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SEISMIC HAZARD AND SCREENING REPORT

IN RESPONSE TO THE 50.54(f) INFORMATION REQUEST REGARDING FUKISHIMA NEAR-TERM TASK FORCE RECOMMENDATION 2.1: SEISMIC

for

CATAWBA NUCLEAR STATION

DUKE ENERGY CAROLINAS

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1 Introduction

Following the accident at the Fukushima Dai-ichi 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 (Reference 1) that requests information to assure that these recommendations are addressed by all U.S. nuclear power plants. The 50.54(f) letter (Reference 1) requests that licensees and holders of construction permits under 10 CFR Part 50 (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 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 in Attachment 1 of the 50.54(f) letter (Reference 1) pertaining to NTTF Recommendation 2.1: Seismic for the Catawba Nuclear Station (Catawba), located in York County, South Carolina. In providing this information, Duke Energy Carolinas (Duke) 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* (Reference 3). The Augmented Approach, *Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic* (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.

The original geologic and seismic siting investigations for Catawba were performed in accordance with Appendix A to 10 CFR Part 100 (Reference 5) and meet General Design Criterion 2 in Appendix A to 10 CFR Part 50 (Reference 2). The Safe Shutdown Earthquake Ground Motion (SSE) was developed in accordance with Appendix A to 10 CFR Part 100 (Reference 5) and used for the design of seismic Category I structures, systems and components (SSC). (Reference 11, Sections 2.5, 3.1 and 3.2)

In response to the 50.54(f) letter (Reference 1) and following the guidance provided in the SPID (Reference 3), a seismic hazard reevaluation was performed. For screening purposes, a Ground Motion Response Spectrum (GMRS) was developed. The GMRS development and supporting seismic hazard analysis (Sections 2.2, 2.3 and 2.4 of this report) for Catawba was performed by the Electric Power Research Institute (EPRI) (Reference 9). Based on the results of the screening evaluation, Catawba screens in for a risk evaluation and a spent fuel pool integrity evaluation.

2

Seismic Hazard Reevaluation

Catawba is located in the north central portion of South Carolina approximately six miles north of Rock Hill and adjacent to Lake Wylie. The site is located in the Charlotte belt of the Piedmont (Reference 11, Section 2.1). The predominant rock type underlying the site is classified as adamellite and is fairly uniform in composition across the site. Mafic dikes constitute a subordinate rock type and are discontinuous and irregular across the site. There are no capable faults within 5 miles of the site or in the region surrounding the site. There is no geological evidence of (capable) surface faulting in the Piedmont, the tectonic province in which the site is located. Major Category I structures are supported by mat foundations which bear on rock or fill concrete to rock. The transition from partially weathered rock to the unweathered rock is somewhat gradational. The upper zones of the bedrock are variably weathered with many partially weathered rock zones between harder, less weathered rock layers. With increasing depth, the weathering decreases until moderately hard to hard continuous bedrock is encountered. (Reference 11, Section 2.5)

There have been no reported earthquakes within historic times with a Modified Mercalli (MM) intensity of more than VII in the Piedmont. The Charleston earthquake of August 31, 1886 produced surface intensities of only VI-VII MM at the site. Therefore, the SSE for the site is based on an earthquake producing surface intensity of VII-VIII occurring adjacent to the site. This is greater than the surface intensity of any earthquake within the Piedmont during historic time, and is greater than the surface intensity at the site from the Charleston earthquake of 1886. The peak ground acceleration (PGA) value for the SSE, chosen for foundations on closely jointed rock and slightly weathered rock, is 0.15g. This bedrock value relates very conservatively with the design surface intensity VII-VIII MM considering the maximum observed surface intensities of VII in the region and the overburden amplification that contributed to those maximum observed surface intensities. (Reference 11, Section 2.5)

2.1 Regional and Local Geology

The site is located in the Piedmont Province. The Piedmont Province is a deeply eroded, plateau-like segment of the Appalachian Mountain System. The Piedmont in this region is about 80 to 120 miles wide. The site is located in the Charlotte belt of the Piedmont. Rocks in this belt consist of a complex series of intrusive rocks, with some schist, quartzite, gneiss and amphibolite probably derived from sedimentary and volcanic deposits. With the exception of a few broad folds such as the anticline at Nanny Mountain, South Carolina and the Davie County Triassic fault basin, the structure of the Charlotte belt is a function of plutonic contacts. (Reference 11, Section 2.5.)

Catawba is located in the north central portion of South Carolina approximately six miles north of Rock Hill and adjacent to Lake Wylie (Reference 11, Section 2.1.). All major nuclear safety related structures are founded on rock or partially weathered rock except for localized portions of the Nuclear Service Water (NSW) pipe lines and the NSW conduit manholes, the Standby Nuclear Service Water Pond Outlet Works and the

Diesel Fuel Oil Tanks. The crystalline bedrock at this site is not subject to long-term deterioration or solution activity. The foundation rock for the nuclear safety related structures will not provide adverse response to seismic activity. Further, the residual soils and underlying crystalline bedrock are such that liquefaction is not a problem. It is concluded from the evidence presented in the Brecciated Zones Report that faulting on the site ended at least 56 million years ago and more likely 150 million years ago. There are no capable faults in the region surrounding the site, and there is no correlation between the locations of earthquake epicenters and regional tectonic structures. (Reference 11, Section 2.5)

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 6) together with the updated EPRI Ground-Motion Model (GMM) for the CEUS (Reference 7). For the PSHA, a lower-bound moment magnitude of 5.0 was used, as specified in the 50.54(f) letter (Reference 1).

For the PSHA, the CEUS-SSC background seismic sources out to a distance of 400 miles (640 km) around Catawba were included. This distance exceeds the 200 mile (320 km) recommendation contained in NRC Reg. Guide 1.208 (Reference 8) 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. Extended Continental Crust—Gulf Coast (ECC_GC)
4. Illinois Basin Extended Basement (IBEB)
5. Mesozoic and younger extended prior – narrow (MESE-N)
6. Mesozoic and younger extended prior – wide (MESE-W)
7. Midcontinent-Craton alternative A (MIDC_A)
8. Midcontinent-Craton alternative B (MIDC_B)
9. Midcontinent-Craton alternative C (MIDC_C)
10. Midcontinent-Craton alternative D (MIDC_D)
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. Reelfoot Rift (RR)
16. Reelfoot Rift including the Rough Creek Graben (RR-RCG)
17. Study region (STUDY_R)

For sources of large magnitude earthquakes, designated as Repeated Large Magnitude Earthquake (RLME) sources in CEUS-SSC (Reference 6), the following sources lie within 621 miles (1,000 km) of the site and were included in the analysis:

1. Charleston
2. Commerce
3. Eastern Rift Margin Fault northern segment (ERM-N)
4. Eastern Rift Margin Fault southern segment (ERM-S)
5. Marianna
6. New Madrid Fault System (NMFS)
7. Wabash Valley

For each of the above background and RLME sources, the mid-continent version of the updated CEUS EPRI GMM (Reference 7) 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 from NUREG/CR-6728 (Reference 17) has been used. Seismic hazard curves are shown below in Section 2.3.7 at the SSE control point elevation (discussed below in Section 3.2).

2.3 SITE RESPONSE EVALUATION

Following the guidance contained in Seismic Enclosure 1 of the 50.54(f) letter (Reference 1) and in the SPID (Reference 3) for nuclear power plant sites that are not founded on hard rock (considered as having a shear-wave velocity of at least 9285 fps (2.83 km/sec), or 9200 fps as approximated in the SPID (Reference 3)), a site response analysis was performed for Catawba.

2.3.1 *Description of Subsurface Material*

Catawba is located in the Piedmont Physiographic Province of South Carolina. The general site conditions consist of residual soils overlying partially weathered rock grading into hard metamorphic igneous rocks (Reference 10). As depth into partially weathered rock increases the degree of weathering decreases as continuous rock, defined as rock quality designation (RQD) of 75% or greater, is encountered.

Catawba consists of two units (1 and 2) with both reactor buildings supported on continuous rock. Table 2.3.1-1 and Table 2.3.1-2 show the geotechnical properties for Units 1 and 2 respectively.

Table 2.3.1-1 Summary of site geotechnical profile for Catawba Unit 1 (Reference 10)

Depth Range[1] (ft.)	Soil/Rock Description	Density (pcf)	Shear-Wave Velocity (fps)	Compressional Wave Velocity (fps)	Poisson's ratio
0-15	Very Stiff Sandy Silt, Dense Silty Sand	132	1393	2205	0.17
15-25	Very Dense Silty Sand to Partially Weathered Rock	127	1537	2624	0.07
25-35	Soft Adamellite - Partially Weathered Rock	138	1633	4052	0.40
35-45	Moderately Hard Adamellite – Weathered Rock	149	2228	1077	0.44
45-49.5	Moderately Hard Adamellite – Rock	159	2508	7490	0.44
49.5-63	Fill Concrete	140	6800	-	-
63-73	Hard Adamellite - Rock	170	5710	8616	0.11
73-83	Hard Adamellite – Rock	170	7002	13766	0.33
83-93	Hard Adamellite – Rock	170	8552	16832	0.33
93-103	Hard Adamellite – Rock	170	8868	17498	0.33
103-110	See Note 2	170	8868	17490	0.33
110+	See Note 2	170	9200	18264	0.33

Reference: UFSAR Figure 2-99 (Boring A-63) (Reference 11)

(1) Depth begins at Yard Grade Elevation 593.5 ft.
(2) Boring was terminated at 103 ft. below Yard Grade Elevation. Velocities beyond this depth are extrapolated, not confirmed by tests.
(3) The control point elevation is taken to be 49.5 ft. below the Yard Grade Elevation.

Table 2.3.1-2 Summary of site geotechnical profile for Catawba Unit 2 (Reference 10)

Depth Range[1] (ft.)	Soil/Rock Description	Density (pcf)	Shear-Wave Velocity (fps)	Compressional Wave Velocity (fps)	Poisson's ratio
0-8	Soft Adamellite – Partially Weathered Rock	138	1300	2048	0.16
8-18	Soft Adamellite – Partially Weathered Rock	146	1557	3163	0.34
18-28	Soft Adamellite – Partially Weathered Rock	160	1858	5188	0.43
28-38	Soft to Mod Hard Adamellite; Partially Weathered Rock to Weathered Rock	160	2313	7502	0.45
38-48	Moderately Hard Adamellite – Rock	168	3760	7335	0.32
48-49.5	Mod Hard to Hard Adamellite – Rock	169	6111	9302	0.12
49.5-61	Fill Concrete	140	6800	-	-
61-68	Hard Adamellite – Rock	169	7751	13197	0.24
68-78	Hard Adamellite – Rock	169	8199	13895	0.23
78-86	Hard Adamellite – Rock	169	8564	15755	0.29
86-102	See Note 2	169	8564	15755	0.29
102+	See Note 2	169	9200	17212	0.30

Reference: UFSAR Figure 2-98 (Boring A-61) (Reference 11)

(1) Depth begins at Yard Grade Elevation 593.5 ft.
(2) Boring was terminated at 86 ft. below Yard Grade Elevation. Velocities beyond this depth are extrapolated, not confirmed by tests.
(3) The control point elevation is taken to be 49.5 ft. below the Yard Grade Elevation.

The following description of the general geology at the site is taken directly from the AMEC Data for Site Amplifications (Reference 10):

> "The site is located in the Charlotte Belt, one of five northeast trending rock belts identified within the Piedmont Physiographic Province at the time the PSAR was prepared. Rocks in this belt consist of a complex series of intrusive rocks, with some schist, quartzite, gneiss and amphibolites probably derived from sedimentary and volcanic deposits. Metamorphic rocks are mainly in the amphibolite facies. The most common intrusive rocks range in composition from granite to gabbro and some of the granitic bodies are of batholithic dimensions. It is mainly the extensive complex of intrusive rocks which distinguishes the Charlotte belt from the adjacent belts."
>
> "The bedrock at the site consists primarily of adamellite which is a metamorphosed igneous rock of the Charlotte belt. The adamellite is a medium grained crystalline rock with faint foliation and uniform texture and mineralogy. The bedrock also includes a secondary rock type in the form of discontinuous and irregular mafic dikes within the adamellite. The mafic dikes are fine grained rocks consisting of predominantly dark colored minerals."

2.3.2 Development of Base Case Profiles and Nonlinear Material Properties

Table 2.3.2-1 shows the recommended shear-wave velocities and unit weights versus depth for the best estimate single profile accommodating profiles for Unit 1 and Unit 2 (as conveyed in Table 2.3.1-1 and Table 2.3.1-2, respectively). In Table 2.3.2-1, depths begin at elevation 593.5 ft., and the Deepest Foundation Elevation (SSE control point) was taken at Elevation 544 ft. Elevation 544 ft. reflects the top of the fill concrete and the base of the mat foundation of the reactor buildings. Based on Table 2.3.2-1 and the adopted location of the SSE control point at a depth of 49.5 ft. (15.1 m), the profile consists of 60.5 ft. (18.4 m) of firm rock (including fill concrete) overlying hard metamorphic basement rock.

Shear-wave velocities for the materials below the fill concrete to a depth of 103 ft. (31.4 m) were based on downhole measurements (Reference 10). The shear-wave velocity for concrete was estimated, based on unit weight, unconfined compressive strength, and assumed Poisson ratio (Reference 10). For the material below a depth of 103 ft. (34.4 m), shear-wave velocities were based on extrapolations of measurements made in the "continuous rock" with the recommended profile reaching hard reference rock conditions at a depth of 110 ft.

Table 2.3.2-1 Summary of site geotechnical profile (average) for Catawba Units 1 and 2 (Reference 10)

Depth Range(1) (ft.)	Soil/Rock Description	Density (pcf)	Shear-Wave Velocity (fps)
0-7.5	See Table 2.3.1-1 and Table 2.3.1-2 for Descriptions	135	1347
7.5-16		139	1475
16-26		143	1698
26-36		149	1973
36-46		158	2994
46-49.5		164	4310
49.5-61.5		140	6800
61.5-63		169	5723
63-75		170	6955
75-86		170	7783
86-93		170	8552
93-103		170	8868
103-110	See Note 2	170	8868
110+	See Note 2	170	9200

(1) Depth begins at Yard Grade Elevation 593.5 ft.
(2) Boring A-61 was terminated at 86 ft. below Yard Grade Elevation. Values for 86-93 and 93-103 are from A-63. Boring A-63 was terminated at 103 ft. below Yard Grade Elevation. Velocities at depths greater than 103 ft. are extrapolated, not confirmed by tests.
(3) The control point elevation is taken to be 49.5 ft. below the Yard Grade Elevation.

Based on the specified shear-wave velocities reflecting a mixture of predominately measured values as well as assumed values, and considering the recommended shear-wave velocities follow the expected trend of increasing with depth (except for fill concrete), a scale 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 3) 10^{th} and 90^{th} fractiles, which implies a 1.28 scale factor on σ_{μ}.

Using the shear-wave velocities specified in Table 2.3.2-1, 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-case profiles P2 and P3 respectively. Profiles P1 and P2 have a mean depth below the SSE control point at elevation 544 ft. of 60.5 ft. (18.4 m) to hard reference rock, randomized ± 12 ft. (± 3.7 m). Profile P3 has a mean depth below the SSE control point at elevation 544 ft. of 25.5 ft. (7.8 m) to hard reference rock, with layers randomized as described in Section 2.3.3. The base-case profiles (P1, P2, and P3) are shown in Figure 2.3.2-1 and listed in Table 2.3.2-2. The depth randomization reflects ± 20% 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.

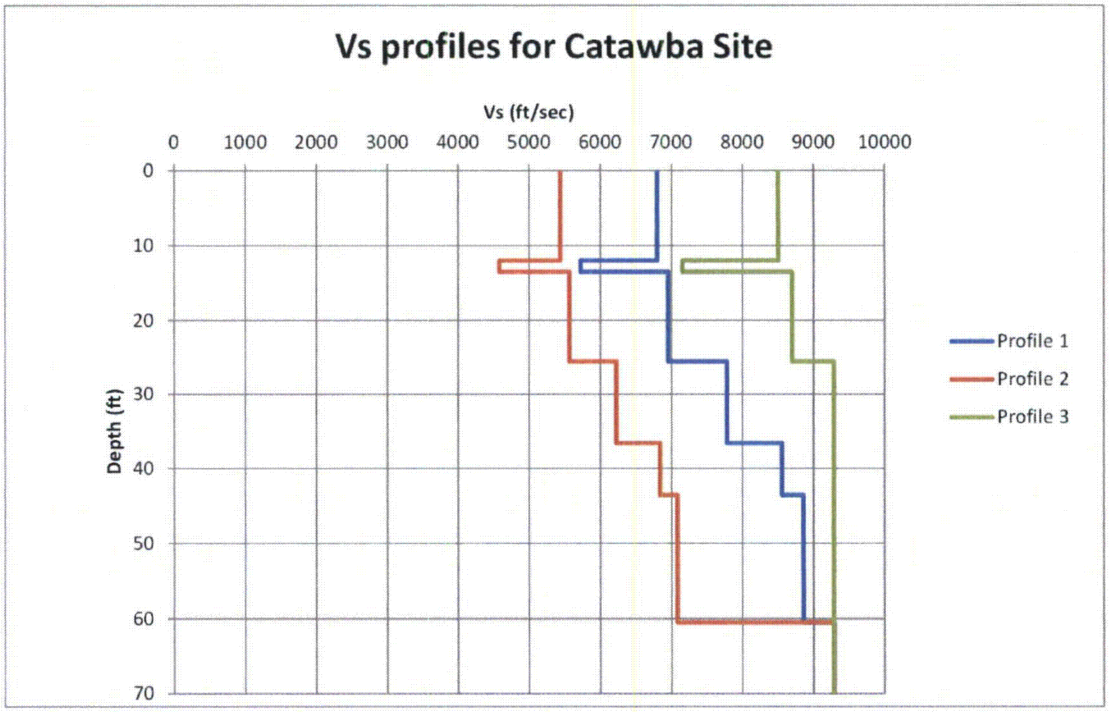

Figure 2.3.2-1 Shear-wave velocity profiles for the Catawba site

Table 2.3.2-2 Layer thicknesses, depths, and shear-wave velocities (V_s) for three profiles, the Catawba 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	6800		0	5440		0	8500
4.0	4.0	6800	4.0	4.0	5440	4.0	4.0	8500
4.0	8.0	6800	4.0	8.0	5440	4.0	8.0	8500
4.0	12.0	6800	4.0	12.0	5440	4.0	12.0	8500
1.5	13.5	5723	1.5	13.5	4578	1.5	13.5	7153
4.0	17.5	6955	4.0	17.5	5564	4.0	17.5	8693
4.0	21.5	6955	4.0	21.5	5564	4.0	21.5	8693
4.0	25.5	6955	4.0	25.5	5564	4.0	25.5	8693
3.7	29.2	7783	3.7	29.2	6226	3.7	29.2	9285
3.7	32.9	7783	3.7	32.9	6226	3.7	32.9	9285
3.7	36.5	7783	3.7	36.5	6226	3.7	36.5	9285
3.5	40.0	8552	3.5	40.0	6841	3.5	40.0	9285
3.5	43.5	8552	3.5	43.5	6841	3.5	43.5	9285
3.3	46.9	8854	3.3	46.9	7083	3.3	46.9	9285
3.3	50.2	8854	3.3	50.2	7083	3.3	50.2	9285
3.3	53.5	8854	3.3	53.5	7083	3.3	53.5	9285
3.5	57.0	8854	3.5	57.0	7083	3.5	57.0	9285
3.5	60.5	8854	3.5	60.5	7083	3.5	60.5	9285
3280.8	3365.7	9285	3280.8	3365.7	9285	3280.8	3365.7	9285

2.3.2.1 Shear Modulus and Damping Curves

No site-specific nonlinear dynamic material properties were determined for the firm rock materials in the initial siting of Catawba. The rock material over the upper 60.5 ft. (18.4 m) was assumed to have behavior that could be modeled as either linear or nonlinear. To represent this potential for either case in the upper 60.5 ft. of firm rock at the Catawba 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) was assumed to represent an equally plausible alternative rock response across loading level. For the linear analyses, the low strain damping from the EPRI rock curves were used as the constant damping values in the upper 60.5 ft.

2.3.2.2 Kappa

For the Catawba site profile of about 60.5 ft. (18.4 m) of firm rock over hard reference rock, the kappa value of 0.006s for hard rock (Reference 3) dominates profile damping. The 60.5 ft. of firm rock, based on the low strain damping from the EPRI rock G/G_{max} and hysteretic damping curves, reflects a contribution of only about 0.0006s (Table 2.3.2-3). As a result, the dominant epistemic uncertainty in low strain kappa was assumed to be incorporated in the reference rock hazard.

Table 2.3.2-3 Kappa values and weights used for site response analyses

Velocity Profile	Kappa (s)	Weights
P1	0.0065	0.4
P2	0.0066	0.3
P3	0.0064	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 Catawba site, random shear-wave velocity profiles were developed from the base case profiles shown in Figure 2.3.2-1. Thirty random velocity profiles were generated for each base case profile. These random velocity profiles were generated using a natural log standard deviation of 0.25 over the upper 50 ft. and a natural log standard deviation of 0.15 below that depth. As specified in the SPID (Reference 3), 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 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 (median PGA ranging from 0.01g to 1.5g) was used in the site response analyses. The characteristics of the seismic source and upper crustal attenuation properties assumed for the analysis of the Catawba 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 Catawba 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 3). The guidance contained in Appendix B of the SPID (Reference 3) on incorporating epistemic uncertainty in shear-wave velocities, kappa, nonlinear dynamic properties and source spectra for plants with limited at-site information was followed for the Catawba 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 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 EPRI rock G/G_{max} and hysteretic damping curves (model M1). 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 Catawba firm rock site, Figure 2.3.6-2 shows the corresponding amplification factors developed with linear site response analyses (model M2). Between the linear and nonlinear (equivalent-linear) analyses, Figure 2.3.6-1 and Figure 2.3.6-2 show only a minor difference across structural frequency as well as loading level. Tabulated values of the amplification factors are provided in Appendix A.

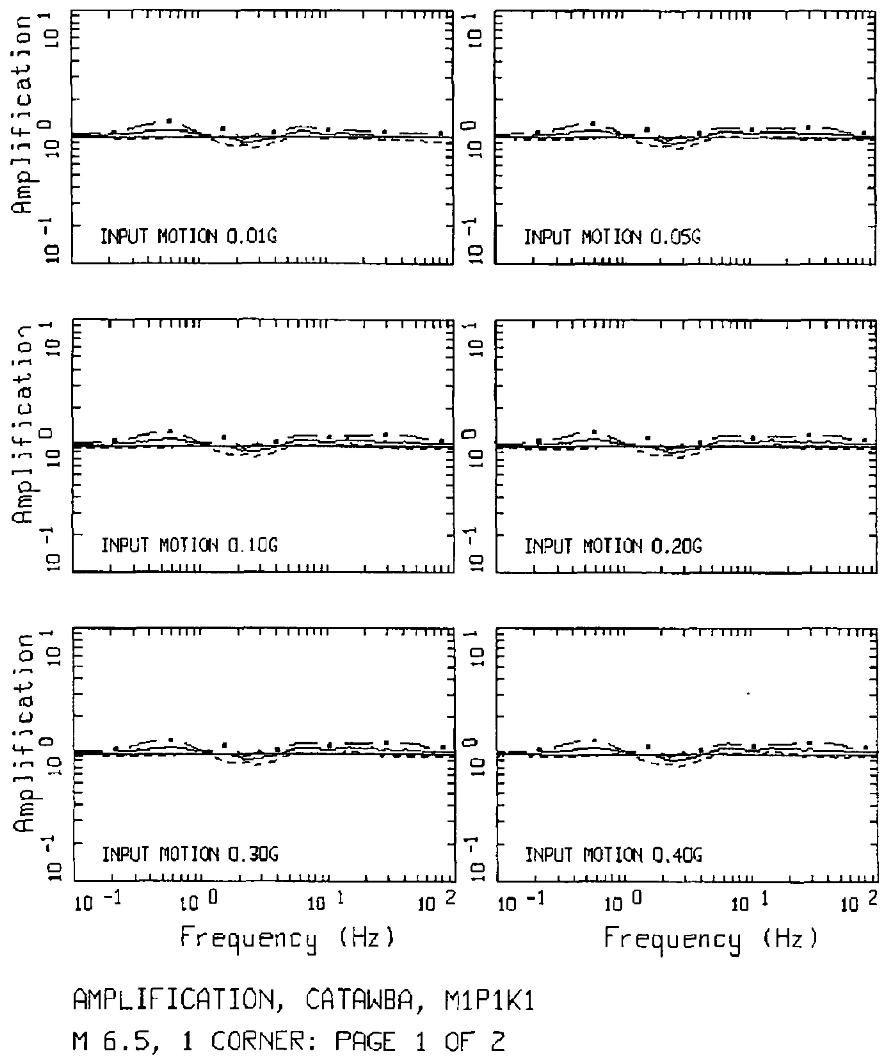

Figure 2.3.6-1 Example suite of amplification factors (5% of critical damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), EPRI rock modulus reduction and hysteretic damping curves (model M1), and base-case kappa (K1) at eleven loading levels of hard rock median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

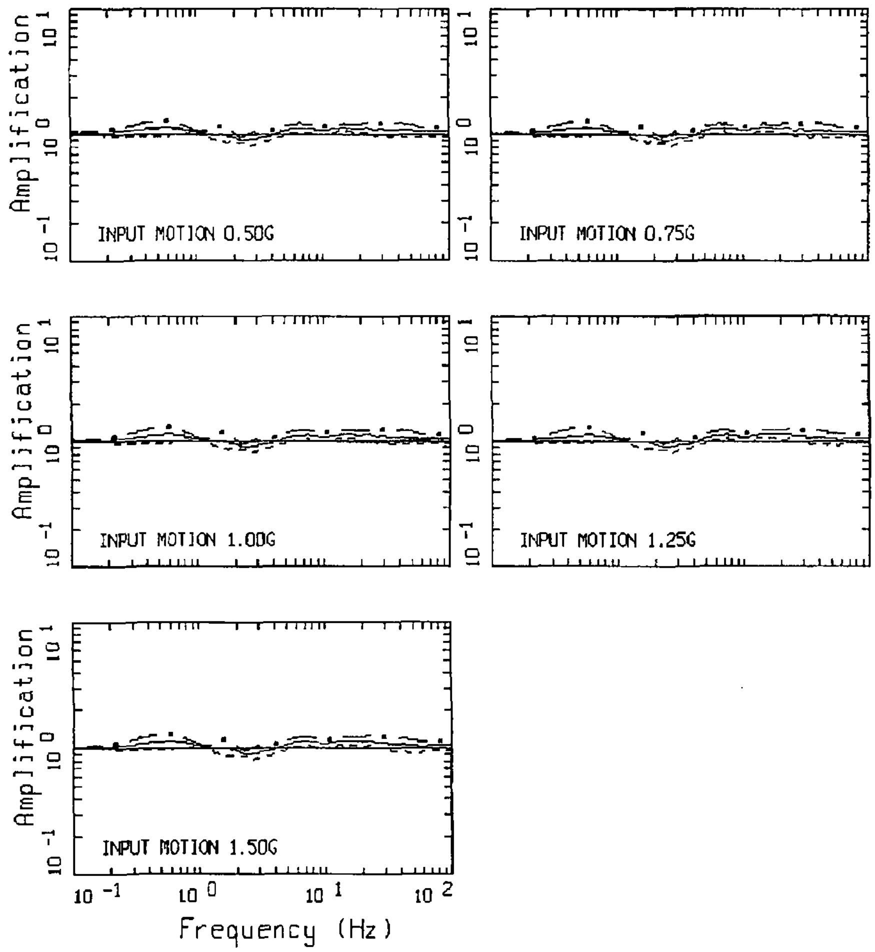

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Figure 2.3.6-1 continued

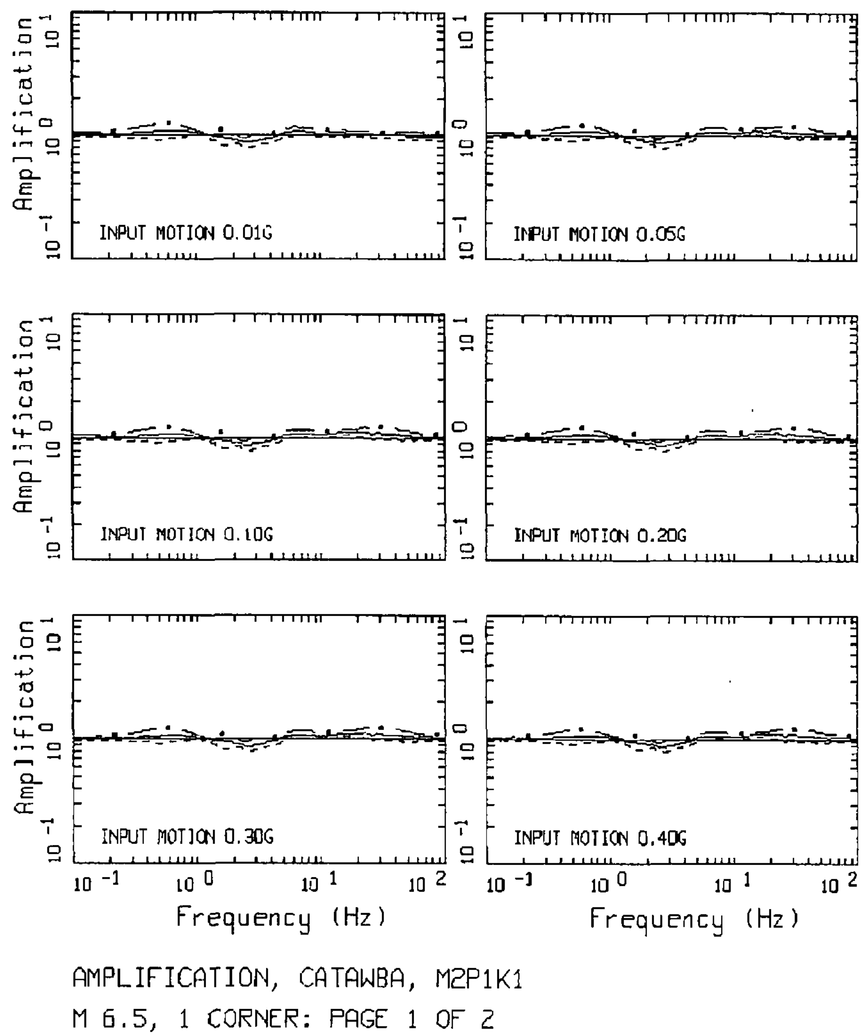

Figure 2.3.6-2 Example suite of amplification factors (5% of critical damping pseudo absolute acceleration spectra) developed for the mean base-case profile (P1), linear site response (model M2), and base-case kappa (K1) at eleven loading levels of hard rock median peak acceleration values from 0.01g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

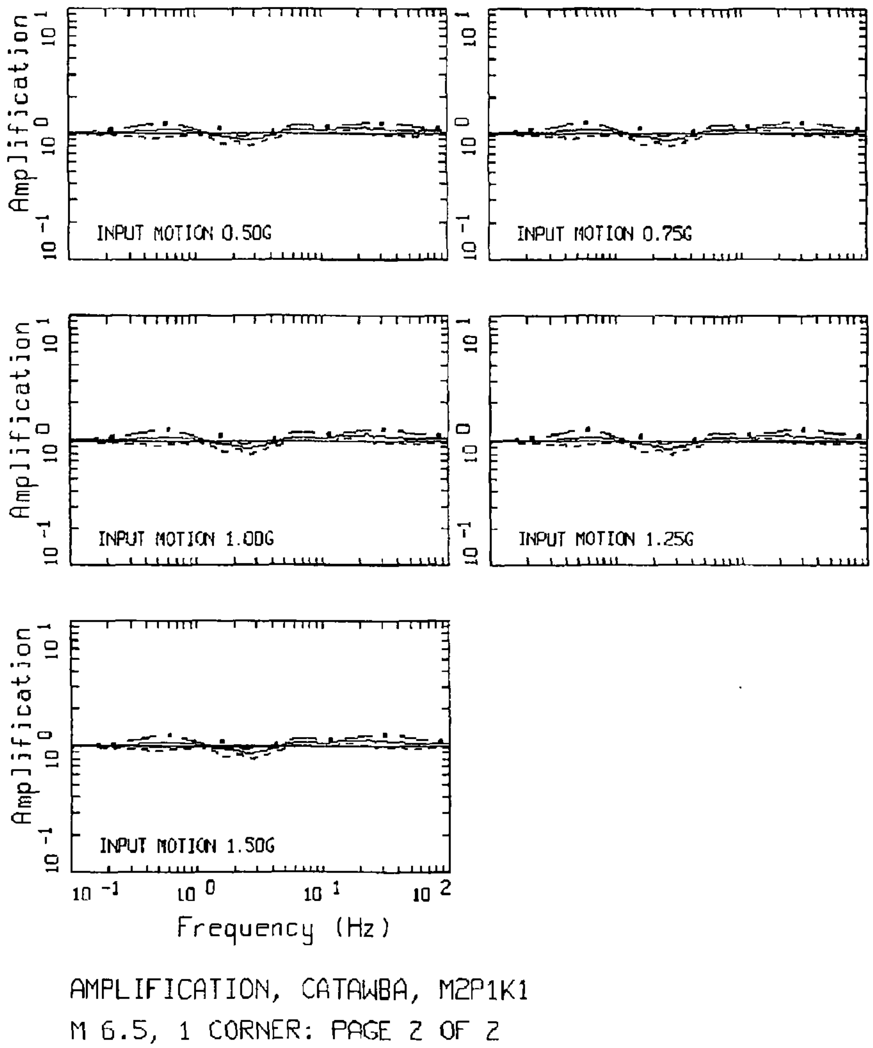

Figure 2.3.6-2 continued

2.3.7 *Control Point Seismic Hazard Curves*

The procedure to develop probabilistic site-specific control point hazard curves used in the present analysis follows the methodology described in Section B-6.0 of the SPID (Reference 3). This procedure (referred to as Method 3 from NUREG/CR-6728 (Reference 17)) 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 Catawba 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.

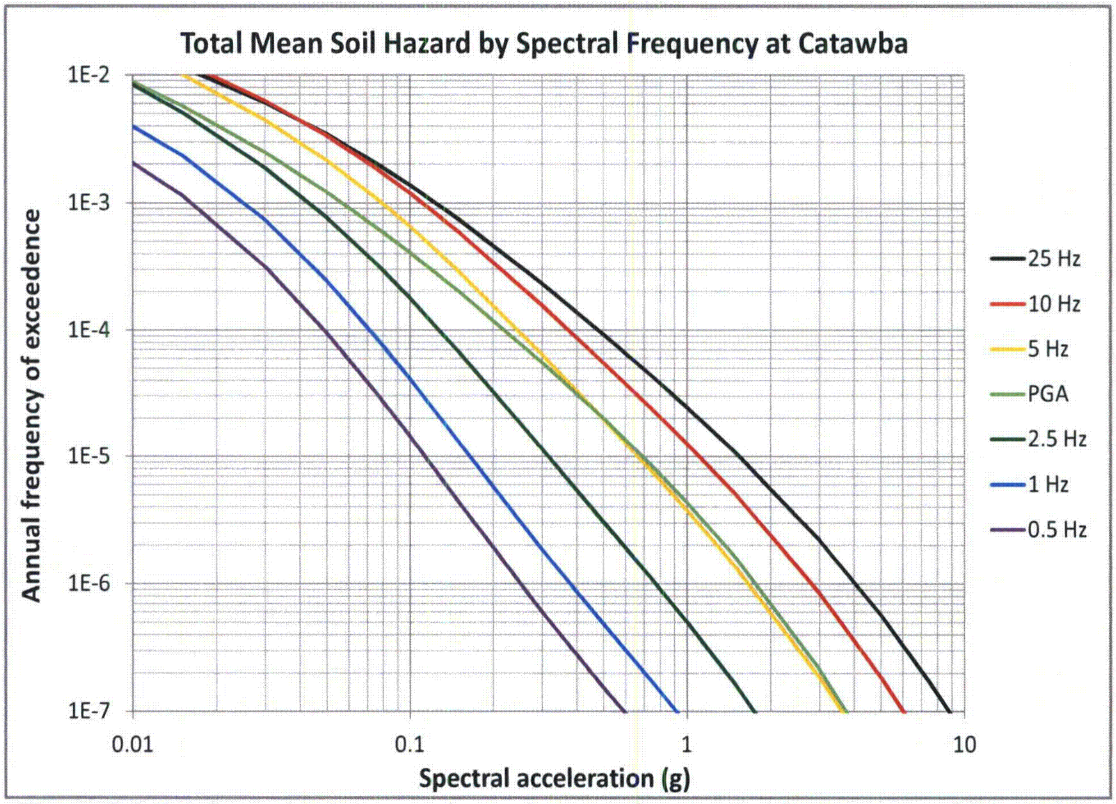

Figure 2.3.7-1 Control point mean hazard curves for spectral frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz (PGA) at Catawba (5% of critical damping)

2.4 CONTROL POINT RESPONSE SPECTRA

The control point mean hazard curves described above have been used to develop uniform hazard response spectra (UHRS) and the 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 NRC Reg. Guide 1.208 (Reference 8). Figure 2.4-1 shows the control point UHRS and GMRS. Table 2.4-1 shows the UHRS and GMRS spectral accelerations for each of the seven frequencies.

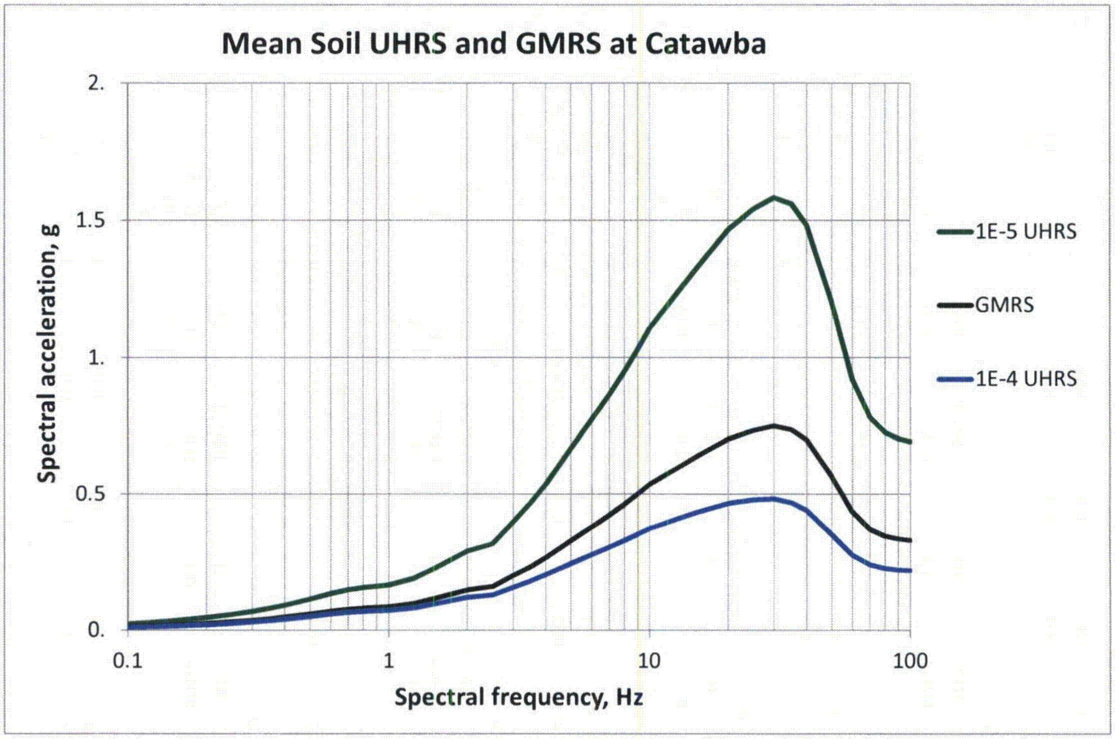

Figure 2.4-1 Plots of 1E-4 and 1E-5 uniform hazard spectra and GMRS at control point for Catawba (5% of critical damping response spectra)

Table 2.4-1 UHRS and GMRS at control point for Catawba (5% of critical damping response spectra)

Freq (Hz)	1E-4 UHRS (g)	1E-5 UHRS (g)	GMRS (g)
100	2.19E-01	6.91E-01	3.29E-01
90	2.21E-01	7.02E-01	3.34E-01
80	2.26E-01	7.25E-01	3.45E-01
70	2.40E-01	7.81E-01	3.70E-01
60	2.75E-01	9.19E-01	4.33E-01
50	3.51E-01	1.20E+00	5.63E-01
40	4.39E-01	1.48E+00	6.98E-01
35	4.67E-01	1.56E+00	7.35E-01
30	4.82E-01	1.58E+00	7.48E-01
25	4.79E-01	1.54E+00	7.31E-01
20	4.66E-01	1.47E+00	6.99E-01
15	4.31E-01	1.32E+00	6.33E-01
12.5	4.06E-01	1.22E+00	5.89E-01
10	3.74E-01	1.11E+00	5.35E-01
9	3.52E-01	1.03E+00	4.98E-01
8	3.29E-01	9.49E-01	4.61E-01
7	3.05E-01	8.63E-01	4.21E-01
6	2.77E-01	7.72E-01	3.77E-01
5	2.45E-01	6.67E-01	3.28E-01
4	2.03E-01	5.36E-01	2.65E-01
3.5	1.80E-01	4.67E-01	2.31E-01
3	1.56E-01	3.97E-01	1.98E-01
2.5	1.27E-01	3.16E-01	1.58E-01
2	1.19E-01	2.90E-01	1.45E-01
1.5	9.49E-02	2.26E-01	1.14E-01
1.25	8.03E-02	1.89E-01	9.55E-02
1	7.15E-02	1.64E-01	8.35E-02
0.9	6.96E-02	1.60E-01	8.14E-02
0.8	6.73E-02	1.55E-01	7.87E-02
0.7	6.36E-02	1.47E-01	7.44E-02
0.6	5.76E-02	1.33E-01	6.74E-02
0.5	4.90E-02	1.13E-01	5.74E-02
0.4	3.92E-02	9.04E-02	4.59E-02
0.35	3.43E-02	7.91E-02	4.02E-02
0.3	2.94E-02	6.78E-02	3.44E-02
0.25	2.45E-02	5.65E-02	2.87E-02
0.2	1.96E-02	4.52E-02	2.29E-02
0.15	1.47E-02	3.39E-02	1.72E-02
0.125	1.22E-02	2.83E-02	1.43E-02
0.1	9.79E-03	2.26E-02	1.15E-02

3

Plant Design Basis Ground Motion

The maximum earthquake intensity at the Catawba site is based upon the greatest earthquake intensity experienced at the site due to the largest earthquake in the tectonic province of the site and surrounding provinces occurring at the point of closest approach. Based on this review, the set of conditions describing the largest vibratory ground motion at the site would be an earthquake occurring in the immediate vicinity of the site and producing the historic maximum intensity VII for the Piedmont tectonic province. Therefore, the SSE for the site is based on an earthquake producing surface intensity of VII-VIII MM occurring adjacent to the site. This is greater than the surface intensity of any earthquake within the Piedmont during historic time, and is greater than the surface intensity at the site from the Charleston earthquake of 1886. (Reference 11, Section 2.5)

3.1 SSE Description of Spectral Shape

The Catawba SSE is defined in terms of a PGA and a design response spectrum shape. The design surface intensity of VII-VIII MM very conservatively relates to a PGA value of 0.15g for the SSE, chosen for foundations on closely jointed rock and slightly weathered rock. The Catawba design response spectrum for the SSE has a Newmark-type spectral shape. (Reference 11, Section 2.5)

For the purposes of NTTF 2.1: Seismic screening, the spectral acceleration values for the Catawba horizontal SSE (5% of critical damping) are shown as a function of frequency in Table 3.1-1 and plotted in Figure 3.1-1. The SSE acceleration values are based upon a Newmark-type spectrum derived from Figure 2-112 of the Catawba Updated Final Safety Analysis Report (UFSAR) (Reference 11).

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

Frequency (Hz)	Spectral Acceleration (g)
0.33	0.06
2	0.36
6	0.36
35/PGA	0.15

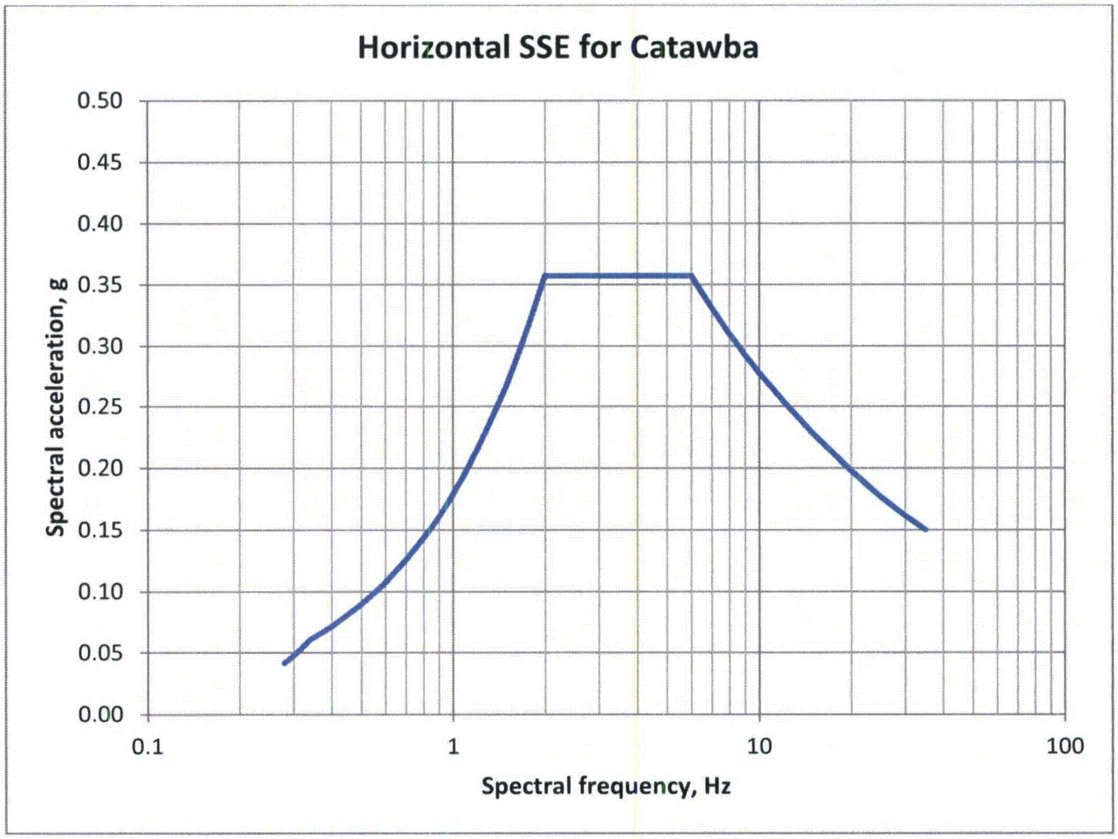

Figure 3.1-1 Horizontal SSE for Catawba (5% of critical damping response spectrum)

3.2 Control Point Elevation

The Catawba UFSAR defines the SSE control point at the top of sound rock (Reference 11, Section 3.7). The top of "continuous" rock has a variation of approximately 100 feet in depth across the site. The irregularity of the rock surface is the result of a differential weathering process common in the Piedmont. All major Category I Powerhouse structures are supported on rock. At a few locations, the top of continuous rock is below the bottom of the substructure mat of significant structures. At those locations, fill concrete is placed to extend from the top of continuous rock up to foundation grade (Reference 11, Section 2.5). Since the top of continuous rock varies across the site, the control point elevation is taken to be at El. 544 ft., which is at the base of the reactor building mat foundations. 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 (Reference 3), a screening evaluation was performed for Catawba as described below.

4.1 Risk Evaluation Screening (1 to 10 Hz)

In the 1 to 10 Hz part of the response spectrum, the GMRS exceeds the SSE for Catawba. Therefore, Catawba screens in for a risk evaluation.

4.2 High Frequency Screening (> 10 Hz)

Above 10 Hz, the GMRS exceeds the SSE for Catawba. The high frequency exceedances can be addressed in the risk evaluation discussed in Section 4.1 above.

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 for Catawba. Therefore, Catawba screens in for a spent fuel pool integrity evaluation.

5 Interim Actions and Assessments

As described in Section 4, the GMRS developed in response to the NTTF 2.1: Seismic portion of the 10 CFR 50.54(f) Request for Information dated March 12, 2012 (Reference 1) exceeds the design basis SSE. The NRC 50.54(f) letter (Reference 1) requests: "interim evaluation 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." These evaluations and actions are discussed below.

Consistent with NRC letter dated February 20, 2014 (Reference 18), the seismic hazard reevaluations presented herein are distinct from the current design and licensing bases of Catawba. 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" (Reference 2, Section 50.72) and 10 CFR 50.73, "Licensee event report system" (Reference 2, Section 50.73).

5.1 Expedited Seismic Evaluation Program

An expedited seismic evaluation process (ESEP) is being performed at Catawba in accordance with the methodology in EPRI 3002000704 (Reference 4) as proposed in a letter to the NRC dated April 9, 2013 (Reference 13) and agreed to by the NRC in a letter dated May 7, 2013 (Reference 14). Duke plans to submit a report on the ESEP to the NRC in December 2014 (Reference 25), in accordance with the schedule in the Nuclear Energy Institute (NEI) April 9, 2013 letter to the NRC (Reference 13).

5.2 Seismic Risk Estimates

The NRC letter (Reference 18) also requests that licensees provide an interim evaluation or actions to address the higher seismic hazard relative to the design basis while the expedited approach and risk evaluations are conducted. In response to that request, NEI letter dated March 12, 2014 (Reference 12) provides seismic core damage risk estimates using the updated seismic hazards for the operating nuclear plants in the CEUS. These risk estimates continue to support the following conclusions of the NRC GI-199 Safety/Risk Assessment (Reference 15):

> "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."

Catawba is included in the March 12, 2014 risk estimates (Reference 12). Using the methodology described in the NEI letter (Reference 12), the seismic core damage risk estimates for all plants were shown to be below 1E-4/year; thus, the above conclusions apply.

5.3 INDIVIDUAL PLANT EXAMINATION OF EXTERNAL EVENTS

An evaluation of beyond-design-basis ground motions was performed for Catawba as part of the IPEEE program. The SPRA methodology was utilized to perform the IPEEE seismic evaluation for Catawba (Reference 23). The results of the SPRA determined the seismic core damage frequency (SCDF) for Catawba to be less than the Commission's Safety Goal subsidiary objective of 1E-4/year (References 22 and 15). The Catawba IPEEE seismic evaluation concluded that there are no fundamental weaknesses or vulnerabilities with regard to severe accident risk, including seismic (Reference 22), and confirmed that the plant poses no undue risk to the public health and safety (Reference 23). Additionally, improvements were made to the plant based on the Catawba IPEEE seismic evaluation, as confirmed in the NTTF 2.3 seismic walkdown reports, to enhance the Catawba seismic margin (References 20 and 21).

Catawba performed an SMA as part of a trial assessment of EPRI's seismic assessment methodology. A review of Catawba, Unit 2, was conducted for a hypothetical Seismic Margin Earthquake (SME), applying the procedures and criteria developed for reassessment of nuclear power plant seismic margin. The SME selected was an 84% non-exceedance site specific response spectrum scaled to 0.3g PGA using the response spectrum developed for the Sequoyah Nuclear Power Plant. The application of the seismic margin criteria to Catawba revealed that the structures and equipment are capable of surviving the SME and that the high-confidence-of-low-probability-of-failure (HCLPF) value exceeds 0.3g PGA. There were 15 relays for which operability HCLPFs exceeding the review level earthquake could not be fully demonstrated at the time of the SMA; however, the SMA report concluded that further work on relay chatter would very likely result in predicted HCLPF values greater than the SME. Although Unit 1 was not specifically included in the study, the units are virtually identical and the conclusions reached on Unit 2 are believed applicable to Unit 1 as well. (Reference 24)

In the frequency range of 1 to 10 Hz, the Catawba SME bounds the GMRS. The Catawba SME is provided for context of demonstrating beyond-design-basis seismic margin capacity; however, the SME is not used for the NTTF 2.1: Seismic screening evaluation. The horizontal SME (5% of critical damping), based on Reference 26, is shown below in Table 5.3-1 and plotted in Figure 5.3-1.

Table 5.3-1 Horizontal SME for Catawba (5% of critical damping response spectrum)

Frequency (Hz)	Spectral Acceleration (g)
0.25	0.020
0.28	0.027
0.31	0.033
0.35	0.041
0.39	0.048
0.44	0.055
0.49	0.063
0.55	0.077
0.62	0.093
0.69	0.110
0.78	0.128
0.87	0.148
0.97	0.167
1.09	0.184
1.22	0.201
1.37	0.220
1.53	0.251
1.71	0.301
1.92	0.374
2.15	0.470
2.4	0.546
2.69	0.589
3.01	0.635
3.38	0.689
3.78	0.729
4.23	0.752
4.74	0.772
5.31	0.792
5.94	0.817
6.66	0.839
7.45	0.806
8.35	0.727
9.35	0.644
10.47	0.566
11.72	0.511
13.13	0.473
14.7	0.445
16.46	0.424
18.43	0.410
20.64	0.398
23.11	0.382
25.88	0.363
28.98	0.341
32.46	0.320
36.34	0.308
40.7	0.301
46	0.300

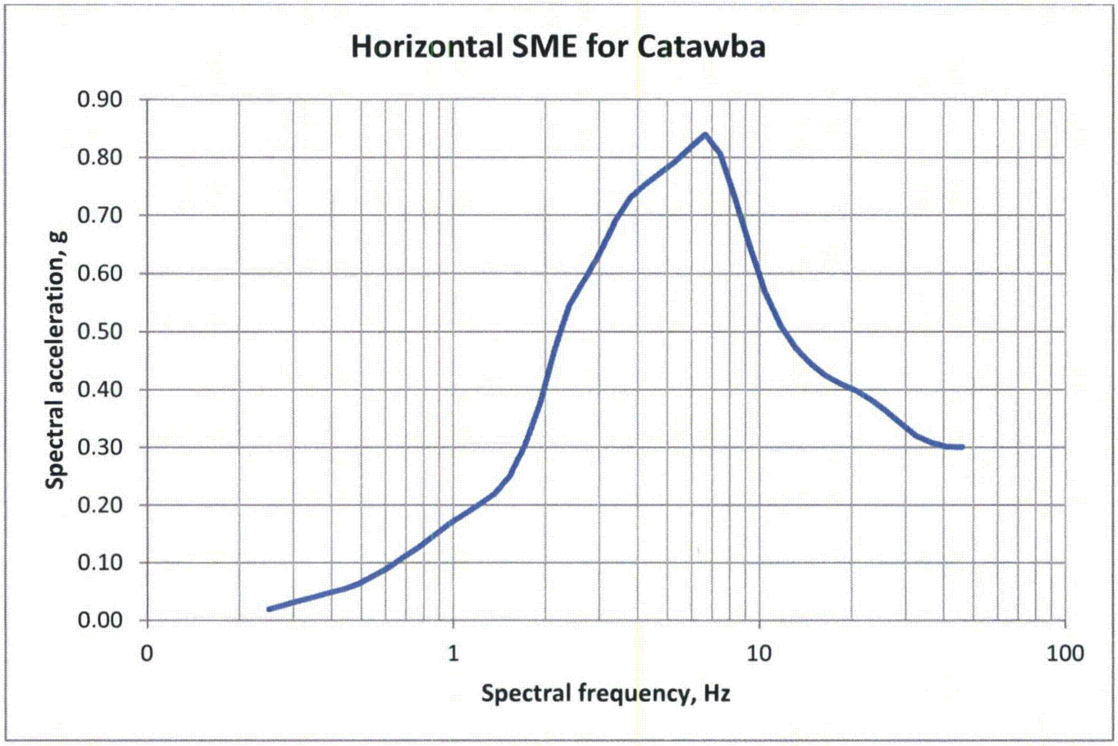

Figure 5.3-1 Horizontal SME for Catawba (5% of critical damping response spectrum)

5.4 WALKDOWNS TO ADDRESS NRC FUKUSHIMA NTTF RECOMMENDATION 2.3

Walkdowns have been completed for Catawba in accordance with the EPRI seismic walkdown guidance (Reference 19); including inaccessible items (References 16, 20 and 21). Potentially adverse seismic conditions (PASC) found were entered into the corrective action program (CAP) for resolution. None of the PASC items challenged operability of the plant. There were no vulnerabilities identified under IPEEE, however, previously identified IPEEE enhancements were reviewed and found to be complete. Duke confirmed through the walkdowns that the existing monitoring and maintenance procedures keep the plant consistent with the design basis. (References 20 and 21)

6 Conclusions

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

Based on the results of the screening evaluation, Catawba screens in for a risk evaluation and a spent fuel pool integrity evaluation.

7 References

1. NRC (E. Leeds and M. Johnson) Letter to All Power Reactor Licensees et al., *Request for Information Pursuant to Title 10 of the Code of Federal Regulations 50.54(f) Regarding Recommendations 2.1, 2.3, and 9.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* dated March 12, 2012, ADAMS Accession No. ML12053A340.

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

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

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

9. EPRI RSM-092513-029, *Catawba Seismic Hazard and Screening Report*, dated October 31, 2013.

10. AMEC Project No. 6234-12-0031, *Data for Site Amplifications – Catawba Phase 2 EPRI Seismic Attenuation and GMRS Project, Catawba Nuclear Station,* July 26, 2012.

11. Duke Energy Company, *Catawba Nuclear Station, Updated Final Safety Analysis Report*, Revision 16.

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

13. NEI (A. R. Pietrangelo) Letter to the NRC, *Proposed Path Forward for NTTF Recommendation 2.1: Seismic Reevaluations,* dated April 9, 2013, ADAMS Accession No. ML13101A379.

14. NRC (E. Leeds) Letter to NEI (J. Pollock), *Electric Power Research Institute Final Draft Report XXXXXX, "Seismic Evaluation Guidance: Augmented Approach for the Resolution of Fukushima Near-Term Task Force Recommendation 2.1: Seismic," as an Acceptable Alternative to the March 12, 2012, Information Request for Seismic Reevaluations*, dated May 7, 2013, ADAMS Accession No. ML13106A331.

15. NRC Memorandum (from P. Hiland to B. Sheron), "Safety/Risk Assessment Results for Generic Issue 199, Implications of Updated Probabilistic Seismic Hazard Estimates in Central and Eastern United States on Existing Plants," dated September 2, 2010, ADAMS Accession No. ML100270582.

16. Duke Energy Carolinas Letter to the NRC, *Response to Request for Additional Information Regarding the Seismic Hazard Walkdowns Associated With Near-Term Task Force Recommendation 2.3, Seismic Walkdowns,* dated December 2, 2013, ADAMS Accession No. ML13338A280.

17. NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-Consistent Ground Motion Spectra Guidelines,* October 2001.

18. NRC (E. Leeds) Letter to All Power Reactor Licensees et al., *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, ADAMS Accession No. ML14030A046.

19. EPRI 1025286, *Seismic Walkdown Guidance for Resolution of Fukushima Near-Term Task Force Recommendation 2.3: Seismic*, Palo Alto, CA, June 2012.

20. Duke Energy Carolina Letter to the NRC, *Response to NRC Request for Information Pursuant to Title 10 Code of Federal Regulations 50.54(f) Regarding the Seismic Aspects of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* dated March 28, 2013, ADAMS Accession No. ML13162A071.

21. Duke Energy Carolina Letter to the NRC, *Response to NRC Request for Information Pursuant to Title 10 Code of Federal Regulations 50.54(f) Regarding the Seismic Aspects of Recommendation 2.3 of the Near-Term Task Force Review of Insights from the Fukushima Dai-ichi Accident,* dated January 14, 2014, ADAMS Accession No. ML14041A294.

22. NRC (P. Tam) Letter to Catawba Nuclear Station (G. Peterson), *Catawba Nuclear Station – Review of Individual Plant Examination of External Events (IPEEE)*, dated April 12, 1999.

23. Duke Power Company Letter to the NRC, *Catawba Nuclear Station, Units 1 and 2, Individual Plant Examination of External Events (IPEEE) Submittal,* dated June 21, 1994.

24. EPRI NP-6359, Volume 1, *Seismic Margin Assessment of the Catawba Nuclear Station*, Palo Alto, CA, April 1989.

25. Duke Energy, LLC (Duke Energy) Letter to the NRC, *Duke Energy Response to NRC 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,* dated April 26, 2013, ADAMS Accession No. ML13121A061.

26. Calculation CNC 1139.01-23-0004, Catawba Nuclear Station, Units 1 & 2, *Auxiliary Building In Structure Response Spectra for Seismic IPEEE and EPRI Seismic Margins Study*, Rev. 0.

A

Additional Tables

Table A-1a Mean and fractile seismic hazard curves for PGA at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.10E-02	3.33E-02	4.37E-02	5.20E-02	5.91E-02	6.36E-02
0.001	4.11E-02	2.39E-02	3.42E-02	4.13E-02	4.90E-02	5.42E-02
0.005	1.64E-02	7.77E-03	1.15E-02	1.60E-02	2.04E-02	2.92E-02
0.01	8.84E-03	3.90E-03	5.35E-03	8.12E-03	1.11E-02	1.95E-02
0.015	5.77E-03	2.25E-03	3.14E-03	5.05E-03	7.55E-03	1.44E-02
0.03	2.49E-03	6.83E-04	1.01E-03	1.87E-03	3.63E-03	7.66E-03
0.05	1.22E-03	2.42E-04	3.73E-04	7.77E-04	1.84E-03	4.43E-03
0.075	6.51E-04	1.01E-04	1.69E-04	3.73E-04	9.65E-04	2.64E-03
0.1	4.06E-04	5.27E-05	9.79E-05	2.25E-04	5.91E-04	1.72E-03
0.15	2.01E-04	2.19E-05	4.63E-05	1.13E-04	2.84E-04	8.47E-04
0.3	5.54E-05	4.43E-06	1.20E-05	3.47E-05	8.23E-05	2.04E-04
0.5	2.00E-05	1.18E-06	3.90E-06	1.32E-05	3.23E-05	6.26E-05
0.75	8.41E-06	3.47E-07	1.46E-06	5.50E-06	1.42E-05	2.57E-05
1.	4.36E-06	1.32E-07	6.73E-07	2.72E-06	7.45E-06	1.36E-05
1.5	1.59E-06	3.14E-08	1.95E-07	9.11E-07	2.76E-06	5.35E-06
3.	2.15E-07	1.82E-09	1.42E-08	9.51E-08	3.52E-07	8.72E-07
5.	3.69E-08	2.64E-10	1.40E-09	1.21E-08	5.50E-08	1.72E-07
7.5	7.41E-09	1.53E-10	2.60E-10	1.90E-09	1.02E-08	3.79E-08
10.	2.10E-09	1.13E-10	1.53E-10	5.12E-10	2.72E-09	1.15E-08

Table A-1b Mean and fractile seismic hazard curves for 25 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.57E-02	4.07E-02	4.90E-02	5.58E-02	6.26E-02	6.73E-02
0.001	4.78E-02	3.23E-02	4.13E-02	4.83E-02	5.50E-02	6.00E-02
0.005	2.47E-02	1.36E-02	1.87E-02	2.42E-02	2.96E-02	3.84E-02
0.01	1.58E-02	8.00E-03	1.11E-02	1.51E-02	1.92E-02	2.80E-02
0.015	1.15E-02	5.58E-03	7.77E-03	1.08E-02	1.44E-02	2.22E-02
0.03	6.09E-03	2.57E-03	3.57E-03	5.50E-03	8.00E-03	1.34E-02
0.05	3.45E-03	1.23E-03	1.74E-03	2.92E-03	4.83E-03	8.47E-03
0.075	2.05E-03	6.09E-04	8.85E-04	1.62E-03	3.01E-03	5.50E-03
0.1	1.37E-03	3.57E-04	5.27E-04	1.04E-03	2.07E-03	3.95E-03
0.15	7.44E-04	1.57E-04	2.49E-04	5.20E-04	1.13E-03	2.32E-03
0.3	2.33E-04	3.68E-05	7.03E-05	1.57E-04	3.47E-04	7.55E-04
0.5	9.23E-05	1.23E-05	2.72E-05	6.64E-05	1.40E-04	2.72E-04
0.75	4.28E-05	4.83E-06	1.20E-05	3.28E-05	6.73E-05	1.18E-04
1.	2.44E-05	2.35E-06	6.45E-06	1.90E-05	4.01E-05	6.54E-05
1.5	1.06E-05	7.45E-07	2.49E-06	8.23E-06	1.82E-05	2.88E-05
3.	2.19E-06	7.77E-08	3.90E-07	1.53E-06	3.90E-06	6.64E-06
5.	5.68E-07	1.15E-08	7.45E-08	3.52E-07	1.02E-06	1.92E-06
7.5	1.69E-07	2.19E-09	1.60E-08	9.11E-08	2.96E-07	6.26E-07
10.	6.54E-08	6.93E-10	4.83E-09	3.09E-08	1.15E-07	2.60E-07

Table A-1c Mean and fractile seismic hazard curves for 10 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.94E-02	4.83E-02	5.35E-02	5.91E-02	6.54E-02	6.93E-02
0.001	5.26E-02	4.01E-02	4.56E-02	5.27E-02	5.91E-02	6.36E-02
0.005	2.83E-02	1.74E-02	2.19E-02	2.84E-02	3.42E-02	3.95E-02
0.01	1.79E-02	9.79E-03	1.29E-02	1.77E-02	2.22E-02	2.76E-02
0.015	1.27E-02	6.73E-03	8.85E-03	1.23E-02	1.60E-02	2.10E-02
0.03	6.31E-03	3.01E-03	3.95E-03	5.91E-03	8.23E-03	1.18E-02
0.05	3.36E-03	1.36E-03	1.87E-03	3.01E-03	4.63E-03	7.03E-03
0.075	1.88E-03	6.45E-04	9.24E-04	1.60E-03	2.72E-03	4.37E-03
0.1	1.19E-03	3.57E-04	5.35E-04	9.65E-04	1.77E-03	2.96E-03
0.15	5.91E-04	1.46E-04	2.29E-04	4.50E-04	8.98E-04	1.62E-03
0.3	1.56E-04	2.72E-05	5.12E-05	1.15E-04	2.39E-04	4.50E-04
0.5	5.52E-05	7.55E-06	1.69E-05	4.19E-05	8.72E-05	1.53E-04
0.75	2.35E-05	2.53E-06	6.45E-06	1.82E-05	3.90E-05	6.26E-05
1.	1.26E-05	1.10E-06	3.14E-06	9.79E-06	2.13E-05	3.42E-05
1.5	5.00E-06	3.09E-07	1.08E-06	3.73E-06	8.60E-06	1.44E-05
3.	8.50E-07	2.49E-08	1.34E-07	5.58E-07	1.46E-06	2.84E-06
5.	1.86E-07	3.09E-09	2.07E-08	1.07E-07	3.19E-07	7.03E-07
7.5	4.77E-08	5.75E-10	3.79E-09	2.32E-08	7.89E-08	1.98E-07
10.	1.66E-08	2.35E-10	1.07E-09	7.03E-09	2.68E-08	7.34E-08

Table A-1d Mean and fractile seismic hazard curves for 5 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.97E-02	4.83E-02	5.35E-02	6.00E-02	6.64E-02	7.03E-02
0.001	5.28E-02	3.95E-02	4.50E-02	5.27E-02	6.00E-02	6.45E-02
0.005	2.64E-02	1.51E-02	1.98E-02	2.64E-02	3.33E-02	3.68E-02
0.01	1.53E-02	7.89E-03	1.10E-02	1.51E-02	1.98E-02	2.32E-02
0.015	1.02E-02	5.12E-03	7.03E-03	9.93E-03	1.34E-02	1.62E-02
0.03	4.47E-03	2.01E-03	2.80E-03	4.19E-03	6.09E-03	7.89E-03
0.05	2.16E-03	8.23E-04	1.16E-03	1.95E-03	3.09E-03	4.31E-03
0.075	1.11E-03	3.52E-04	5.27E-04	9.51E-04	1.67E-03	2.49E-03
0.1	6.56E-04	1.82E-04	2.84E-04	5.35E-04	1.01E-03	1.60E-03
0.15	2.92E-04	6.73E-05	1.11E-04	2.25E-04	4.50E-04	7.77E-04
0.3	6.39E-05	1.05E-05	2.07E-05	4.83E-05	1.01E-04	1.77E-04
0.5	1.97E-05	2.39E-06	5.58E-06	1.51E-05	3.23E-05	5.35E-05
0.75	7.60E-06	6.73E-07	1.84E-06	5.75E-06	1.29E-05	2.13E-05
1.	3.78E-06	2.60E-07	8.12E-07	2.76E-06	6.45E-06	1.10E-05
1.5	1.35E-06	5.91E-08	2.35E-07	9.11E-07	2.35E-06	4.25E-06
3.	1.87E-07	3.42E-09	1.98E-08	1.02E-07	3.28E-07	6.83E-07
5.	3.50E-08	4.25E-10	2.29E-09	1.49E-08	5.91E-08	1.44E-07
7.5	7.84E-09	1.62E-10	4.13E-10	2.60E-09	1.25E-08	3.47E-08
10.	2.47E-09	1.49E-10	1.82E-10	7.34E-10	3.79E-09	1.13E-08

Table A-1e Mean and fractile seismic hazard curves for 2.5 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	5.55E-02	4.25E-02	4.77E-02	5.58E-02	6.26E-02	6.73E-02
0.001	4.56E-02	3.14E-02	3.68E-02	4.56E-02	5.42E-02	5.91E-02
0.005	1.70E-02	9.24E-03	1.21E-02	1.67E-02	2.19E-02	2.57E-02
0.01	8.43E-03	4.19E-03	5.58E-03	8.12E-03	1.13E-02	1.38E-02
0.015	5.13E-03	2.32E-03	3.19E-03	4.83E-03	7.03E-03	8.98E-03
0.03	1.87E-03	6.54E-04	9.79E-04	1.67E-03	2.76E-03	3.79E-03
0.05	7.67E-04	2.07E-04	3.28E-04	6.36E-04	1.21E-03	1.79E-03
0.075	3.38E-04	7.34E-05	1.23E-04	2.53E-04	5.42E-04	8.98E-04
0.1	1.78E-04	3.33E-05	5.75E-05	1.25E-04	2.88E-04	5.05E-04
0.15	6.73E-05	1.01E-05	1.87E-05	4.37E-05	1.08E-04	1.98E-04
0.3	1.15E-05	1.05E-06	2.46E-06	7.23E-06	1.90E-05	3.42E-05
0.5	3.07E-06	1.72E-07	5.20E-07	1.84E-06	5.27E-06	9.79E-06
0.75	1.08E-06	3.57E-08	1.36E-07	6.00E-07	1.92E-06	3.73E-06
1.	5.02E-07	1.10E-08	4.90E-08	2.53E-07	8.98E-07	1.87E-06
1.5	1.61E-07	1.87E-09	1.01E-08	6.73E-08	2.88E-07	6.54E-07
3.	1.81E-08	1.79E-10	5.50E-10	4.77E-09	2.92E-08	8.23E-08
5.	2.79E-09	1.21E-10	1.53E-10	5.50E-10	3.95E-09	1.31E-08
7.5	5.31E-10	9.37E-11	1.21E-10	1.72E-10	7.34E-10	2.53E-09
10.	1.48E-10	9.11E-11	1.01E-10	1.53E-10	2.68E-10	7.66E-10

Table A-1f Mean and fractile seismic hazard curves for 1 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	4.04E-02	2.35E-02	3.05E-02	4.13E-02	4.98E-02	5.50E-02
0.001	2.78E-02	1.44E-02	1.98E-02	2.80E-02	3.52E-02	4.07E-02
0.005	8.00E-03	3.42E-03	4.90E-03	7.66E-03	1.10E-02	1.38E-02
0.01	3.96E-03	1.27E-03	2.01E-03	3.57E-03	5.91E-03	7.89E-03
0.015	2.36E-03	6.09E-04	1.02E-03	2.04E-03	3.68E-03	5.20E-03
0.03	7.34E-04	1.21E-04	2.32E-04	5.58E-04	1.20E-03	1.98E-03
0.05	2.44E-04	2.88E-05	6.00E-05	1.62E-04	4.19E-04	7.45E-04
0.075	8.86E-05	8.47E-06	1.82E-05	5.12E-05	1.53E-04	2.92E-04
0.1	4.07E-05	3.37E-06	7.34E-06	2.16E-05	6.93E-05	1.38E-04
0.15	1.29E-05	8.47E-07	1.98E-06	6.26E-06	2.19E-05	4.50E-05
0.3	1.85E-06	6.36E-08	1.92E-07	7.89E-07	3.09E-06	7.23E-06
0.5	4.86E-07	7.66E-09	3.01E-08	1.69E-07	7.89E-07	2.10E-06
0.75	1.70E-07	1.29E-09	6.17E-09	4.56E-08	2.64E-07	7.89E-07
1.	7.89E-08	4.01E-10	1.90E-09	1.67E-08	1.15E-07	3.79E-07
1.5	2.49E-08	1.60E-10	4.07E-10	3.63E-09	3.14E-08	1.21E-07
3.	2.72E-09	1.01E-10	1.53E-10	2.72E-10	2.49E-09	1.27E-08
5.	4.20E-10	9.11E-11	1.01E-10	1.53E-10	3.68E-10	1.82E-09
7.5	8.10E-11	9.11E-11	1.01E-10	1.53E-10	1.60E-10	4.07E-10
10.	2.30E-11	9.11E-11	9.11E-11	1.53E-10	1.53E-10	1.92E-10

Table A-1g Mean and fractile seismic hazard curves for 0.5 Hz at Catawba, 5% of critical damping

AMPS(g)	MEAN	0.05	0.16	0.50	0.84	0.95
0.0005	2.27E-02	1.31E-02	1.74E-02	2.22E-02	2.80E-02	3.28E-02
0.001	1.43E-02	7.66E-03	1.02E-02	1.36E-02	1.84E-02	2.25E-02
0.005	4.26E-03	1.23E-03	2.04E-03	3.90E-03	6.45E-03	8.47E-03
0.01	2.04E-03	3.42E-04	6.73E-04	1.64E-03	3.42E-03	4.98E-03
0.015	1.15E-03	1.34E-04	2.88E-04	8.23E-04	2.01E-03	3.28E-03
0.03	3.16E-04	2.01E-05	4.77E-05	1.77E-04	5.58E-04	1.11E-03
0.05	9.52E-05	4.13E-06	1.02E-05	4.13E-05	1.67E-04	3.73E-04
0.075	3.24E-05	1.05E-06	2.76E-06	1.10E-05	5.50E-05	1.31E-04
0.1	1.43E-05	3.90E-07	1.07E-06	4.19E-06	2.35E-05	5.83E-05
0.15	4.37E-06	8.85E-08	2.68E-07	1.10E-06	6.64E-06	1.87E-05
0.3	5.99E-07	5.12E-09	1.98E-08	1.16E-07	8.00E-07	3.05E-06
0.5	1.55E-07	5.35E-10	2.53E-09	2.10E-08	1.79E-07	8.47E-07
0.75	5.44E-08	1.77E-10	5.20E-10	4.98E-09	5.27E-08	2.96E-07
1.	2.56E-08	1.53E-10	2.19E-10	1.67E-09	2.07E-08	1.36E-07
1.5	8.40E-09	1.01E-10	1.53E-10	4.07E-10	4.98E-09	4.19E-08
3.	1.01E-09	9.11E-11	1.01E-10	1.53E-10	4.19E-10	4.25E-09
5.	1.70E-10	9.11E-11	1.01E-10	1.53E-10	1.57E-10	6.73E-10
7.5	3.55E-11	9.11E-11	9.11E-11	1.53E-10	1.53E-10	2.13E-10
10.	1.07E-11	9.11E-11	9.11E-11	1.53E-10	1.53E-10	1.53E-10

Table A- 2 Amplification functions for Catawba, 5% of critical damping

PGA	Median AF	Sigma ln(AF)	25 Hz	Median AF	Sigma ln(AF)	10 Hz	Median AF	Sigma ln(AF)	5 Hz	Median AF	Sigma ln(AF)
1.00E-02	1.01E+00	7.32E-02	1.30E-02	1.05E+00	7.22E-02	1.90E-02	1.06E+00	7.28E-02	2.09E-02	1.05E+00	8.91E-02
4.95E-02	1.05E+00	6.72E-02	1.02E-01	1.12E+00	1.01E-01	9.99E-02	1.07E+00	7.12E-02	8.24E-02	1.05E+00	8.83E-02
9.64E-02	1.06E+00	6.96E-02	2.13E-01	1.13E+00	1.05E-01	1.85E-01	1.08E+00	7.09E-02	1.44E-01	1.06E+00	8.79E-02
1.94E-01	1.06E+00	7.23E-02	4.43E-01	1.13E+00	1.06E-01	3.56E-01	1.08E+00	7.07E-02	2.65E-01	1.06E+00	8.77E-02
2.92E-01	1.06E+00	7.38E-02	6.76E-01	1.13E+00	1.07E-01	5.23E-01	1.08E+00	7.07E-02	3.84E-01	1.06E+00	8.76E-02
3.91E-01	1.06E+00	7.47E-02	9.09E-01	1.13E+00	1.07E-01	6.90E-01	1.08E+00	7.07E-02	5.02E-01	1.06E+00	8.75E-02
4.93E-01	1.07E+00	7.53E-02	1.15E+00	1.13E+00	1.08E-01	8.61E-01	1.08E+00	7.08E-02	6.22E-01	1.06E+00	8.75E-02
7.41E-01	1.07E+00	7.64E-02	1.73E+00	1.13E+00	1.08E-01	1.27E+00	1.08E+00	7.11E-02	9.13E-01	1.06E+00	8.75E-02
1.01E+00	1.07E+00	7.71E-02	2.36E+00	1.13E+00	1.09E-01	1.72E+00	1.08E+00	7.15E-02	1.22E+00	1.06E+00	8.75E-02
1.28E+00	1.07E+00	7.74E-02	3.01E+00	1.13E+00	1.09E-01	2.17E+00	1.08E+00	7.21E-02	1.54E+00	1.06E+00	8.76E-02
1.55E+00	1.07E+00	7.77E-02	3.63E+00	1.13E+00	1.09E-01	2.61E+00	1.09E+00	7.26E-02	1.85E+00	1.06E+00	8.77E-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	8.83E-01	8.40E-02	1.27E-02	1.03E+00	4.66E-02	8.25E-03	1.10E+00	1.45E-01
7.05E-02	8.85E-01	8.38E-02	3.43E-02	1.03E+00	4.61E-02	1.96E-02	1.10E+00	1.42E-01
1.18E-01	8.86E-01	8.36E-02	5.51E-02	1.03E+00	4.60E-02	3.02E-02	1.10E+00	1.41E-01
2.12E-01	8.87E-01	8.34E-02	9.63E-02	1.03E+00	4.59E-02	5.11E-02	1.10E+00	1.41E-01
3.04E-01	8.87E-01	8.33E-02	1.36E-01	1.03E+00	4.58E-02	7.10E-02	1.09E+00	1.41E-01
3.94E-01	8.88E-01	8.32E-02	1.75E-01	1.03E+00	4.58E-02	9.06E-02	1.09E+00	1.40E-01
4.86E-01	8.88E-01	8.32E-02	2.14E-01	1.03E+00	4.58E-02	1.10E-01	1.09E+00	1.40E-01
7.09E-01	8.88E-01	8.31E-02	3.10E-01	1.03E+00	4.58E-02	1.58E-01	1.09E+00	1.40E-01
9.47E-01	8.89E-01	8.31E-02	4.12E-01	1.03E+00	4.58E-02	2.09E-01	1.09E+00	1.40E-01
1.19E+00	8.89E-01	8.31E-02	5.18E-01	1.03E+00	4.58E-02	2.62E-01	1.09E+00	1.40E-01
1.43E+00	8.89E-01	8.31E-02	6.19E-01	1.03E+00	4.58E-02	3.12E-01	1.09E+00	1.40E-01

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 1E-4 and 1E-5 mean annual frequency of exceedance. These tables concentrate on the frequency range of 0.5 Hz to 25 Hz, with values up to 100 Hz included, and a single value at 0.1 Hz included for completeness. 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=0.194		M1P1K1		PGA=0.741	
Freq (Hz)	Soil SA	Median AF	Sigma ln(AF)	Freq (Hz)	Soil SA	Median AF	Sigma ln(AF)
100.0	0.203	1.046	0.068	100.0	0.776	1.048	0.071
87.1	0.208	1.048	0.069	87.1	0.802	1.048	0.072
75.9	0.219	1.050	0.070	75.9	0.850	1.050	0.074
66.1	0.239	1.053	0.075	66.1	0.947	1.048	0.080
57.5	0.280	1.055	0.087	57.5	1.136	1.043	0.094
50.1	0.344	1.080	0.102	50.1	1.418	1.068	0.108
43.7	0.409	1.086	0.112	43.7	1.683	1.072	0.115
38.0	0.452	1.091	0.115	38.0	1.844	1.083	0.117
33.1	0.474	1.080	0.112	33.1	1.913	1.079	0.115
28.8	0.482	1.097	0.104	28.8	1.921	1.100	0.107
25.1	0.481	1.086	0.093	25.1	1.891	1.090	0.097
21.9	0.474	1.122	0.082	21.9	1.835	1.129	0.086
19.1	0.462	1.108	0.074	19.1	1.764	1.117	0.078
16.6	0.448	1.116	0.070	16.6	1.684	1.125	0.073
14.5	0.431	1.126	0.068	14.5	1.602	1.134	0.070
12.6	0.414	1.109	0.068	12.6	1.516	1.115	0.070
11.0	0.395	1.086	0.071	11.0	1.433	1.091	0.072
9.5	0.376	1.081	0.074	9.5	1.348	1.085	0.075
8.3	0.356	1.108	0.076	8.3	1.263	1.112	0.077
7.2	0.337	1.120	0.076	7.2	1.185	1.123	0.077
6.3	0.315	1.115	0.081	6.3	1.100	1.118	0.081
5.5	0.296	1.095	0.078	5.5	1.023	1.097	0.078
4.8	0.276	1.046	0.090	4.8	0.950	1.048	0.090
4.2	0.258	1.006	0.079	4.2	0.880	1.008	0.079
3.6	0.240	0.961	0.091	3.6	0.813	0.962	0.091
3.2	0.221	0.940	0.091	3.2	0.746	0.941	0.091
2.8	0.206	0.924	0.113	2.8	0.692	0.925	0.112
2.4	0.187	0.906	0.069	2.4	0.623	0.907	0.069
2.1	0.174	0.931	0.072	2.1	0.579	0.932	0.071
1.8	0.164	0.982	0.123	1.8	0.543	0.982	0.123
1.6	0.147	1.010	0.147	1.6	0.482	1.010	0.146
1.4	0.126	1.009	0.097	1.4	0.412	1.009	0.097
1.2	0.112	1.013	0.035	1.2	0.363	1.013	0.035
1.0	0.103	1.032	0.023	1.0	0.331	1.032	0.023
0.91	0.096	1.061	0.045	0.91	0.308	1.060	0.045
0.79	0.089	1.091	0.075	0.79	0.284	1.090	0.075
0.69	0.081	1.116	0.109	0.69	0.256	1.114	0.109
0.60	0.072	1.127	0.135	0.60	0.224	1.125	0.135
0.52	0.061	1.124	0.145	0.52	0.189	1.122	0.144
0.46	0.050	1.111	0.139	0.46	0.155	1.110	0.138
0.10	0.002	1.022	0.036	0.10	0.006	1.017	0.031

Table A2-b2 Median AFs and sigmas for Model 2, Profile 1, for 2 PGA levels

M2P1K1		PGA=0.194		M2P1K1		PGA=0.741	
Freq (Hz)	Soil SA	Median AF	Sigma ln(AF)	Freq (Hz)	Soil SA	Median AF	Sigma ln(AF)
100.0	0.200	1.033	0.055	100.0	0.770	1.039	0.059
87.1	0.206	1.034	0.055	87.1	0.796	1.041	0.061
75.9	0.216	1.037	0.057	75.9	0.846	1.044	0.063
66.1	0.235	1.037	0.063	66.1	0.943	1.044	0.071
57.5	0.274	1.034	0.075	57.5	1.130	1.038	0.085
50.1	0.336	1.054	0.089	50.1	1.410	1.061	0.097
43.7	0.401	1.065	0.098	43.7	1.679	1.069	0.103
38.0	0.448	1.080	0.107	38.0	1.850	1.087	0.111
33.1	0.475	1.081	0.112	33.1	1.927	1.087	0.116
28.8	0.485	1.104	0.109	28.8	1.936	1.108	0.112
25.1	0.484	1.091	0.096	25.1	1.896	1.094	0.098
21.9	0.474	1.123	0.079	21.9	1.830	1.126	0.081
19.1	0.460	1.103	0.064	19.1	1.749	1.107	0.065
16.6	0.444	1.106	0.054	16.6	1.662	1.110	0.055
14.5	0.425	1.109	0.052	14.5	1.572	1.113	0.052
12.6	0.406	1.090	0.054	12.6	1.485	1.092	0.054
11.0	0.387	1.064	0.057	11.0	1.399	1.066	0.056
9.5	0.367	1.056	0.060	9.5	1.314	1.058	0.060
8.3	0.347	1.082	0.065	8.3	1.231	1.083	0.065
7.2	0.328	1.089	0.072	7.2	1.151	1.090	0.071
6.3	0.309	1.092	0.072	6.3	1.075	1.093	0.071
5.5	0.286	1.059	0.067	5.5	0.989	1.060	0.066
4.8	0.271	1.026	0.085	4.8	0.931	1.027	0.084
4.2	0.249	0.971	0.075	4.2	0.849	0.972	0.075
3.6	0.235	0.943	0.084	3.6	0.798	0.944	0.083
3.2	0.215	0.914	0.084	3.2	0.724	0.915	0.084
2.8	0.198	0.889	0.106	2.8	0.665	0.890	0.105
2.4	0.183	0.890	0.057	2.4	0.611	0.890	0.057
2.1	0.172	0.920	0.081	2.1	0.572	0.921	0.081
1.8	0.158	0.946	0.118	1.8	0.524	0.946	0.117
1.6	0.140	0.963	0.128	1.6	0.460	0.963	0.127
1.4	0.122	0.978	0.073	1.4	0.400	0.978	0.073
1.2	0.110	1.000	0.030	1.2	0.358	1.000	0.030
1.0	0.102	1.026	0.038	1.0	0.329	1.025	0.037
0.91	0.095	1.049	0.060	0.91	0.304	1.048	0.060
0.79	0.087	1.067	0.088	0.79	0.278	1.066	0.087
0.69	0.078	1.076	0.115	0.69	0.247	1.075	0.114
0.60	0.068	1.076	0.133	0.60	0.214	1.075	0.132
0.52	0.058	1.069	0.136	0.52	0.180	1.068	0.135
0.46	0.048	1.059	0.126	0.46	0.148	1.058	0.126
0.10	0.002	1.010	0.033	0.10	0.006	1.006	0.027