4).

The Nine Mile Point site is located in the Eastern Stable Platform tectonic province (Reference 12, Section 2.5.1.2.3). The site area is considered relatively tectonically stable and is free of major active tectonic structures. There are no known capable faults within 8 km of the site, and there is no potential for surface faulting within the site area (Reference 12, Section 2.5). The history of seismic activity in the northeastern United States and adjacent Canada is typical of intraplate tectonic regimes, showing only a few shocks that can be classified as major over the approximately two to three centuries of historical record. Only two of the recognized earthquake activity zones occur within 200 miles of the site, and the site locale itself may be characterized as virtually aseismic within a 90 mile radius (Reference 12, Section 2.5.2.1).

The investigation of historical seismic activity in the region for design and licensing of NMP1 indicated that a magnitude 7 (Intensity IX) earthquake 50 miles from the site was adequately conservative for the site maximum credible earthquake. A value of 11% of gravity (0.11g) was recommended to be used for ridged critical structures as the design parameter for the "maximum possible" condition in the foundation rock at the proposed power plant site (Reference 29, Appendix C).

As part of design and licensing for NMP2, the maximum earthquake potential was represented by a Modified Mercalli Intensity VI earthquake adjacent to NMP2, resulting in a peak horizontal ground motion of 0.07g. A very conservative value of 15% of gravity (0.15g) was adopted (Reference 12, Section 2.5.2.4).

2.1 Regional and Local Geology

The Nine Mile Point site is located in the Erie-Ontario Lowlands subdivision of the Central Lowlands physiographic province. The Erie-Ontario Lowlands extend southward from the site about 35 miles to the Portage Escarpment which forms the boundary between the lowlands and the Appalachian Uplands Province, and westward into Canada near Niagara Falls. The generally flat to gently undulating topography of the Erie-Ontario Lowlands is superimposed upon an erosional bedrock surface of irregular, low relief. A veneer of glacial deposits, such as tills, glaciofluvial sediment and proglacial lake sediments covers most of the area. (Reference 12, Section 2.5.1.1.1) The region is characterized by rocks at the surface which, although very old, have not been subjected to large-scale, orogenic processes. In consequence, few major structural features are known within 100 miles of the Nine Mile Point site (Reference 12, Section 2.5.2.2.1).

The Nine Mile Point site is located approximately six to seven miles northeast of Oswego, NY and is bounded on the north by Lake Ontario (Reference 12, Section 2.5.1.2.1). Overall, the site morphology reflects a bedrock surface modified by repeated Pleistocene glaciations that eroded weathered rock and deposited glacially-derived sediments. However, the site does not display any of the prominent drumlins that are characteristic of the Erie-Ontario Lowlands (Reference 12, Section 2.5.1.2.1). There are several zones of bedrock deformation that intersect the site excavations. Several Quaternary, low-angle thrust faults also intersect the main site excavations. These faults were judged by a panel of experts to have a negligible impact on the site engineering structures (Reference 12, Section 2.5.1.2.7).

2.2 PROBABILISTIC SEISMIC HAZARD ANALYSIS

2.2.1 Probabilistic Seismic Hazard Analysis Results

In accordance with the 50.54(f) letter (Reference 1) and following the guidance in the SPID (Reference 3), 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 (Reference 7) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (Reference 8). For the PSHA, a minimum moment magnitude of 5.0 was used, as specified in the 50.54(f) letter.

For the PSHA, the CEUS-SSC background seismic sources out to a distance of 400 miles (640 km) around Nine Mile Point were included. This distance exceeds the 200 mile (320 km) recommendation contained in NRC Reg. Guide 1.208 (Reference 10) 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 (Reference 7), the following sources lie within 1,000 km of the site and were included in the analysis:

1. Charlevoix
2. 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 (Reference 3), 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 2.3.7 at the SSE control point elevation.

2.3 SITE RESPONSE EVALUATION

Following the guidance contained in Enclosure 1 of the 50.54(f) letter Request for Information (Reference 1) and the SPID (Reference 3) for nuclear power plant sites that are not founded on hard rock (defined as having a shear wave velocity of at least 9285 fps or 9200 fps, as approximated in the SPID (Reference 3), a site response analysis was performed for Nine Mile Point.

2.3.1 Description of Subsurface Material

NMP1 and NMP2 are founded on firm rock of the Oswego formation. There are about 1,745 ft. of firm Ordovician sedimentary rocks which overlie Precambrian Basement. 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 Summary of geotechnical profile data for Nine Mile Point (Reference 21)

Soil/Rock Descriptions	El. (1) [ft.]	Depth (2) [ft.]	γ (3) [pcf]	V_P (4) [fps]	V_S (4) [fps]	ν (4)
Till/Fill (5)	261	0	140	1871	1000	0.30
Oswego (6), (8)	245	16	164	13638	6000	0.38
Oswego Transition (6)	225	36	164	14309	6500	0.37
Pulaski A (6)	210	51	168	14252	8000	0.27
Pulaski B (6)	170	91	168	14252	8000	0.27
Pulaski C (6)	153	108	168	14967	8000	0.30
Whetstone Gulf (7)	113	148	167	15911	7000	0.38
	-287	548	167	17048	7500	0.38
Trenton Group (7)	-700	961	170	16389	8600	0.31
Precambrian Grenville (7)	-1500	1761	177	16643	9200	0.28

NOTES:
(1) Corresponds to site grade NMP2
(2) Measured from plant grade
(3) Obtained based on NMP1 Unit Weight recommendations
(4) Determined from Cross-hole measurements near NMP2 or from NMP3 (deeper strata)
(5) Assumed (typical fill)
(6) Parameters determined from Cross-hole measurements near NMP2
(7) Parameters determined from NMP3 site investigation
(8) SSE control point elevation at top of layer, El. 245 ft.

The following description of the Paleozoic sequence is taken directly from the Data Request for Site Amplification Calculations (Reference 21).

In general, the soils at the sites can be divided into the following stratigraphic units:

- **Sand, Marls, Peats** of about 3 ft. thick and not present throughout all site.
- **Lake Iroquois Deposits**, which are deep water sediments up to 4 ft. thick, which directly overlie gray till, bedrock or ice marginal lake till where they occur in the site area. These sediments consist of laminated to massive, reddish brown or gray clayey silt or silty fine sand with lenses and laminations of fine to medium sand and a little gravel.

- **Glacial till**, gray or brown, up to approximately 15 ft. thick which directly overlies either bedrock, or in places, a 1 in. layer of gray sand.

- **Oswego Sandstone**, which consists of unfossiliferous, greenish-gray, medium- to fine-grained, massive sandstone and is rather uniform and monotonously similar. Thin black shale and siltstone beds are minor, although clay galls (shale intraclasts) are common. The sandstone is characteristically composed of subangular and subrounded quartz grains, sometimes with well-rounded rock fragments and a small amount of feldspar and clay matrix. The thickness of the Oswego Sandstone varies from 35 ft. near the lakeshore to nearly 120 ft. in the southern portion of the site. At the center of containment, the Oswego Sandstone presents a thickness of about 35 ft.

- **Oswego Transition Sandstone**, that corresponds to the lowermost 15 ft. of the Oswego Sandstone, where the bedrock consists of alternating, laminated to thick-bedded, fine- to medium-grained sandstone, argillaceous sandstone, dark siltstone and shale.

- **Pulaski Formation Unit A**, which is medium to thick-bedded and consists of mottled, dark gray argillaceous sandstone (referred to as graywacke) interbedded with light gray sandstone and few beds of dark gray shale and siltstone. The stratigraphic thickness of Unit A is approximately 40 ft.

- **Pulaski Formation Unit B**, which consists of interbedded light gray sandstone, black siltstone and shale with few beds of dark gray mottled graywacke. The bedding is regular and massive and commonly attains thicknesses of several feet. The thickness of Unit B is 15 to 20 ft. The base of the foundation of Unit 2 rests on the Pulaski Formation Unit B.

- **Pulaski Formation Unit C**, which consists of dark gray to black siltstone and shale interbedded with light gray sandstone. The thickness of Unit C is about 40 ft.

- **Whetstone Gulf Formation**, which extends to the greatest explored depth at the site. The gross composition of the formation is quite similar to Unit C of the Pulaski Formation. It consists of a well-bedded sequence of dark gray shale, siltstone and light gray medium-grained sandstone. The sandstone content can be used to divide the formation into two units. The top of the formation, Unit A, has few sandstone intercalations and is approximately 30 ft. thick. The rest of the explored section is categorized as Unit B. According to the NMP3 site investigation, the thickness of the Whetstone Gulf Formation is approximately 700 ft.

The previous stratigraphic units were also identified in the NMP3 Combined Operating License Application (COLA) site investigation. The Oswego Sandstone extends throughout the site area and most of the site vicinity.

Deeper geologic units are identified in the NMP3 FSAR and were introduced to estimate the depth at which the shear wave velocity has a value of 9200 fps. According to Section 2.5.1.1.3 of the NMP3 FSAR (Reference 24), the closest deep boring, which extends through the Paleozoic sedimentary sequence and advances to Precambrian gneissic basement, is approximately 7 miles southwest of the site in Oswego County and it penetrated the Trenton Group.

- **Trenton Group / Black River Group**, which is an Ordovician formation of gently dipping sandstone, siltstone and shale. Its thickness is about 700 to 800 ft. The NMP3 FSAR (Reference 24) provides a contour map of the top elevation of the Trenton formation (reproduced in Reference 23 as Figure 2-1). The presence and position of the Trenton Group is applicable to the NMP1 and NMP2 sites.

- **Precambrian Grenville Crystalline Basement Rock** (V_S > 9200 fps, treated as bedrock).

2.3.2 Development of Base Case Profiles and Nonlinear Material Properties

Table 2.3.1-1 shows the recommended shear-wave velocities and unit weights along with elevations and corresponding stratigraphy. From Reference 21, the SSE control point is at elevation 245 ft. at the top of the Oswego Sandstone with a measured shear-wave velocity of 6,000 fps (see Section 3.2 for further control point discussion). Cross-hole shear-wave velocity measurements extend to about 325 ft. below the SSE Control Point (Reference 21). Deeper values are from the NMP3 site investigation (Table 2.3.1-1). Based on the measurements and site investigations, shear-wave velocities with geology specific attributes were provided to a depth of about 1,745 ft. (elevation -1,500 ft., Table 2.3.1-1). From Table 2.3.1-1, with the SSE at an elevation of 245 ft., the depth below the SSE to Precambrian Basement is about 1,745 ft. (532 m).

Based on the site cross-hole measurements and the proximity to the nearby COLA measurements, a scale factor of 1.25 for developing upper and lower base-cases was judged to be a more appropriate reflection of epistemic uncertainty at this site. The scale factor of 1.25 reflects a $\sigma_{\mu ln}$ of about 0.2 based on the SPID (Reference 3) 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-profiles were developed using the scale factor of 1.25 (Reference 27). 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 respectively. Profiles P1, P2 and P3, mean, lower, and upper range base-cases respectively, extended to shallow hard rock conditions at a depth (below the SSE control point) of 1,745 ft. (532 m), randomized ± 524 ft. (± 160 m). The base-case profiles (P1, P2, and P3) are shown in Figure 2.3.2 1 (Reference 27) and listed in Table 2.3.2 1 (Reference 27). The depth randomization reflects ± 30% of the depth and was included to provide a realistic broadening of the fundamental resonance at deep sites rather than reflect actual random variations to basement shear-wave velocities across a footprint.

Figure 2.3.2-1 Shear-Wave Velocity Profiles for the Nine Mile Point Site

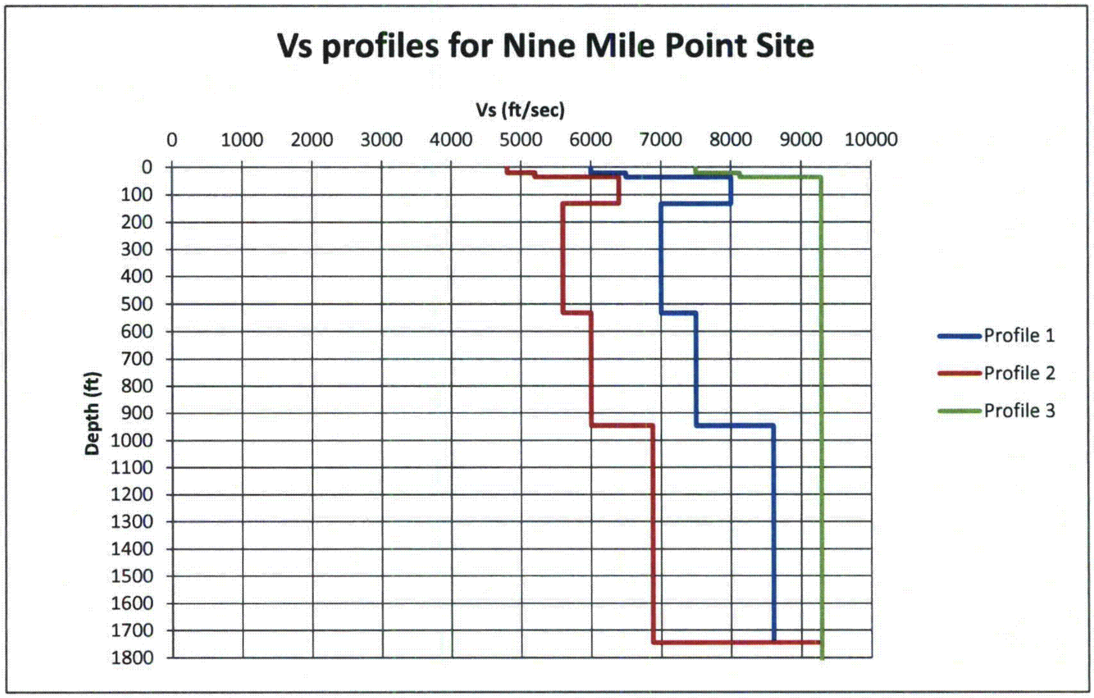


Table 2.3.2-1 Geologic Profile And Estimated Layer Thicknesses For Nine Mile Point Site

Profile 1			Profile 2			Profile 3		
Thickness (ft.)	Depth (ft.)	V_s (fps)	Thickness (ft.)	Depth (ft.)	V_s (fps)	Thickness (ft.)	Depth (ft.)	V_s (fps)
	0	6000		0	4800		0	7500
10.0	10.0	6000	10.0	10.0	4800	10.0	10.0	7500
10.0	20.0	6000	10.0	20.0	4800	10.0	20.0	7500
15.0	35.0	6500	15.0	35.0	5200	15.0	35.0	8125
16.2	51.2	8000	16.2	51.2	6400	16.2	51.2	9285
16.2	67.4	8000	16.2	67.4	6400	16.2	67.4	9285
16.2	83.6	8000	16.2	83.6	6400	16.2	83.6	9285
16.2	99.8	8000	16.2	99.8	6400	16.2	99.8	9285
16.2	116.0	8000	16.2	116.0	6400	16.2	116.0	9285
16.2	132.3	8000	16.2	132.3	6400	16.2	132.3	9285
9.0	141.3	7000	9.0	141.2	5600	9.0	141.2	9285
20.0	161.2	7000	20.0	161.2	5600	20.0	161.2	9285
20.0	181.2	7000	20.0	181.2	5600	20.0	181.2	9285
20.0	201.2	7000	20.0	201.2	5600	20.0	201.2	9285
10.0	211.2	7000	10.0	211.2	5600	10.0	211.2	9285
20.0	231.2	7000	20.0	231.2	5600	20.0	231.2	9285
20.0	251.2	7000	20.0	251.2	5600	20.0	251.2	9285
20.0	271.2	7000	20.0	271.2	5600	20.0	271.2	9285
20.0	291.2	7000	20.0	291.2	5600	20.0	291.2	9285
20.0	311.2	7000	20.0	311.2	5600	20.0	311.2	9285

Profile 1			Profile 2			Profile 3		
Thickness (ft.)	Depth (ft.)	V_s (fps)	Thickness (ft.)	Depth (ft.)	V_s (fps)	Thickness (ft.)	Depth (ft.)	V_s (fps)
20.0	331.2	7000	20.0	331.2	5600	20.0	331.2	9285
20.0	351.2	7000	20.0	351.2	5600	20.0	351.2	9285
20.0	371.2	7000	20.0	371.2	5600	20.0	371.2	9285
20.0	391.2	7000	20.0	391.2	5600	20.0	391.2	9285
20.0	411.2	7000	20.0	411.2	5600	20.0	411.2	9285
20.0	431.2	7000	20.0	431.2	5600	20.0	431.2	9285
20.0	451.2	7000	20.0	451.2	5600	20.0	451.2	9285
20.0	471.2	7000	20.0	471.2	5600	20.0	471.2	9285
20.0	491.2	7000	20.0	491.2	5600	20.0	491.2	9285
8.7	500.0	7000	8.7	500.0	5600	8.7	500.0	9285
32.3	532.2	7000	32.3	532.2	5600	32.3	532.2	9285
9.0	541.2	7500	9.0	541.2	6000	9.0	541.2	9285
10.0	551.2	7500	10.0	551.2	6000	10.0	551.2	9285
10.0	561.2	7500	10.0	561.2	6000	10.0	561.2	9285
10.0	571.2	7500	10.0	571.2	6000	10.0	571.2	9285
10.0	581.2	7500	10.0	581.2	6000	10.0	581.2	9285
36.4	617.6	7500	36.4	617.6	6000	36.4	617.6	9285
36.4	654.0	7500	36.4	654.0	6000	36.4	654.0	9285
36.4	690.4	7500	36.4	690.4	6000	36.4	690.4	9285
36.4	726.8	7500	36.4	726.8	6000	36.4	726.8	9285
36.4	763.2	7500	36.4	763.2	6000	36.4	763.2	9285
36.4	799.6	7500	36.4	799.6	6000	36.4	799.6	9285
36.4	836.0	7500	36.4	836.0	6000	36.4	836.0	9285
36.4	872.4	7500	36.4	872.4	6000	36.4	872.4	9285
36.4	908.8	7500	36.4	908.8	6000	36.4	908.8	9285
36.4	945.2	7500	36.4	945.2	6000	36.4	945.2	9285
80.0	1025.2	8600	80.0	1025.2	6880	80.0	1025.2	9285
80.0	1105.2	8600	80.0	1105.2	6880	80.0	1105.2	9285
80.0	1185.2	8600	80.0	1185.2	6880	80.0	1185.2	9285
80.0	1265.2	8600	80.0	1265.2	6880	80.0	1265.2	9285
80.0	1345.2	8600	80.0	1345.2	6880	80.0	1345.2	9285
80.0	1425.2	8600	80.0	1425.2	6880	80.0	1425.2	9285
80.0	1505.2	8600	80.0	1505.2	6880	80.0	1505.2	9285
80.0	1585.2	8600	80.0	1585.2	6880	80.0	1585.2	9285
80.0	1665.2	8600	80.0	1665.2	6880	80.0	1665.2	9285
79.6	1744.7	8600	79.6	1744.8	6880	79.6	1744.8	9285
3280.8	5025.6	9285	3280.8	5025.6	9285	3280.8	5025.6	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 Nine Mile Point. 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 Nine Mile Point site, two sets of shear modulus reduction and hysteretic damping curves were used. Consistent with the SPID (Reference 3), 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) were assumed to represent an equally plausible alternative rock response across loading levels. 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 (Reference 3) 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 0.006s for the underlying hard rock. For the Nine Mile Point site, with about 1,745 ft. (532 m) of firm rock, the kappa estimates were 0.014s, 0.016s and 0.006s for profiles P1, P2 and P3. The range of kappa from 0.006s to 0.016s does not reflect a reasonable assessment of epistemic uncertainty. To augment the uncertainty in kappa to a more realistic range, a 25% increase to the base-case kappa estimates was added for profiles P2, the softest profile. The base-case kappa estimate of 0.016s was augmented with increase in kappa of 0.004s to a value of 0.020s. The suite of kappa estimates and associated weights are listed in Table 2.3.2-2 (Reference 27).

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

Velocity Profile	Kappa (s)	Weights
P1	0.014	0.4
P2	0.020	0.3
P3	0.006	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 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 Nine Mile Point site, random shear wave velocity profiles were developed from the base case profiles shown in Figure **2.3.2-1** (Reference 27). Consistent with the discussion in Appendix B of the SPID (Reference 3), 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) Reference 9 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 (Reference 3), 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 (Reference 3), 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 (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 Nine Mile Point site, were the same as those identified in Tables B-4, B-5, B-6 and B-7 of the SPID (Reference 3) as appropriate for typical CEUS sites.

2.3.5 Methodology

To perform the site response analyses for the Nine Mile Point 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 the existing NRC guidance and the SPID (Reference 3). The guidance contained in Appendix B of the SPID (Reference 3) 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 Nine Mile Point 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 (Reference 3), 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 the SPID (Reference 3) 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 Nine Mile Point firm rock site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2). Tabulated values of the amplification factors are provided in Appendix A.

Figure 2.3.6-1

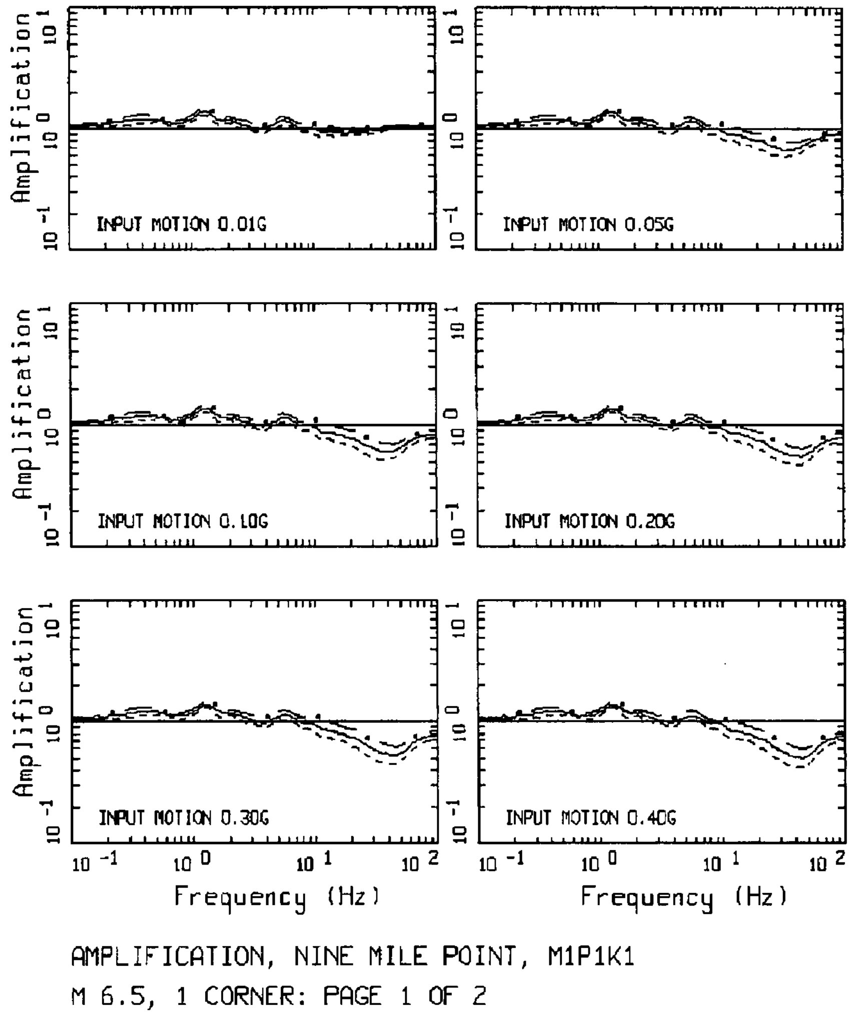
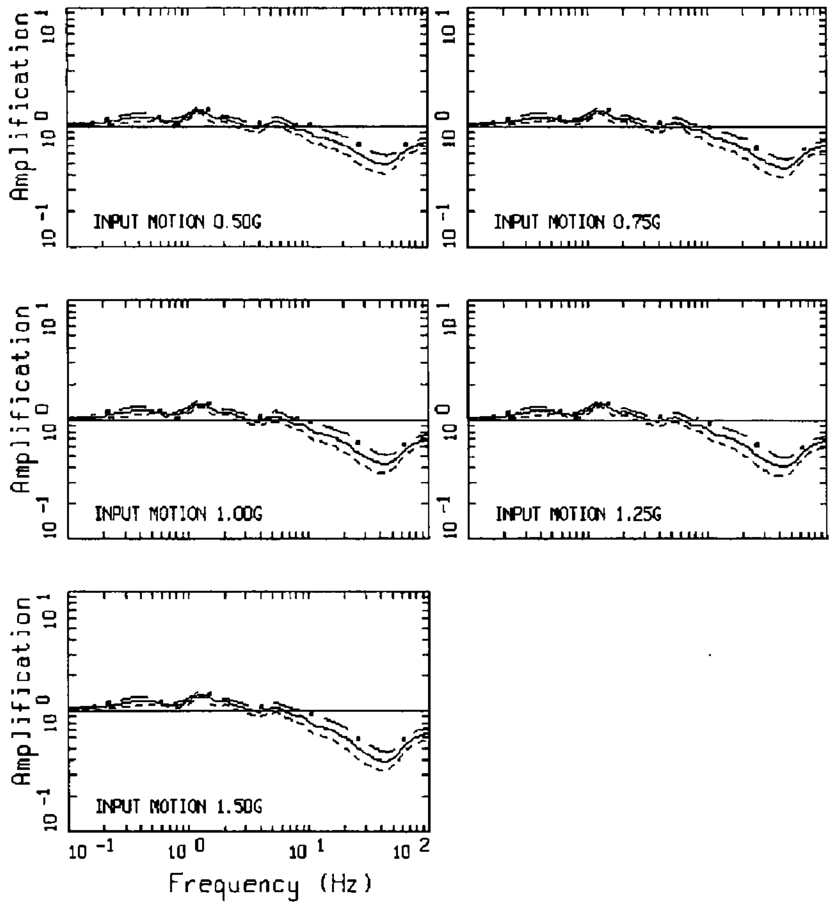


Figure 2.3.6-1 Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), EPRI rock modulus reduction and hysteretic damping curves (model M1), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g; **M** 6.5 and single-corner source model (Reference 3)

Figure 2.3.6-1 (cont'd)



AMPLIFICATION, NINE MILE POINT, M1P1K1
M 6.5, 1 CORNER: PAGE 2 OF 2

Figure 2.3.6-2

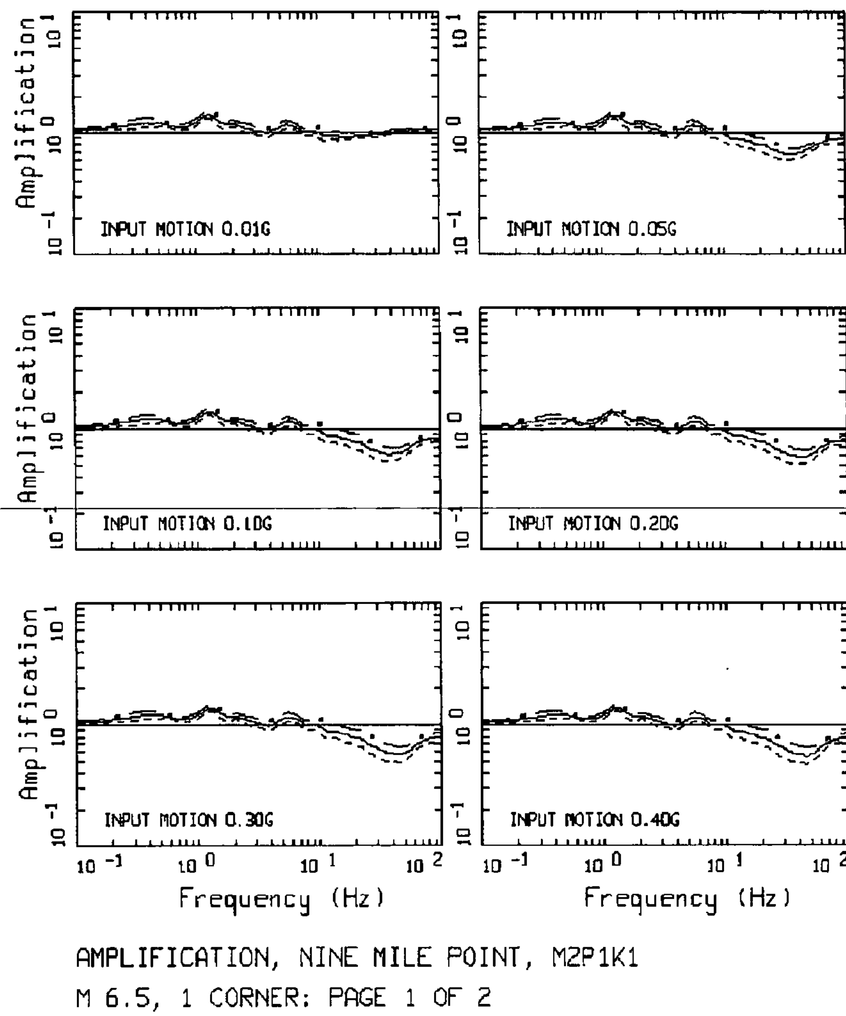
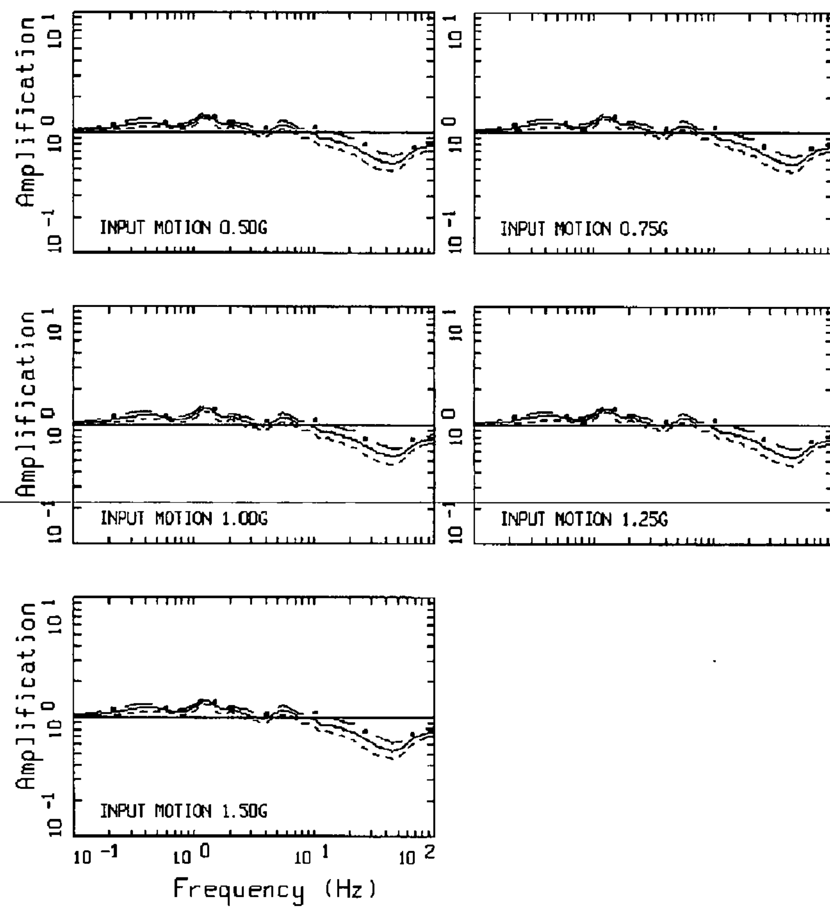


Figure 2.3.6-2 Example suite of amplification factors (5% damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), linear site response (model M2), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g to 1.50g; **M** 6.5 and single-corner source model (Reference 3)

Figure 2.3.6-2 (cont'd)



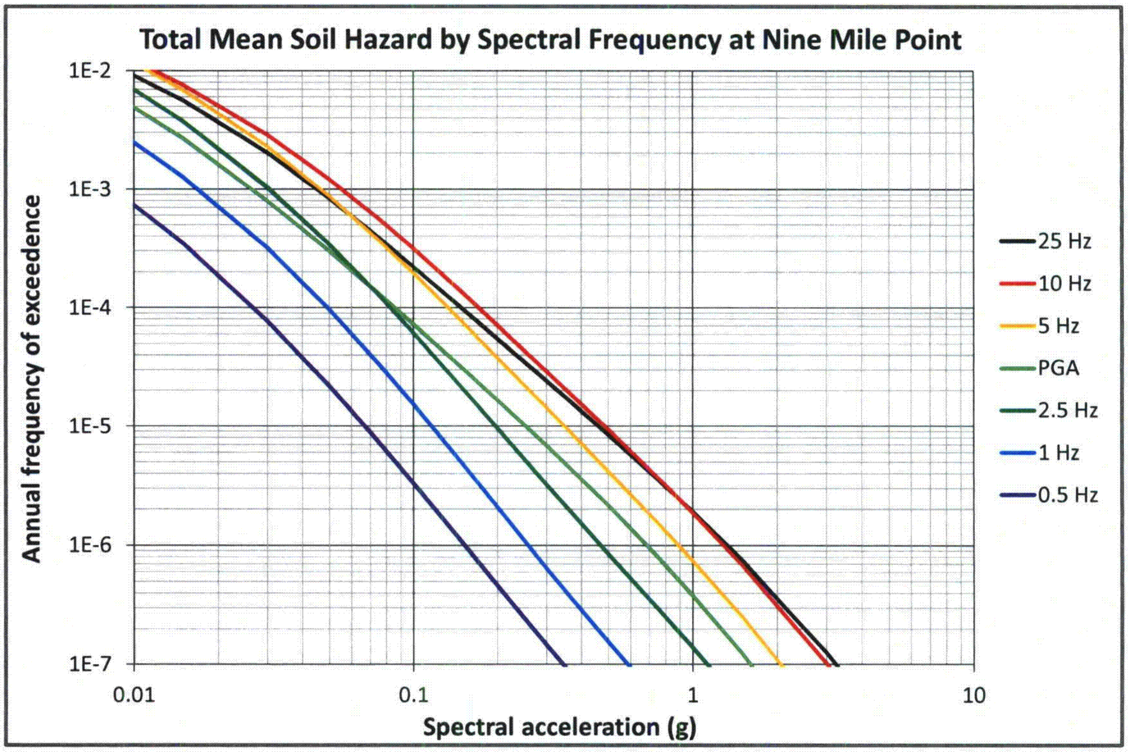
AMPLIFICATION, NINE MILE POINT, M2P1K1

M 6.5, 1 CORNER: PAGE 2 OF 2

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 (Reference 3). 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 Nine Mile Point are shown in Figure 2.3.7-1 (Reference 27) for the seven oscillator frequencies for which the GMM is defined. Tabulated values of the control point 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 (PGA) at Nine Mile Point



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.

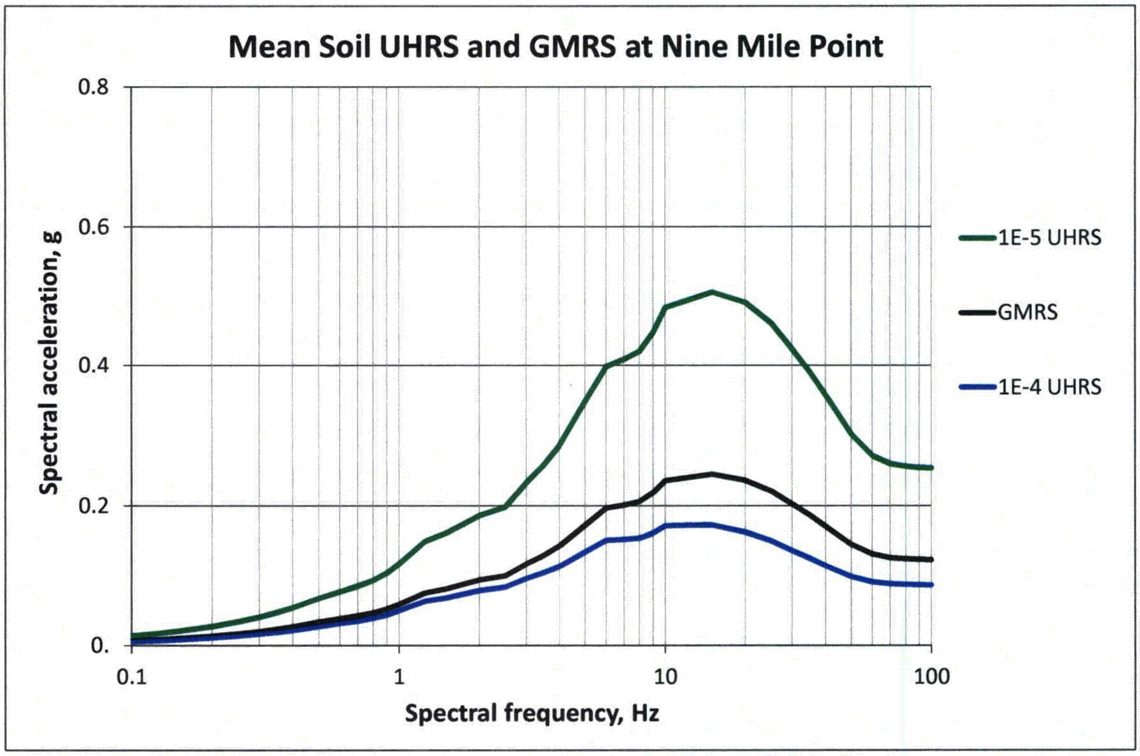
The 1E-4 and 1E-5 UHRS, along with a design factor (DF) are used to compute the GMRS at the control point using the criteria in Regulatory Guide 1.208 (Reference 10). Table 2.4-1 shows the UHRS and GMRS spectral accelerations (Reference 27).

Table 2.4-1 UHRS for 1E-4 and 1E-5 and GMRS at control point for Nine Mile Point

Freq. (Hz)	1E-4 UHRS (g)	1E-5 UHRS (g)	GMRS (g)
100	8.59E-02	2.53E-01	1.22E-01
90	8.61E-02	2.54E-01	1.23E-01
80	8.66E-02	2.55E-01	1.23E-01
70	8.76E-02	2.59E-01	1.25E-01
60	9.05E-02	2.70E-01	1.30E-01
50	9.83E-02	3.01E-01	1.44E-01
40	1.14E-01	3.57E-01	1.70E-01
35	1.24E-01	3.89E-01	1.86E-01
30	1.35E-01	4.23E-01	2.02E-01
25	1.49E-01	4.61E-01	2.21E-01
20	1.62E-01	4.91E-01	2.36E-01
15	1.72E-01	5.06E-01	2.45E-01
12.5	1.72E-01	4.95E-01	2.41E-01
10	1.71E-01	4.83E-01	2.36E-01
9	1.60E-01	4.47E-01	2.19E-01
8	1.53E-01	4.20E-01	2.06E-01
7	1.51E-01	4.08E-01	2.01E-01
6	1.50E-01	3.98E-01	1.96E-01
5	1.33E-01	3.48E-01	1.72E-01
4	1.12E-01	2.85E-01	1.42E-01
3.5	1.04E-01	2.58E-01	1.29E-01
3	9.50E-02	2.32E-01	1.16E-01
2.5	8.28E-02	1.97E-01	9.96E-02
2	7.79E-02	1.85E-01	9.34E-02
1.5	6.77E-02	1.60E-01	8.10E-02
1.25	6.27E-02	1.48E-01	7.48E-02
1	4.94E-02	1.16E-01	5.88E-02
0.9	4.34E-02	1.03E-01	5.21E-02
0.8	3.85E-02	9.27E-02	4.67E-02
0.7	3.46E-02	8.44E-02	4.24E-02
0.6	3.10E-02	7.65E-02	3.83E-02
0.5	2.67E-02	6.68E-02	3.34E-02
0.4	2.13E-02	5.35E-02	2.67E-02
0.35	1.87E-02	4.68E-02	2.34E-02
0.3	1.60E-02	4.01E-02	2.00E-02
0.25	1.33E-02	3.34E-02	1.67E-02
0.2	1.07E-02	2.67E-02	1.33E-02
0.15	8.00E-03	2.01E-02	1.00E-02
0.125	6.67E-03	1.67E-02	8.34E-03
0.1	5.33E-03	1.34E-02	6.67E-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 (Reference 27).

Figure 2.4-1 UHRS for 1E-4 and 1E-5 and GMRS at Control Point for Nine Mile Point



3 Plant Design Basis [and Beyond Design Basis Evaluation Ground Motion]

The design basis earthquake for NMP1 is identified in Section III of the UFSAR (Reference 11) and the design basis earthquake for NMP2 is identified in Section 2.5.2.3 of the UFSAR (Reference 12).

3.1 SSE Description of Spectral Shape

The SSE is defined in terms of a PGA and a design response spectrum. The SSE shape and anchor point are different for NMP1 and NMP2 and are each discussed below.

NMP1 SSE was based upon the maximum credible earthquake motion, which was determined to be a magnitude 7 (Intensity IX) shock 50 miles from the site. The maximum value of 11% of gravity (0.11g) was recommended to be used for ridged critical structures as the design parameter for the "maximum possible" condition in the foundation rock at the proposed power plant site (Reference 29). The associated maximum ground motion spectral shape is a Housner-type spectrum anchored to 0.11g PGA (Reference 11).

As part of the NMP1 seismic reevaluation program, an upgraded design basis SSE ground response spectrum (GRS) was developed based upon methods established during the NRC's Systematic Evaluation Program (SEP). Based upon the seismogenic zones and their credibility, the upgraded SSE is anchored to a PGA of 0.13g (Reference 15). The upgraded SSE was used in the NMP1 seismic evaluations in USI A-46 and Individual Plant Examination for External Events (IPEEE) (References 13, 14 and 16). Therefore, the upgraded GRS anchored to 0.13g PGA represents the current SSE for NMP1 and is appropriate for NTTF 2.1: Seismic screening.

The NMP1 upgraded SSE spectral shape is based upon the PGA of 0.13g and the spectrum amplification factors provided in NUREG/CR-0098 (References 15 and 28).

The NMP2 SSE was developed in accordance with 10 CFR 100, Appendix A through an evaluation of the maximum earthquake potential for the region surrounding the site. The maximum expected earthquake intensity was developed considering historical earthquakes caused by the two geological structures as well as the tectonic provinces within 200 miles from the site. The controlling event leading to the maximum earthquake intensity at the site is based on the occurrence of a Modified Mercalli Intensity VI earthquake adjacent to the site related to earthquake activity originating from the site tectonic province (Reference 12, Section 2.5.2.4).

Considering the maximum earthquake intensity of VI for NMP2, the peak horizontal acceleration was found to be 7% of gravity (0.07g). However, to be very conservative, an acceleration of 15% of gravity (0.15g) was adopted for the horizontal design response spectrum PGA (Reference 12, Section 2.5.2.6). The design spectrum anchored to a PGA of 0.15g should effectively envelop structural response resulting from the occurrence of the design event near the site or response from long-duration, low-amplitude motion generated by large, distant events. The site design response spectrum for the NMP2 SSE has a Reg. Guide 1.60 spectral shape (Reference 12, Section 3.7A.1.1).

The 5% damping horizontal SSE for both NMP1 and NMP2 are shown in Figure 3.1-1 and Figure 3.1-2 respectively. Table 3.1-1 shows the spectral acceleration values as a function of frequency for the horizontal SSE for NMP1 (Reference 15). Table 3.1-2 shows the spectral acceleration values as a function of frequency horizontal SSE for NMP2 (Reference 12, Figure 3.7A-1).

Table 3.1-1 Horizontal Safe Shutdown Earthquake Response Spectrum for NMP1

Freq (Hz)	SSE (g)
0.2	0.018
0.3	0.027
0.4	0.036
0.5	0.045
0.6	0.054
0.7	0.063
0.8	0.072
0.9	0.080
1	0.089
1.5	0.134
2	0.179
2.5	0.224
3	0.268
4	0.276
8	0.276
9	0.259
10	0.245
12.5	0.218
15	0.197
20	0.170
25	0.151
40	0.130
100/PGA	0.130

Figure 3.1-1 Horizontal SSE for NMP1

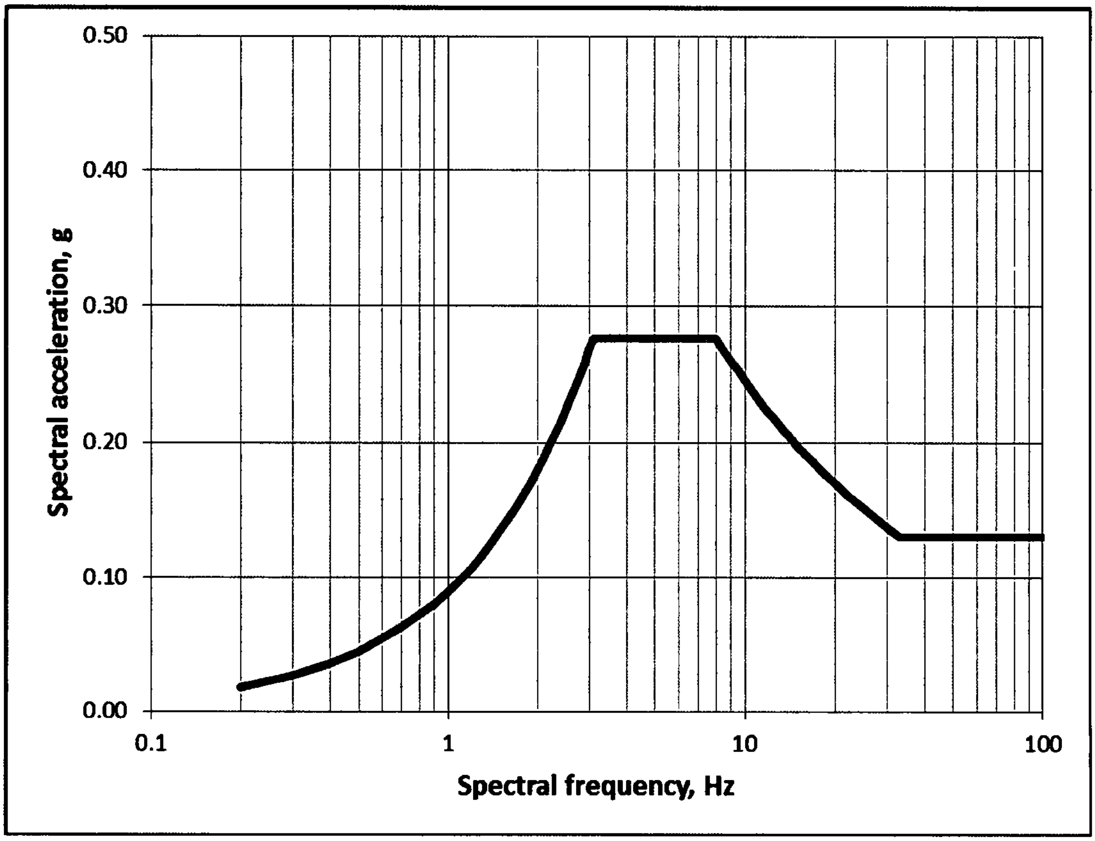
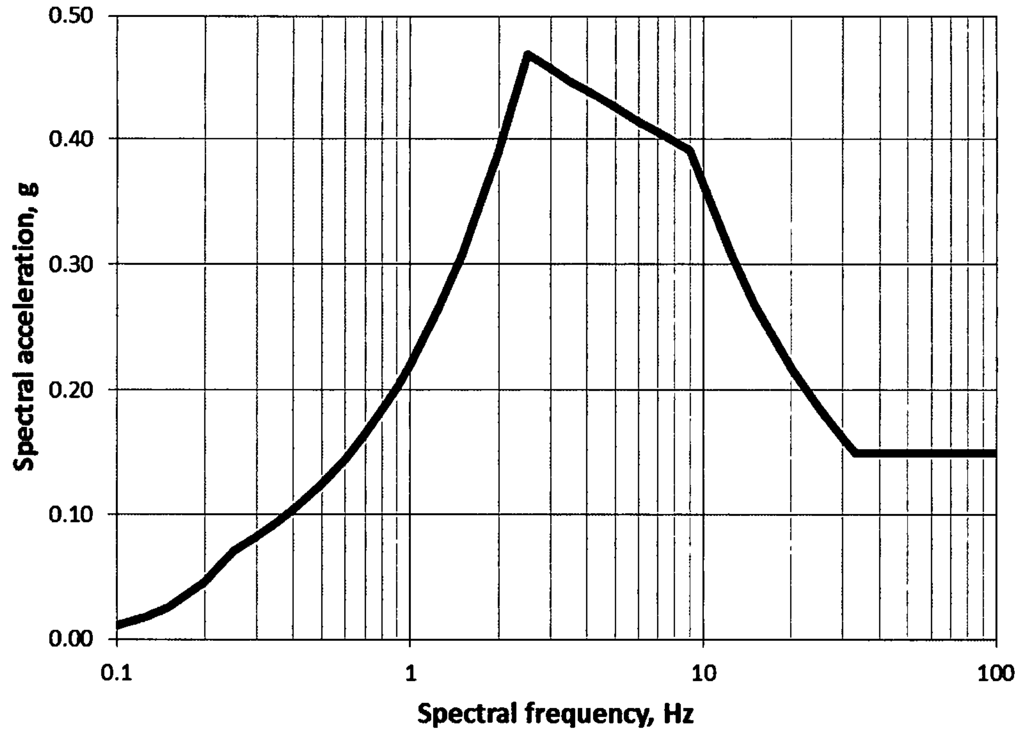


Table 3.1-2 Horizontal Safe Shutdown Earthquake Response Spectrum for NMP2

Freq (Hz)	SSE (g)
0.1	0.01
0.25	0.07
2.5	0.47
9	0.39
33	0.15
100/PGA	0.15

Figure 3.1-2 Horizontal SSE for NMP2



3.2 CONTROL POINT ELEVATION

Neither the UFSAR for NMP1 nor the USAR for NMP2 defines an SSE control point. The site lies within the Lake Iroquois lake plain, and lacustrine sediments subdue the surface of glacial till overlying bedrock (Reference 12, Section 2.5.1.2.1). Major Category I structures are founded on sound rock (bedrock) (Reference 11, Section III and Reference 12, Section 3.7A.1.4). The top of bedrock is encountered at elevations ranging from 246 ft. to 240 ft. (Reference 12, Section 3.7A.1.4). Because NMP is considered a rock site, the SSE control point elevation is taken to be at the top of the rock surface at approximately Elevation 245 ft. USLS-35 (or approximately Elevation 245 ft. MSL). This definition of the control point is consistent with the approach described in the SPID (Reference 3, Section 2.4.2).

4 Screening Evaluation

In accordance with the 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 of the response spectrum for both NMP1 and NMP2, the SSE exceeds the GRMS. Therefore, a risk evaluation will not be performed.

4.2 High Frequency Screening (>10Hz)

For a portion of the range above 10 Hz, the GMRS exceeds the SSE. Therefore, both NMP1 and NMP2 screens in for a high frequency confirmation.

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

In the 1 to 10 Hz part of the response spectrum, the SSE exceeds the GMRS for both NMP1 and NMP2. Therefore, a spent fuel pool evaluation will not be performed.

5 Interim Actions

Based on the screening evaluation outcome described in Section 4 of this report, existing NMP1 and NMP2 seismic capacity is sufficient to demonstrate and ensure continued seismic safety of each plant relative to the reevaluated seismic hazard. Although a comprehensive seismic risk assessment (SPRA or SMA) is not required, high frequency confirmations will be performed for NMP1 and NMP2 in response to the section of the 50.54(f) letter (Reference 1) request for information pertaining to seismic hazard reevaluation in response to NTTF Recommendations 2.1.

5.1 Expedited Seismic Evaluation Process (ESEP)

The nuclear power industry has proposed, and CENG has committed to follow the "Augmented Approach" guidance document (Reference 6) to fulfill Enclosure 1: Seismic of Reference 1 request for information, regarding seismic aspects of NTTF Recommendation 2.1. The ESEP, contained within Reference 6, adds the additional short term aspect to the overall response to NTTF Recommendation 2.1. The ESEP addresses the part of Reference 1 that requests "interim evaluations and actions taken or planned to address the higher seismic hazard relative to the design basis, as appropriate, prior to completion of the risk evaluation." Specifically, the ESEP focuses initial industry efforts on short term evaluations that will lead to prompt modifications to some of the most important components that could improve plant seismic safety.

As described in Section 4 of this report, the SSE envelopes the GMRS between 1 and 10 Hz. Therefore, Nine Mile Point screens out of the ESEP based on Section 2.2 of Reference 6.

5.2 Interim Evaluation of Seismic Hazard

Consistent with NRC letter dated February 20, 2014 (Reference 30) the seismic hazard reevaluations presented herein are distinct from the current design and licensing bases of NMP1 and NMP2. Therefore, the results do not call into question the operability or functionality of SSCs and are not reportable pursuant to10 CFR 50.72, "Immediate notification requirements for operating nuclear power reactors," and10 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, Nuclear Energy Institute (NEI) letter dated March 12, 2014 (Reference 31) 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 (Reference 23):

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.

NMP1 and NMP2 are included in the March 12, 2014 (Reference 31) 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.

5.3 SEISMIC WALKDOWN INSIGHTS

In response to NTTF Recommendation 2.3, Seismic Walkdowns for the Nine Mile Point station have been completed as documented in References 18, 19, 25 and 26. All potentially degraded, nonconforming, or unanalyzed conditions identified as a result of the seismic walkdowns were entered into the corrective action program to be addressed. The primary findings of the Near-Term Task Force Recommendations 2.3 seismic walkdowns for both NMP1 and NMP2 (References 19 and 26), identified no adverse seismic conditions that challenged the licensing basis for either unit. Also, verified that no major plant vulnerabilities or physical plant improvements were outstanding from the station Individual Plant Examination of External Events (IPEEE) submittals.

The seismic walkdowns for NMP1 have been completed (Reference 18 and 26), and for NMP2, seismic walkdown of 17 items were deferred for completion in the spring 2014 Refueling Outage. These items are identified in Table E-2 in Attachment 1 in Reference 25.

Seismic walkdown results indicate that the seismic capability of systems, structures and components is being maintained.

5.4 BEYOND DESIGN BASIS SEISMIC INSIGHTS

For the seismic Individual Plant Examination of External Events (IPEEE) analysis, NMP1 and NMP2 were categorized as a 0.3g focused-scope plant (per NUREG-1407, Reference 22). Both plants performed an EPRI seismic margin analysis (EPRI SMA) (References 16 and 17). For NMP2 a review level earthquake (RLE) of 0.5g was conservatively used for screening, rather than the value of 0.3g recommended in NUREG-1407. In addition to performing an EPRI SMA, NMP2 also performed a seismic PRA (Reference 17).

The IPEEE submittal for NMP1 (Reference 16) identified a number of improvements or initiatives during the IPEEE process that were completed. The IPEEE seismic improvement initiatives for the equipment in the success path included; improving lateral supports for main control room panels, strengthening/improving cabinet base anchorages, and improving cable tray supports. The High Confidence Low Probability of Failure (HCLPF) value of ~0.3g was used for these improvements. These improvements were completed by the end of the 1999 refueling outage. Following the improvements which were completed to increase the HCLPF capacities of low-rugged components, NMP1 plant HCLPF value is 0.27g. In addition, relays that were susceptible to "chatter" in the emergency diesel generator system were replaced.

The key NMP2 IPEEE (Reference 17) findings reported for the seismic events is that all structures, systems and components in the simplified success path screened out for a HCLPF value equal to or greater than 0.5g. The mean seismic core damage frequency was calculated to be 2.5×10^{-7} per reactor-year (ry), using the EPRI seismic hazard results, and 1.2×10^{-6} per reactor year using the LLNL seismic hazard results.

The IPEEE submittal for NMP2 (Reference 17) identified one initiative during the IPEEE that was completed. The IPEEE seismic improvement was to install seismic restraints on emergency switchgear panel hoist assemblies. It was noted that several safety-related electrical cabinets included a hoist assembly located on the top of the panel which could move and jar equipment during an earthquake. Rail stops were installed to preclude this problem from occurring.

6 Conclusions

In accordance with the 50.54(f) letter (Reference 1), a seismic hazard and screening evaluation was performed for Nine Mile Point. A GMRS was developed solely for purpose of screening for additional evaluations in accordance with the SPID (Reference 3).

Based on the results of the screening evaluation, the SSE envelopes the GMRS in the frequency range of 1 to 10 Hz, therefore both NMP1 and NMP2 are not required to perform a risk evaluation or spent fuel pool integrity evaluations. Additionally, NMP1 and NMP2 screen out of the ESEP interim action per the "Augmented Approach" guidance document, Section 2.2 (Reference 4).

Based on the results of the screening evaluation, the GMRS exceeds both SSEs in the frequency range above 10 Hz, high frequency confirmations will be performed for both units in accordance with the SPID Section 3.4 (Reference 3). These evaluations will be completed based upon the schedule for CEUS nuclear plants provided in the April 9, 2013 letter from industry to the NRC (Reference 6).

7 References

1. NRC Letter (E. J. Leeds) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3, of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* March 2012.

2. Title 10 Code of Federal Regulations Part 50

3. EPRI 1025287, *Seismic Evaluation Guidance: Screening, Prioritization and Implementation Details (SPID) for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, Palo Alto, CA, February 2013.

4. EPRI 3002000704, *Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic*, Palo Alto, CA, May 2013.

5. Title 10 Code of Federal Regulations Part 100

6. NEI Letter (A. R. Pietrangelo) to the NRC, *Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations,* dated April 9, 2013.

7. EPRI 1021097 (NUREG-2115), *Central and Eastern United States Seismic Source Characterization for Nuclear Facilities*, Palo Alto, CA, January 2012.

8. EPRI 3002000717, *EPRI (2004, 2006) Ground-Motion Model (GMM) Review Project*, Palo Alto, CA, June 2013.

9. Silva, W.J., N. Abrahamson, G. Toro and C. Costantino, "Description and validation of the stochastic ground motion model," Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573, 1997.

10. NRC Regulatory Guide 1.208, "A performance-based approach to define the site-specific earthquake ground motion," 2007.

11. Nine Mile Point Nuclear Station Unit 1 Final Safety Analysis Report (Updated) (UFSAR), Revision 22.

12. Nine Mile Point Nuclear Station Unit 2 Updated Safety Analysis Report (USAR), Revision 20.

13. Niagara Mohawk Power Corporation letter to the NRC, *Response to Supplement 1 to Generic Letter 87-02 dated May 22, 1992,* NMP1L 0695, dated September 18, 1992.

References (cont'd)

14. Niagara Mohawk Power Corporation letter to the NRC, *Summary Report for Resolution of USI A-46,* NMP1L 1044, dated March 11, 1996.

15. Nine Mile Point Nuclear Station Unit 1, Calculation No. S0.0-S3.7.1-SPECTR02, *Dames and Moore Seismic Response Spectra Calculation,* Revision 1.

16. Nine Mile Point Nuclear Station Unit 1, *Individual Plant Examination for External Events (IPEEE)*, SAS-TR-96-001, dated August 1996.

17. Nine Mile Point Nuclear Station Unit 2, *Individual Plant Examination for External Events (IPEEE)*, SAS-TR-95-001, dated June 1995.

18. CENG letter (J. A. Spina) to the NRC, *Nine Mile Point Nuclear Station, Unit 1 – Supplemental Response to 10 CFR 50.54(f) Request for Information, Recommendation 2.3, Seismic*, NMP1L 2821, dated July 12, 2013.

19. CENG letter (M. G. Korsnick) to the NRC, *Nine Mile Point Nuclear Station, Unit 2 – Response to 10 CFR 50.54(f) Request for Information, Recommendation 2.3, Seismic*, NMP2L 2459, dated *November 27*, 2012.

20. CENG letter (M. G. Korsnick) to the NRC, *Response to Request for Information: Near-Term Task Force Recommendation 2.1, Seismic Reevaluation*, NMP1L 2795, dated April 26, 2013.

21. Engineering Position Paper, *Data Request for Site Amplification Calculations for Nine Mile Point Nuclear Station Unit 1 and 2*, Revision 1, dated August 13, 2013

22. NUREG-1407, *Procedural and Submittal Guidance for the Individual Plant Examination of External Events (IPEEE) for Severe Accident Vulnerabilities,* dated June 1991

23. NUREG-0933, A Prioritization of Generic Safety Issues, Supplement 34, Resolution of Generic Safety issues, Revision 1, September, 2011.

24. Final Safety Analysis Report (FSAR) of the Nine Mile Point Unit 3 Combined License Application, Revision 1, Section 2.5 – Geology, Seismology, and Geotechnical Engineering, Unistar Nuclear Energy (UNE).

25. CENG letter (J. A. Spina) to the NRC, Nine Mile Point Nuclear Station, Submittal of Nine Mile Point Nuclear Station, Unit 2 Supplemental Seismic Walkdown Report, NMP2L 2463, dated January 31, 2013

26. CENG letter (M. G. Korsnick) to the NRC, Nine Mile Point Nuclear Station, Unit 1 – Response to 10 CFR 50.54(f) Request for Information, Recommendation 2.3, Seismic, NMP2L 2722, dated November 27, 2012.

27. Lettis Consultants International (LCI) Inc., Project No. 1041, *Nine Mile Point Seismic Hazard and Screening Report*, dated November 25, 2013.

References (cont'd)

28. NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants," May 1978.

29. Preliminary Hazards Summary Report (PHSR) Nine Mile Point Nuclear Station (Unit 1), Volume II, Appendix C, Report – Site Evaluation Study – Phase II Proposed Nine Mile Point Nuclear Power Plant Near Oswego, New York

30. NRC Letter (E.J. Leeds) to All Power Reactor Licensees and Holders of Construction Permits in Active or Deferred Status, *Supplemental Information Related for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Seismic Hazard Reevaluations for Recommendation 2.1 of the Near-Term Task Force Review of Insights from Fukushima Dai-ichi Accident* dated February 20, 2014.

31. NEI Letter (A. R. Pietrangelo) to the NRC), *Seismic Risk Evaluations for Plants in the Central and Eastern United States,* dated March 12, 2014.

A

Additional Tables

The following additional tables are included in Appendix A:

- Table A-1a: Mean and Fractile Seismic Hazard Curves for PGA (100 Hz) at NMP
- Table A-1b: Mean and Fractile Seismic Hazard Curves for 25 Hz at NMP
- Table A-1c: Mean and Fractile Seismic Hazard Curves for 10 Hz at NMP
- Table A-1d: Mean and Fractile Seismic Hazard Curves for 5 Hz at NMP
- Table A-1e: Mean and Fractile Seismic Hazard Curves for 2.5 Hz at NMP
- Table A-1f: Mean and Fractile Seismic Hazard Curves for 1 Hz at NMP
- Table A-1g: Mean and Fractile Seismic Hazard Curves for 0.5 Hz at NMP
- Table A-2: Medians and Logarithmic Sigmas Of Amplification Factors for NMP
- Table A-2-b1: Median AFs and Sigmas for Model 1 for 2 PGA Levels
- Table A-2-b2: Median AFs and Sigmas for Model 2 for 2 PGA Levels

Table A-1a Mean and Fractile Seismic Hazard Curves for PGA (100Hz) at NMP

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.59E-02	3.14E-02	4.37E-02	5.58E-02	6.93E-02	7.77E-02
0.001	4.18E-02	1.95E-02	3.05E-02	4.13E-02	5.42E-02	6.36E-02
0.005	1.16E-02	3.52E-03	6.36E-03	1.01E-02	1.62E-02	2.46E-02
0.01	4.91E-03	1.25E-03	2.13E-03	3.90E-03	6.83E-03	1.34E-02
0.015	2.68E-03	6.09E-04	9.93E-04	1.95E-03	3.73E-03	8.47E-03
0.03	8.04E-04	1.36E-04	2.13E-04	4.63E-04	1.08E-03	3.23E-03
0.05	3.00E-04	3.68E-05	6.00E-05	1.36E-04	3.95E-04	1.34E-03
0.075	1.32E-04	1.32E-05	2.29E-05	5.42E-05	1.72E-04	6.09E-04
0.1	7.31E-05	6.83E-06	1.21E-05	3.01E-05	9.37E-05	3.33E-04
0.15	3.11E-05	2.88E-06	5.50E-06	1.38E-05	4.07E-05	1.32E-04
0.3	6.92E-06	6.09E-07	1.36E-06	3.68E-06	9.65E-06	2.39E-05
0.5	2.16E-06	1.51E-07	3.95E-07	1.21E-06	3.33E-06	7.03E-06
0.75	8.08E-07	3.95E-08	1.21E-07	4.37E-07	1.32E-06	2.72E-06
1.	3.85E-07	1.27E-08	4.56E-08	1.95E-07	6.45E-07	1.36E-06
1.5	1.25E-07	2.04E-09	9.37E-09	5.35E-08	2.13E-07	4.70E-07
3.	1.36E-08	1.07E-10	3.84E-10	3.52E-09	2.13E-08	5.91E-08
5.	1.99E-09	4.56E-11	9.11E-11	3.42E-10	2.72E-09	9.24E-09
7.5	3.49E-10	3.79E-11	5.05E-11	9.51E-11	4.50E-10	1.69E-09
10.	9.02E-11	3.01E-11	4.01E-11	9.11E-11	1.42E-10	4.77E-10

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.06E-02	4.01E-02	4.90E-02	6.00E-02	7.34E-02	8.12E-02
0.001	4.84E-02	2.80E-02	3.79E-02	4.77E-02	6.00E-02	6.93E-02
0.005	1.79E-02	6.93E-03	1.13E-02	1.62E-02	2.42E-02	3.37E-02
0.01	9.11E-03	2.92E-03	4.90E-03	7.77E-03	1.27E-02	2.04E-02
0.015	5.60E-03	1.62E-03	2.72E-03	4.63E-03	8.00E-03	1.38E-02
0.03	2.03E-03	4.77E-04	7.66E-04	1.51E-03	2.96E-03	5.91E-03
0.05	8.37E-04	1.57E-04	2.53E-04	5.50E-04	1.23E-03	2.72E-03
0.075	3.90E-04	5.66E-05	9.79E-05	2.29E-04	5.75E-04	1.34E-03
0.1	2.22E-04	2.76E-05	4.98E-05	1.23E-04	3.28E-04	7.77E-04
0.15	9.86E-05	1.07E-05	2.01E-05	5.20E-05	1.44E-04	3.52E-04
0.3	2.42E-05	2.60E-06	5.20E-06	1.32E-05	3.57E-05	7.89E-05
0.5	8.47E-06	8.60E-07	1.90E-06	5.05E-06	1.31E-05	2.60E-05
0.75	3.61E-06	3.28E-07	7.77E-07	2.25E-06	5.83E-06	1.08E-05
1.	1.93E-06	1.55E-07	3.95E-07	1.21E-06	3.23E-06	5.91E-06
1.5	7.58E-07	4.63E-08	1.32E-07	4.63E-07	1.31E-06	2.42E-06
3.	1.26E-07	3.84E-09	1.40E-08	6.26E-08	2.25E-07	4.56E-07
5.	2.71E-08	4.70E-10	1.84E-09	1.05E-08	4.83E-08	1.07E-07
7.5	6.87E-09	1.16E-10	3.23E-10	1.98E-09	1.16E-08	2.92E-08
10.	2.37E-09	9.11E-11	1.23E-10	5.75E-10	3.79E-09	1.05E-08

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.70E-02	5.12E-02	5.58E-02	6.54E-02	7.89E-02	8.72E-02
0.001	5.69E-02	3.90E-02	4.56E-02	5.58E-02	6.83E-02	7.66E-02
0.005	2.34E-02	1.08E-02	1.57E-02	2.22E-02	3.14E-02	3.84E-02
0.01	1.22E-02	4.70E-03	7.13E-03	1.10E-02	1.72E-02	2.22E-02
0.015	7.57E-03	2.68E-03	4.07E-03	6.64E-03	1.10E-02	1.49E-02
0.03	2.85E-03	8.72E-04	1.31E-03	2.35E-03	4.25E-03	6.45E-03
0.05	1.21E-03	3.33E-04	5.05E-04	9.37E-04	1.82E-03	3.05E-03
0.075	5.63E-04	1.38E-04	2.13E-04	4.13E-04	8.35E-04	1.55E-03
0.1	3.15E-04	7.03E-05	1.11E-04	2.22E-04	4.70E-04	9.24E-04
0.15	1.34E-04	2.60E-05	4.19E-05	8.85E-05	1.98E-04	4.13E-04
0.3	2.90E-05	4.63E-06	8.23E-06	1.90E-05	4.37E-05	8.98E-05
0.5	9.26E-06	1.36E-06	2.64E-06	6.36E-06	1.44E-05	2.72E-05
0.75	3.69E-06	5.05E-07	1.05E-06	2.60E-06	5.91E-06	1.04E-05
1.	1.88E-06	2.32E-07	5.27E-07	1.34E-06	3.09E-06	5.27E-06
1.5	6.93E-07	7.03E-08	1.74E-07	4.83E-07	1.16E-06	2.04E-06
3.	1.02E-07	5.91E-09	1.77E-08	6.26E-08	1.74E-07	3.37E-07
5.	1.97E-08	6.73E-10	2.22E-09	9.93E-09	3.37E-08	7.45E-08
7.5	4.56E-09	1.38E-10	3.68E-10	1.84E-09	7.66E-09	1.87E-08
10.	1.47E-09	9.11E-11	1.32E-10	5.20E-10	2.39E-09	6.54E-09

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.83E-02	5.12E-02	5.75E-02	6.64E-02	8.00E-02	8.85E-02
0.001	5.89E-02	3.95E-02	4.63E-02	5.83E-02	7.23E-02	8.00E-02
0.005	2.38E-02	1.04E-02	1.51E-02	2.32E-02	3.28E-02	3.95E-02
0.01	1.16E-02	4.19E-03	6.64E-03	1.10E-02	1.67E-02	2.13E-02
0.015	6.84E-03	2.25E-03	3.63E-03	6.26E-03	1.02E-02	1.32E-02
0.03	2.28E-03	6.54E-04	1.08E-03	1.95E-03	3.52E-03	4.98E-03
0.05	8.70E-04	2.25E-04	3.73E-04	7.03E-04	1.36E-03	2.07E-03
0.075	3.72E-04	8.85E-05	1.44E-04	2.84E-04	5.91E-04	9.37E-04
0.1	1.96E-04	4.31E-05	7.03E-05	1.46E-04	3.09E-04	5.20E-04
0.15	7.59E-05	1.46E-05	2.49E-05	5.42E-05	1.20E-04	2.13E-04
0.3	1.43E-05	2.29E-06	4.25E-06	9.93E-06	2.25E-05	4.13E-05
0.5	4.16E-06	5.66E-07	1.16E-06	2.92E-06	6.73E-06	1.18E-05
0.75	1.55E-06	1.77E-07	4.01E-07	1.07E-06	2.53E-06	4.50E-06
1.	7.50E-07	7.23E-08	1.77E-07	5.12E-07	1.25E-06	2.25E-06
1.5	2.56E-07	1.77E-08	4.98E-08	1.64E-07	4.43E-07	8.12E-07
3.	3.29E-08	1.10E-09	3.73E-09	1.67E-08	5.66E-08	1.21E-07
5.	5.76E-09	1.49E-10	4.25E-10	2.25E-09	9.51E-09	2.35E-08
7.5	1.24E-09	9.11E-11	1.10E-10	3.95E-10	1.90E-09	5.42E-09
10.	3.79E-10	4.07E-11	9.11E-11	1.38E-10	5.66E-10	1.74E-09

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	6.48E-02	4.70E-02	5.27E-02	6.36E-02	7.77E-02	8.60E-02
0.001	5.26E-02	3.28E-02	3.95E-02	5.20E-02	6.64E-02	7.55E-02
0.005	1.66E-02	6.93E-03	9.93E-03	1.57E-02	2.35E-02	2.92E-02
0.01	6.99E-03	2.46E-03	3.79E-03	6.45E-03	1.04E-02	1.34E-02
0.015	3.75E-03	1.18E-03	1.90E-03	3.33E-03	5.66E-03	7.66E-03
0.03	1.04E-03	2.76E-04	4.56E-04	8.60E-04	1.64E-03	2.42E-03
0.05	3.43E-04	7.77E-05	1.31E-04	2.64E-04	5.58E-04	8.60E-04
0.075	1.29E-04	2.57E-05	4.37E-05	9.51E-05	2.13E-04	3.47E-04
0.1	6.17E-05	1.10E-05	1.95E-05	4.37E-05	1.01E-04	1.74E-04
0.15	2.09E-05	3.19E-06	5.91E-06	1.42E-05	3.42E-05	6.17E-05
0.3	3.24E-06	3.63E-07	7.66E-07	2.04E-06	5.35E-06	1.01E-05
0.5	8.56E-07	6.36E-08	1.62E-07	5.05E-07	1.44E-06	2.80E-06
0.75	3.00E-07	1.40E-08	4.25E-08	1.62E-07	5.20E-07	1.04E-06
1.	1.40E-07	4.43E-09	1.53E-08	6.93E-08	2.42E-07	5.12E-07
1.5	4.54E-08	7.89E-10	3.19E-09	1.87E-08	7.89E-08	1.79E-07
3.	5.28E-09	9.37E-11	1.98E-10	1.40E-09	8.35E-09	2.35E-08
5.	8.52E-10	4.56E-11	9.11E-11	1.84E-10	1.20E-09	3.95E-09
7.5	1.70E-10	3.57E-11	4.50E-11	9.11E-11	2.35E-10	8.23E-10
10.	4.92E-11	3.01E-11	4.01E-11	9.11E-11	1.05E-10	2.64E-10

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	4.57E-02	2.25E-02	3.09E-02	4.56E-02	6.00E-02	6.93E-02
0.001	3.03E-02	1.23E-02	1.84E-02	2.96E-02	4.19E-02	5.05E-02
0.005	6.50E-03	1.72E-03	3.05E-03	5.83E-03	9.93E-03	1.36E-02
0.01	2.50E-03	4.83E-04	9.51E-04	2.07E-03	4.01E-03	5.91E-03
0.015	1.27E-03	1.98E-04	4.07E-04	9.93E-04	2.13E-03	3.33E-03
0.03	3.23E-04	3.23E-05	7.34E-05	2.19E-04	5.58E-04	9.79E-04
0.05	9.72E-05	7.03E-06	1.67E-05	5.66E-05	1.69E-04	3.28E-04
0.075	3.39E-05	1.90E-06	4.70E-06	1.72E-05	5.83E-05	1.21E-04
0.1	1.53E-05	7.45E-07	1.82E-06	6.93E-06	2.57E-05	5.66E-05
0.15	4.82E-06	1.84E-07	4.77E-07	1.90E-06	7.66E-06	1.87E-05
0.3	6.52E-07	1.38E-08	4.50E-08	2.10E-07	9.51E-07	2.60E-06
0.5	1.56E-07	1.57E-09	6.64E-09	4.19E-08	2.22E-07	6.64E-07
0.75	5.08E-08	2.80E-10	1.29E-09	1.07E-08	7.03E-08	2.25E-07
1.	2.28E-08	1.16E-10	4.01E-10	3.79E-09	3.01E-08	1.05E-07
1.5	7.05E-09	9.11E-11	1.13E-10	8.35E-10	8.23E-09	3.28E-08
3.	7.93E-10	4.01E-11	6.54E-11	9.79E-11	6.73E-10	3.42E-09
5.	1.30E-10	3.01E-11	4.01E-11	9.11E-11	1.23E-10	5.12E-10
7.5	2.68E-11	3.01E-11	4.01E-11	9.11E-11	9.11E-11	1.34E-10
10.	8.05E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11

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

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	2.24E-02	1.04E-02	1.51E-02	2.16E-02	2.96E-02	3.68E-02
0.001	1.27E-02	5.05E-03	8.00E-03	1.18E-02	1.74E-02	2.29E-02
0.005	2.18E-03	4.31E-04	8.98E-04	1.79E-03	3.47E-03	5.20E-03
0.01	7.42E-04	8.98E-05	2.13E-04	5.35E-04	1.27E-03	2.10E-03
0.015	3.49E-04	3.01E-05	7.66E-05	2.25E-04	6.17E-04	1.08E-03
0.03	7.74E-05	3.57E-06	9.93E-06	3.79E-05	1.38E-04	2.84E-04
0.05	2.17E-05	6.45E-07	1.82E-06	8.00E-06	3.73E-05	8.85E-05
0.075	7.35E-06	1.55E-07	4.37E-07	2.07E-06	1.15E-05	3.19E-05
0.1	3.31E-06	5.50E-08	1.57E-07	7.66E-07	4.77E-06	1.46E-05
0.15	1.05E-06	1.10E-08	3.63E-08	1.92E-07	1.34E-06	4.77E-06
0.3	1.47E-07	5.35E-10	2.39E-09	1.79E-08	1.51E-07	7.03E-07
0.5	3.51E-08	1.02E-10	2.96E-10	2.84E-09	3.01E-08	1.72E-07
0.75	1.13E-08	9.11E-11	9.79E-11	6.17E-10	8.00E-09	5.35E-08
1.	4.96E-09	4.56E-11	9.11E-11	2.22E-10	2.96E-09	2.25E-08
1.5	1.51E-09	4.01E-11	5.50E-11	9.11E-11	6.93E-10	6.26E-09
3.	1.65E-10	3.01E-11	4.01E-11	9.11E-11	9.79E-11	5.35E-10
5.	2.68E-11	3.01E-11	4.01E-11	9.11E-11	9.11E-11	1.16E-10
7.5	5.50E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11
10.	1.64E-12	3.01E-11	4.01E-11	9.11E-11	9.11E-11	9.11E-11

Table A-2 Medians and Logarithmic Sigmas of Amplification Factors for NMP

PGA (100 Hz)	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	1.04E+00	4.12E-02	1.30E-02	9.44E-01	4.44E-02	1.90E-02	1.02E+00	6.51E-02	2.09E-02	1.11E+00	9.02E-02
4.95E-02	9.23E-01	4.89E-02	1.02E-01	7.81E-01	7.40E-02	9.99E-02	9.98E-01	7.39E-02	8.24E-02	1.11E+00	9.16E-02
9.64E-02	8.80E-01	5.16E-02	2.13E-01	7.55E-01	8.11E-02	1.85E-01	9.93E-01	7.50E-02	1.44E-01	1.11E+00	9.15E-02
1.94E-01	8.48E-01	5.39E-02	4.43E-01	7.38E-01	8.54E-02	3.56E-01	9.85E-01	7.63E-02	2.65E-01	1.10E+00	9.15E-02
2.92E-01	8.31E-01	5.50E-02	6.76E-01	7.28E-01	8.73E-02	5.23E-01	9.79E-01	7.71E-02	3.84E-01	1.10E+00	9.16E-02
3.91E-01	8.20E-01	5.58E-02	9.09E-01	7.21E-01	8.86E-02	6.90E-01	9.74E-01	7.79E-02	5.02E-01	1.09E+00	9.18E-02
4.93E-01	8.12E-01	5.65E-02	1.15E+00	7.14E-01	8.97E-02	8.61E-01	9.70E-01	7.88E-02	6.22E-01	1.09E+00	9.19E-02
7.41E-01	7.97E-01	5.78E-02	1.73E+00	7.01E-01	9.17E-02	1.27E+00	9.61E-01	8.09E-02	9.13E-01	1.09E+00	9.23E-02
1.01E+00	7.86E-01	5.87E-02	2.36E+00	6.91E-01	9.33E-02	1.72E+00	9.52E-01	8.29E-02	1.22E+00	1.08E+00	9.24E-02
1.28E+00	7.78E-01	5.90E-02	3.01E+00	6.82E-01	9.40E-02	2.17E+00	9.44E-01	8.47E-02	1.54E+00	1.08E+00	9.22E-02
1.55E+00	7.72E-01	5.90E-02	3.63E+00	6.75E-01	9.43E-02	2.61E+00	9.38E-01	8.64E-02	1.85E+00	1.08E+00	9.19E-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	1.09E+00	7.72E-02	1.27E-02	1.23E+00	5.33E-02	8.25E-03	1.15E+00	6.17E-02
7.05E-02	1.08E+00	7.60E-02	3.43E-02	1.22E+00	5.19E-02	1.96E-02	1.15E+00	5.93E-02
1.18E-01	1.08E+00	7.54E-02	5.51E-02	1.22E+00	5.14E-02	3.02E-02	1.15E+00	5.86E-02
2.12E-01	1.08E+00	7.47E-02	9.63E-02	1.22E+00	5.12E-02	5.11E-02	1.14E+00	5.81E-02
3.04E-01	1.08E+00	7.42E-02	1.36E-01	1.22E+00	5.11E-02	7.10E-02	1.14E+00	5.78E-02
3.94E-01	1.08E+00	7.39E-02	1.75E-01	1.22E+00	5.12E-02	9.06E-02	1.14E+00	5.78E-02
4.86E-01	1.08E+00	7.36E-02	2.14E-01	1.22E+00	5.13E-02	1.10E-01	1.14E+00	5.77E-02
7.09E-01	1.08E+00	7.31E-02	3.10E-01	1.22E+00	5.16E-02	1.58E-01	1.15E+00	5.76E-02
9.47E-01	1.08E+00	7.29E-02	4.12E-01	1.22E+00	5.18E-02	2.09E-01	1.15E+00	5.77E-02
1.19E+00	1.08E+00	7.26E-02	5.18E-01	1.22E+00	5.21E-02	2.62E-01	1.15E+00	5.77E-02
1.43E+00	1.08E+00	7.24E-02	6.19E-01	1.22E+00	5.23E-02	3.12E-01	1.15E+00	5.77E-02

Tables A2-b1 and A2-b2 are tabular versions of the typical amplification factors provided in Figures 2.3.6-1 and 2.3.6-2. Values are provided for two input motion levels at approximately 10^{-4} and 10^{-5} mean annual frequency of exceedence. These factors are unverified and provided for information only. The figures should be considered the governing information.

Table A-2-b1 Median AFs and Sigmas for Model 1, for 2 PGA Levels

M1P1K1	Rock PGA=0.0964			M1P1K1	PGA=0.292		
Freq. (Hz)	PGA	med. AF	sigma ln(AF)	Freq. (Hz)	PGA	med. AF	sigma ln(AF)
100.0	0.081	0.842	0.062	100.0	0.224	0.765	0.071
87.1	0.082	0.831	0.063	87.1	0.226	0.750	0.073
75.9	0.083	0.812	0.064	75.9	0.229	0.724	0.075
66.1	0.084	0.773	0.067	66.1	0.235	0.675	0.079
57.5	0.088	0.707	0.073	57.5	0.246	0.599	0.088
50.1	0.094	0.642	0.086	50.1	0.268	0.539	0.106
43.7	0.103	0.597	0.106	43.7	0.298	0.506	0.132
38.0	0.112	0.588	0.127	38.0	0.330	0.512	0.152
33.1	0.121	0.592	0.129	33.1	0.357	0.527	0.152
28.8	0.131	0.630	0.119	28.8	0.384	0.570	0.139
25.1	0.141	0.662	0.112	25.1	0.411	0.609	0.128
21.9	0.152	0.739	0.116	21.9	0.442	0.691	0.129
19.1	0.161	0.782	0.104	19.1	0.465	0.741	0.116
16.6	0.167	0.829	0.113	16.6	0.475	0.792	0.119
14.5	0.177	0.910	0.109	14.5	0.497	0.872	0.117
12.6	0.178	0.930	0.112	12.6	0.495	0.895	0.121
11.0	0.187	0.991	0.097	11.0	0.513	0.956	0.104
9.5	0.179	0.985	0.092	9.5	0.492	0.964	0.094
8.3	0.166	0.980	0.092	8.3	0.451	0.959	0.094
7.2	0.171	1.072	0.090	7.2	0.458	1.045	0.092
6.3	0.178	1.176	0.092	6.3	0.472	1.148	0.092
5.5	0.169	1.161	0.101	5.5	0.448	1.144	0.103
4.8	0.152	1.058	0.092	4.8	0.401	1.049	0.093
4.2	0.144	1.027	0.101	4.2	0.377	1.020	0.102
3.6	0.141	1.027	0.094	3.6	0.367	1.021	0.094
3.2	0.140	1.081	0.067	3.2	0.363	1.077	0.067
2.8	0.133	1.080	0.073	2.8	0.344	1.077	0.072
2.4	0.127	1.110	0.054	2.4	0.326	1.109	0.053
2.1	0.121	1.160	0.053	2.1	0.310	1.159	0.052
1.8	0.107	1.138	0.070	1.8	0.271	1.138	0.069
1.6	0.100	1.223	0.102	1.6	0.252	1.222	0.102
1.4	0.096	1.352	0.073	1.4	0.239	1.349	0.073
1.2	0.083	1.333	0.047	1.2	0.207	1.331	0.047
1.0	0.069	1.213	0.044	1.0	0.170	1.212	0.044
0.91	0.058	1.125	0.055	0.91	0.143	1.125	0.055
0.79	0.052	1.095	0.061	0.79	0.126	1.095	0.060
0.69	0.047	1.106	0.050	0.69	0.113	1.106	0.050
0.60	0.042	1.138	0.039	0.60	0.101	1.137	0.038
0.52	0.037	1.171	0.048	0.52	0.088	1.169	0.047
0.46	0.032	1.190	0.067	0.46	0.075	1.188	0.065
0.10	0.001	1.059	0.020	0.10	0.003	1.054	0.020

Table A-2-b2 Median AFs and Sigmas for Model 2, for 2 PGA Levels

M2P1K1	PGA=0.0964			M2P1K1	PGA=0.292		
Freq. (Hz)	PGA	med. AF	sigma ln(AF)	Freq. (Hz)	PGA	med. AF	sigma ln(AF)
100.0	0.082	0.847	0.047	100.0	0.231	0.792	0.052
87.1	0.082	0.836	0.048	87.1	0.234	0.777	0.053
75.9	0.083	0.816	0.048	75.9	0.237	0.751	0.054
66.1	0.085	0.778	0.050	66.1	0.244	0.702	0.057
57.5	0.088	0.712	0.054	57.5	0.258	0.627	0.064
50.1	0.095	0.647	0.063	50.1	0.284	0.570	0.078
43.7	0.103	0.602	0.078	43.7	0.318	0.540	0.097
38.0	0.113	0.593	0.097	38.0	0.353	0.547	0.116
33.1	0.123	0.598	0.104	33.1	0.382	0.564	0.120
28.8	0.133	0.638	0.098	28.8	0.412	0.612	0.110
25.1	0.143	0.671	0.097	25.1	0.439	0.650	0.107
21.9	0.154	0.748	0.098	21.9	0.469	0.733	0.105
19.1	0.163	0.790	0.090	19.1	0.489	0.780	0.095
16.6	0.168	0.836	0.099	16.6	0.497	0.828	0.103
14.5	0.178	0.918	0.091	14.5	0.521	0.913	0.094
12.6	0.179	0.937	0.095	12.6	0.515	0.932	0.097
11.0	0.188	0.999	0.084	11.0	0.535	0.996	0.086
9.5	0.180	0.991	0.086	9.5	0.505	0.988	0.087
8.3	0.166	0.984	0.088	8.3	0.461	0.981	0.089
7.2	0.172	1.078	0.088	7.2	0.472	1.075	0.089
6.3	0.179	1.183	0.087	6.3	0.485	1.181	0.087
5.5	0.170	1.166	0.096	5.5	0.456	1.165	0.096
4.8	0.152	1.062	0.092	4.8	0.405	1.060	0.092
4.2	0.144	1.030	0.100	4.2	0.380	1.028	0.100
3.6	0.141	1.029	0.092	3.6	0.369	1.027	0.092
3.2	0.140	1.083	0.066	3.2	0.365	1.081	0.066
2.8	0.134	1.082	0.076	2.8	0.345	1.080	0.075
2.4	0.127	1.111	0.053	2.4	0.326	1.109	0.053
2.1	0.121	1.160	0.053	2.1	0.309	1.158	0.053
1.8	0.107	1.138	0.070	1.8	0.271	1.136	0.069
1.6	0.100	1.223	0.101	1.6	0.252	1.220	0.100
1.4	0.096	1.352	0.072	1.4	0.239	1.347	0.072
1.2	0.083	1.333	0.047	1.2	0.207	1.329	0.047
1.0	0.069	1.213	0.044	1.0	0.170	1.211	0.043
0.91	0.058	1.125	0.055	0.91	0.143	1.124	0.054
0.79	0.052	1.095	0.060	0.79	0.126	1.094	0.059
0.69	0.047	1.106	0.050	0.69	0.113	1.106	0.049
0.60	0.042	1.138	0.038	0.60	0.101	1.137	0.038
0.52	0.037	1.171	0.048	0.52	0.088	1.169	0.047
0.46	0.032	1.190	0.067	0.46	0.075	1.188	0.066
0.10	0.001	1.059	0.020	0.10	0.003	1.054	0.020

ATTACHMENT (4)

REGULATORY COMMITMENTS CONTAINED

IN THIS CORRESPONDENCE

ATTACHMENT (4)
REGULATORY COMMITMENTS CONTAINED IN THIS CORRESPONDENCE

The following table identifies actions committed to in this document for Nine Mile Point Nuclear Station, LLC (NMPNS), Units 1 (NMP1) and 2 (NMP2), R.E. Ginna Nuclear Power Plant, LLC (Ginna), and Calvert Cliffs Nuclear Power Plant, LLC (CCNPP), Units 1 and 2. Any other statements in this submittal are provided for information purposes and are not considered to be regulatory commitments.

Site	Regulatory Commitment	Date
CCNPP	CCNPP will submit an Expedited Seismic Evaluation Process (ESEP) report	12/31/2014
CCNPP	CCNPP will complete the Spent Fuel Pool Evaluation	In accordance with the NRC prioritization process
CCNPP	CCNPP will complete a full scope relay review	In accordance with the schedule provided in the letter from the industry to the NRC dated October 3, 2013
Ginna	Ginna will perform a High Frequency Confirmation	In accordance with the NRC prioritization process
NMP1 and NMP2	NMP1 and NMP2 will perform a High Frequency Confirmation	In accordance with the NRC prioritization process