Vogtle PEmails

From: Christian Araguas
Sent: Wednesday, November 12, 2008 3:18 PM
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Subject: Advanced Safety Evaluation Report on the VEGP ESP and LWA Part 3b/3
Attachments: Vogtle ESP Advanced SER Chapter 2.5.2 - 2.5.6.pdf

Jim,

Attached are the pdf files for each of the individual chapters in the advanced SER. Please begin your proprietary review. Following your review, the documents will be made public in ADAMS and will be posted on the NRC's public website. Let me know if you have any questions. The signed cover letter should go out today. Please confirm receipt of the attached files. Thanks.

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2.5.2 Vibratory Ground Motion

2.5.2.1 Introduction/Overview/General

SSAR Section 2.5.2 describes the applicant's determination of the ground motion response spectrum (GMRS) at the Early Site Permit (ESP) site from potential earthquakes in the site area and region. SSAR Section 2.5.2.1 describes the earthquake catalog used for the ESP site, SSAR Section 2.5.2.2 summarizes the geologic structures and tectonic activity that could potentially result in ground motion at the ESP site, and SSAR Section 2.5.2.3 describes the correlation of earthquake activity with geologic structures or tectonic provinces. SSAR Section 2.5.2.4 describes the earthquake potential for seismic sources in the region surrounding the ESP site, SSAR Section 2.5.2.5 describes the seismic wave transmission characteristics of the site, SSAR Section 2.5.2.6 provides the horizontal GMRS, SSAR Section 2.5.2.7 provides the vertical GMRS, SSAR Section 2.5.2.8 discusses the operating-basis earthquake ground motion spectrum, and SSAR Section 2.5.2.9 describes the results of site response sensitivity calculations.

The applicant stated that the information provided in SSAR Section 2.5.2 of the ESP application uses the procedures recommended in RG 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," issued March 1997, for performing the Probabilistic Seismic Hazard Analysis (PSHA) for the ESP site. However, rather than using the reference-probability approach described in Regulatory Guide (RG) 1.165 for determining the SSE, the applicant developed the GMRS using the performance-based method described in RG 1.208, A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion," issued March 2007. According to RG 1.208, the GMRS represents the first part of the development of the safe-shutdown earthquake (SSE) for a site. In addition, the applicant used the 1986 EPRI [Electric Power Research Institute] Project (EPRI NP-4726) seismic source model for the Central and Eastern United States (CEUS) as an input for its seismic ground motion calculations. RG 1.165 indicates that applicants may use the seismic source interpretations developed by Lawrence Livermore National Laboratory (LLNL (1993) or EPRI as inputs for a site-specific analysis. RG 1.165 also recommends a review and update, if necessary, of both the seismic source and ground motion models used to develop the SSE ground motion for the ESP site.

To determine whether an update of the seismic source and ground motion models used in the 1989 EPRI PSHA (EPRI NP-6395-D) was necessary, the applicant reviewed the literature published since the mid-to-late 1980s. This literature review identified the need for changes to the source characterization parameters of the Charleston seismic zone. In addition, the applicant determined that the ground motion models used for the 1989 EPRI PSHA needed to be updated.

2.5.2.2 Summary of Application

2.5.2.2.1 Seismicity

SSAR Section 2.5.2.1 describes the development of a current earthquake catalog for the ESP site. The applicant started with the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is complete through 1984. To update the earthquake catalog, the applicant used

information from the Advanced National Seismic System (ANSS) and the South Eastern United States Seismic Network (SEUSS).

The EPRI catalog covers the time period from 1627 to 1984 and contains earthquakes that occurred within the CEUS. Earthquakes comprising the EPRI catalog are characterized by a variety of different size measures, including local magnitude (ML), surface-wave magnitude (MS), duration or coda magnitude (Md or Mc), body-wave magnitude (mbLg), felt area (FA), and epicentral Modified Mercalli (MM) intensity (Io). Earthquake measures such as ML, MS, Md, Mc, and mbLg are based on characteristics of instrumentally recorded events. Md and Mc are related to the duration of a recorded earthquake, while ML, MS, and mbLg are related to the amplitude of a recorded earthquake. FA and Io, are based on qualitative descriptions of the effects of the earthquake at a particular location (Kramer 1996).

All earthquakes comprising the EPRI catalog are described in terms of mb. The applicant converted all earthquakes that were not originally characterized by mb to best, or expected, estimates of mb (E[mb]) using conversion factors developed in EPRI NP-4726-A (1988). EPRI NP-4726-A (1988) developed these conversion factors from regression models relating mb to ML, MS, Md or Mc; FA; and Io. In addition, the 1988 EPRI study calculated a uniform magnitude (mb*) from Emb and the variance of mb (σ2mb) in order to account for the uncertainty in estimating mb.

The applicant only selected earthquakes from the EPRI historical catalog that occurred within the site region (320-kilometer (km) or 200-mile (mi) radius). In addition, the applicant updated the EPRI historical seismicity catalog to incorporate earthquakes that have occurred within the site region since 1984. To update the EPRI earthquake catalog, the applicant used information from the ANSS and the SEUSS. Of these two catalogs, the applicant primarily used the SEUSS catalog for the period from 1985 to 2005. Events in the SEUSS and ANSS catalogs that have occurred since 1985 are primarily reported as mbLg, ML, Mc, and Md. To be consistent with the mb estimates provided in the EPRI catalog, the applicant converted the magnitudes given in both the SEUSS and ANSS catalogs to E[mb]. The applicant included a total of 61 events with E[mb] magnitude greater than 3.0 in the update of the EPRI NP-4726-A (1988) seismicity catalog. The applicant also calculated mb* using E[mb] and σ2mb (estimated from the ANSS and SEUSS catalogs).

As shown in Figure 2.5.2-1 of this SER, a comparison of the geographic distribution of earthquakes contained in the EPRI catalog (1627–1984) and the earthquakes contained in the updated catalog (1985–2005) shows a very similar spatial distribution. The cluster of events along the coast of South Carolina is related to the Charleston Seismic Zone, while the cluster of events in eastern Tennessee is associated with the Eastern Tennessee Seismic Zone (ETSZ). The ETSZ extends from southwest Virginia to northeast Alabama.

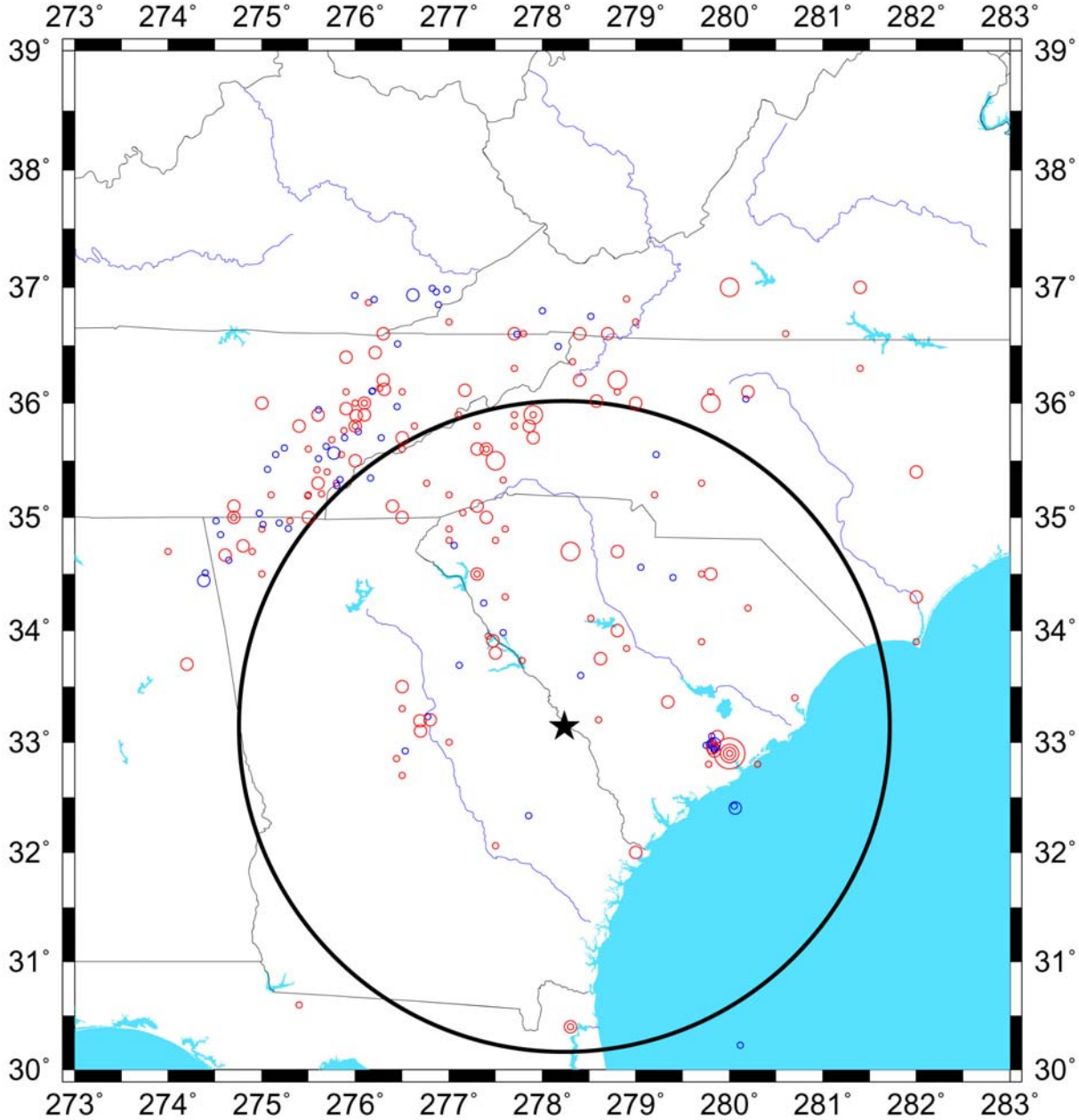


Figure 2.5.2-1 - A comparison of events (mb greater than 3) from the EPRI historical catalog (depicted by blue circles) with events from the applicant's updated catalog (depicted by red circles). The star corresponds to the location of the ESP site and the large black circle corresponds to the 200-mi site radius.

2.5.2.2.2 Geologic and Tectonic Characteristics of the Site and Region

SSAR Section 2.5.2.2 describes the seismic sources and seismicity parameters that the applicant used to calculate the seismic ground motion hazard for the ESP site. Specifically, the applicant described the seismic source interpretations from the 1986 EPRI Project (EPRI NP-4726 1986), relevant post-EPRI seismic source characterization studies, and its updated EPRI seismic source zone for the Charleston area based on more recent data.

Summary of EPRI Seismic Sources

The applicant used the 1986 EPRI seismic source model for the CEUS as a starting point for its seismic ground motion calculations. The 1986 EPRI seismic source model is comprised of input from six independent earth science teams (ESTs), which included the Bechtel Group, Dames and Moore, Law Engineering, Rondout Associates, Weston Geophysical Corporation, and Woodward-Clyde Consultants. Each team evaluated geological, geophysical, and seismological data to develop a model of seismic sources in the CEUS. The 1989 EPRI PSHA study (EPRI NP-6395-D 1989) subsequently incorporated each of the EST models. SSAR Sections 2.5.2.2.1.1 through 2.5.2.2.1.6 provide a summary of the primary seismic sources developed by each of the six ESTs. As stated in SSAR Section 2.5.2.2.1, the 1989 EPRI seismic hazard calculations implemented screening criteria to include only those sources with a combined hazard that exceeded 99 percent of the total hazard from all sources for two ground-motion measures (EPRI NP-6395-D 1989).

Each EST representation of seismic source zones affecting the ESP site region differs significantly in terms of total number of source zones and source characterization parameters such as geometry and maximum magnitudes (and associated weights). For example, the total number of primary source zones identified by each EST ranged from 2 (Rondout Associates team) to 15 (Law Engineering team). However, all teams identified and characterized one or more seismic source zones or background sources that accounted for seismicity in the vicinity of the ESP site. In addition, all of the ESTs identified and characterized one or more seismic source zones to account for the occurrence of Charleston-type earthquakes.

SER Table 2.5.2-1 provides the sources that account for Charleston-type earthquakes. The largest maximum magnitudes (Mmax) assigned to the Charleston source zone by each team ranged from mb 6.8 (Law Engineering, with a weight of 1) to mb 7.5 (Woodward-Clyde, with a weight of 0.33). This corresponds to a moment magnitude (M) range of 6.8 to 8.0.

Table 2.5.2-1 - Summary of EPRI EST Charleston Seismic Sources (Based on Information Provided in SSAR Tables 2.5.2-2 to 2.5.2-7)

EPRI EST	Source	Description	Probability of Activity	Mmax (mb) and Weights
Bechtel	H	Charleston Area	0.50	6.8 [0.20] 7.1 [0.40] 7.4 [0.40]
	N3	Charleston Faults	0.53	6.8 [0.20] 7.1 [0.40] 7.4 [0.40]
Dames & Moore	54	Charleston Seismic Zone	1.00	6.6 [0.75] 7.2 [0.25]
Law Engineering	35	Charleston Seismic Zone	0.45	6.8 [1.0]
Rondout	24	Charleston	1.0	6.6 [0.20] 6.8 [0.60] 7.0 [0.20]
Weston	25	Charleston Seismic Zone	0.99	6.6 [0.90] 7.2 [0.10]
Woodward-Clyde	30	Charleston (includes NOTA)	0.573	6.8 [0.33] 7.3 [0.34] 7.5 [0.33]
	29	S. Carolina Gravity Saddle (Extended)	0.122	6.7 [0.33] 7.0 [0.34] 7.4 [0.33]
	29A	S. Carolina Gravity Saddle No. 2 (Combo C3)	0.305	6.7 [0.33] 7.0 [0.34] 7.4 [0.33]

Post-EPRI Seismic Source Characterization Studies

SSAR Section 2.5.2.2.2 focuses on the Charleston seismic source zone. The applicant described several PSHA studies that were completed after the 1989 EPRI PSHA, which involved the characterization of seismic sources within the ESP site region. These PSHA studies developed models of the Charleston seismic source that differed from those used in the 1989 EPRI PSHA study because they incorporated recent paleoliquefaction data. The applicant also provided its justification for not updating the EPRI seismic source parameters for the ETSZ, which is situated at the edge of the 320-km (200-mi) site region radius.

Charleston Seismic Source Zone. SSAR Section 2.5.2.2.2 describes three post-EPRI (1989) PSHA studies that characterized the seismic sources within the ESP site region. These studies include the USGS National Seismic Hazard Mapping Project (Frankel et al. 1996, 2002) and the South Carolina DOT (SCDOT) seismic hazard mapping project (Chapman and Talwani 2002). Unlike the EPRI study, these PSHA studies developed models of the Charleston seismic source that incorporated recent paleoliquefaction data.

The applicant stated that abundant soil liquefaction features induced by the 1886 Charleston earthquake, as well as other large prehistoric earthquakes that date back to the mid-Holocene (at least 5000 years), are preserved in geologic deposits at numerous locations within the 1886 meizoseismal area and along the South Carolina coast. In 2001, Talwani and Schaeffer (2001) reevaluated all of the liquefaction data previously compiled for the Charleston area and, based on recalibrated radiocarbon dates for liquefaction features, provided an estimate of earthquake recurrence for the region. Talwani and Schaeffer (2001) reinterpreted radiocarbon dates for previously published liquefaction features documented along the coast of South Carolina. Radiocarbon dates are useful in providing contemporary, minimum, and maximum limiting ages for liquefaction features. Talwani and Schaeffer (2001) recalculated previously compiled age data to account for fluctuations in atmospheric carbon-14 over time. They used the calibrated data to correlate ages of past individual earthquakes and then to estimate earthquake recurrence. Talwani and Schaeffer (2001) also identified individual earthquake episodes based on samples with a "contemporary" age constraint that had overlapping calibrated radiocarbon ages at the 68 percent (1-sigma) confidence interval. They calculated the estimated age of each earthquake from the weighted averages of overlapping contemporary ages. Talwani and Schaeffer (2001) identified a total of eight events from the paleoliquefaction record, including the 1886 Charleston event. These events are referred to as 1886, A, B, C, D, E, F, and G (in order of increasing age).

Talwani and Schaeffer (2001) proposed two scenarios to explain the distribution and timing of paleoliquefaction features (shown in SSAR Table 2.5.2-13). In Scenario 1, they interpreted events A, B, E, and G to be large Charleston-type events, while they interpreted events C, D, and F to be smaller, moderate magnitude (~**M** 6) events. In Scenario 2, Talwani and Schaeffer (2001) interpreted all events as large, Charleston-type events. In addition, they combined events C and D into a large event C' based on the observation that the calibrated radiocarbon ages that constrain the timing of Events C and D are indistinguishable at the 95 percent (2-sigma) confidence interval.

In 2002, the USGS updated the seismic hazard maps for the contiguous United States based on new seismological, geophysical, and geologic information (Frankel et al. 2002). The 2002 USGS update included modifications to the geometry, recurrence, and Mmax of the Charleston seismic source zone. In its update, the USGS represented Charleston-type earthquakes by two equally weighted areal sources. One of these seismic source zones envelops most of the tectonic features and liquefaction data in the greater Charleston area, while the other source envelops the southern half of the southern segment of the East Coast Fault System (ECFS). Frankel et al. (2002) adopted a mean paleoliquefaction-based recurrence interval of 550 years for Charleston-type earthquakes which ranged from **M** 6.8 to 7.5.

The SCDOT model (Chapman and Talwani 2002) characterized Charleston-type earthquakes by using a combination of three equally weighted line and area sources. The SCDOT model comprises a coastal South Carolina areal source zone that includes most of the paleoliquefaction sites, a source that captures the intersection of the Woodstock and Ashley River faults, and a source that represents the southern ECFS source zone. For Charleston-type earthquakes, which ranged from **M** 7.1 to 7.5, Chapman and Talwani (2002) also adopted a mean paleoliquefaction-based recurrence interval of 550 years.

The applicant briefly mentioned the Trial Implementation Project (TIP) study in the SSAR. However, the applicant did not explicitly include the findings of this study in the SSAR because

the TIP study primarily focused on the implementation of the Senior Seismic Hazard Advisory Committee (SSHAC) methodology, rather than the actual seismic hazard estimation.

Eastern Tennessee Seismic Zone. In SSAR Section 2.5.2.2.2.5, the applicant concluded that no new information regarding the ETSZ has been developed since 1986 that would require a significant revision to the original EPRI seismic source model. The applicant noted that despite being one of the most active seismic zones in Eastern North America, no evidence for larger prehistoric earthquakes, such as paleoliquefaction features, has been discovered. The largest earthquake recorded in the ETSZ was a magnitude 4.6 and occurred in 1973. The applicant also noted that a much higher degree of uncertainty is associated with the assignment of Mmax for the ETSZ than for other CEUS seismic source zones where values of Mmax are constrained by paleoliquefaction data.

The 1986 EPRI seismic source model (EPRI NP-4726 1986) included various source geometries and parameters to represent the seismicity of the ETSZ. All of the EPRI ESTs, except for the Law Engineering team, represented this area of seismicity with one or more local source zones. The Law Engineering team's Eastern Basement source zone included the ETSZ seismic source zone. With the exception of the Law Engineering team's Eastern Basement source, none of the other ETSZ sources contributed more than 1 percent to the site hazard, and thus were excluded from the final 1989 EPRI PSHA hazard calculations (EPRI NP-6452-D 1989).

Upper-bound maximum values of Mmax developed by the EPRI teams for the ETSZ ranged from **M** 4.8 to 7.5. The applicant found that Mmax estimates for the ETSZ in more recent studies fall within the range of magnitudes captured by the EPRI model. Bollinger (1992) estimated an Mmax of **M** 6.3, while the USGS hazard model (Frankel et al. 2002) assigned a single Mmax value of **M** 7.5 for the ETSZ.

Updated EPRI Seismic Sources

Based on the results of several post-EPRI PSHA studies (Frankel et al. 2002; Chapman and Talwani 2002) and the availability of paleoliquefaction data (Talwani and Schaeffer 2001), the applicant updated the EPRI characterization of the Charleston seismic source zone as part of the ESP application. SSAR Section 2.5.2.2.2.4 describes how the applicant used post-EPRI information to recharacterize the source geometry, Mmax, and magnitude recurrence for the Charleston seismic source zone. The applicant stated that it updated the Charleston seismic source zone using the guidelines provided in RG 1.165. Specifically, the applicant performed an SSHAC Level 2 study to incorporate current literature and data and the understanding of experts into an update of the Charleston seismic source model. The applicant referred to the updated model in the SSAR as the Updated Charlestown Seismic Source (UCSS) model. Bechtel (2006) describes the development of the UCSS model in greater detail.

UCSS Geometry. To represent the Charleston seismic source, the applicant developed four mutually exclusive source zone geometries. The applicant based the geometries of these four source zones, referred to as A, B, B', and C, on the following information:

- current understanding of geologic and tectonic features in the 1886 Charleston earthquake epicentral region
- the 1886 Charleston earthquake shaking intensity
- distribution of seismicity

- geographic distribution, age, and density of liquefaction features associated with both the 1886 and prehistoric earthquakes

SER Figure 2.5.2-2, reproduced from SSAR Figure 2.5.2-9, depicts the geometries of the applicant's four source zones. As shown in SER Figure 2.5.2-2, Geometry A is an approximately 100 x 50 km, northeast-oriented area centered on the 1886 Charleston meizoseismal area and envelops the following:

- the 1886 earthquake MMI X (severe damage) isoseismal (Bollinger 1977)
- the majority of identified Charleston-area tectonic features and inferred fault intersections
- the area of ongoing concentrated seismicity
- the area of greatest density for the 1886 and prehistoric liquefaction features

Based on the available geologic and seismologic evidence, the applicant concluded that Geometry A defines the area where future Charleston-type earthquakes will most likely occur. For this reason, the applicant assigned a weight of 0.70 to Geometry A in the UCSS model. However, in order to capture the uncertainty that future events may not be entirely restricted to Geometry A, the applicant developed three additional geometries, referred to as B, B', and C, that were each assigned a weight of 0.1.

As shown in SER Figure 2.5.2-2, Geometry B is a coast-parallel source, with an area of approximately 260 x 100 kilometers (161.6 x 62.1 miles), that incorporates all of Geometry A. The elongation and orientation of Geometry B roughly parallels both the regional structural grain as well as the elongation of the 1886 isoseismals (damage contours). Paleoliquefaction features mapped by Amick (1990), Amick et al. (1990a, 1990b), and Talwani and Schaeffer (2001) define the northeastern and southwestern extents of Geometry B. In addition, Geometry B extends to the southeast to include the offshore Helena Banks fault zone; offshore earthquakes in 2002 (mb 3.5 and 4.4) suggest a possible spatial association with the mapped trace of the Helena Banks fault zone. Multiple reflection profiles clearly show the Helena Banks fault, which demonstrates late Miocene (23.8 to 5.3 million years ago (mya)) offset (Behrendt and Yuan 1987).

Geometry B' is an approximately 260 x 50-km (161.6 x 31.1-mi) source area that is identical to Geometry B with the exception that Geometry B' does not include the offshore Helena Banks fault system. The applicant excluded the Helena Banks fault system from Geometry B' because the majority of data and evaluations (e.g., Behrendt and Yuan 1987) suggest that this fault system is no longer active.

Geometry C is an approximately 200 x 30-km (124.3 x 18.6-mi), north-northeast-oriented source area that envelops the southern segment of the ECFS as depicted by Marple and Talwani (2000). Both the U. S. Geological Survey (USGS) hazard model (Frankel et al. 2002) and the SCDOT hazard model (Chapman and Talwani 2002) explicitly incorporate the southern segment of the ECFS as a source zone. However, the USGS hazard model (Frankel et al. 2002) truncated the northern extent of the southern fault segment, while the SCDOT hazard model (Chapman and Talwani 2002) extended the southern segment to include, in part, the liquefaction features in southeastern South Carolina (Chapman 2005). The applicant concluded that the liquefaction features in southeastern South Carolina are captured in source zones B and B'. The applicant further concluded that the truncation of the northern extent of the southern fault segment of the ECFS in the USGS hazard model is not supported by any available data.

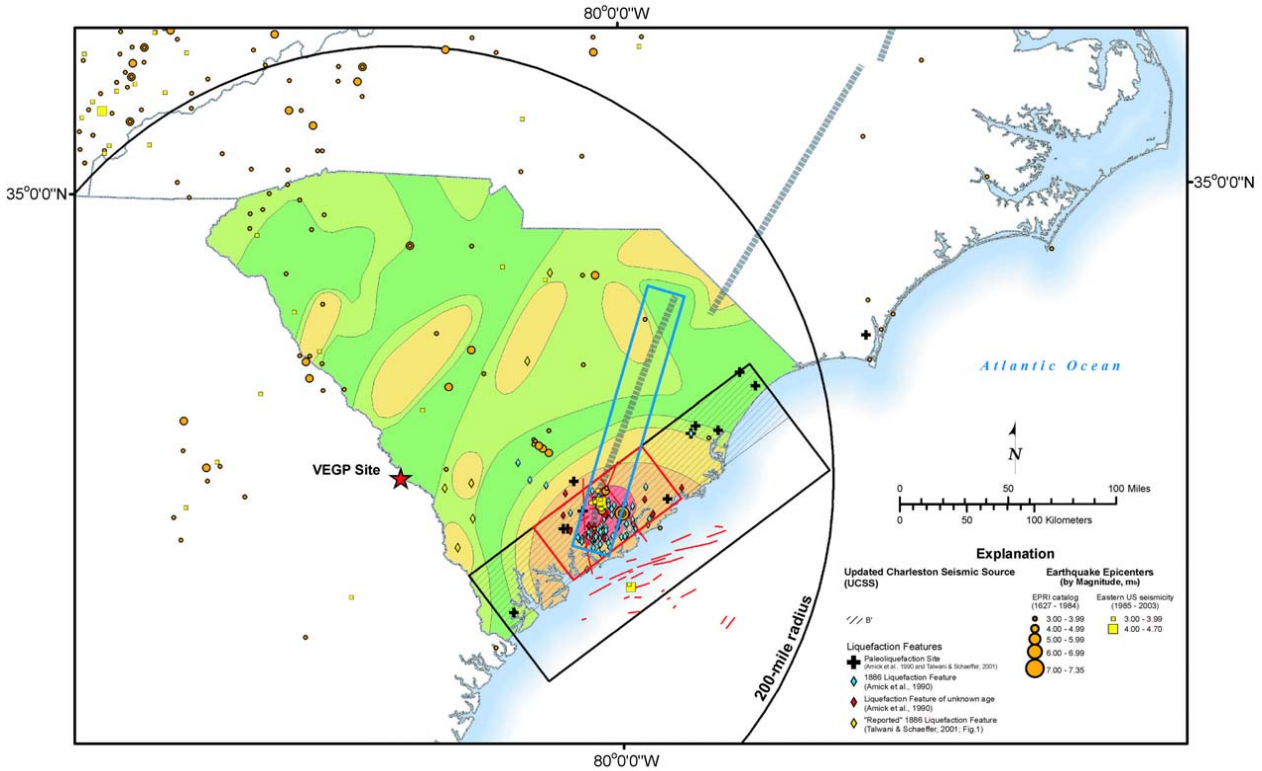


Figure 2.5.2-2 - Alternative geometries comprising the UCSS model updated Charleston seismic source (reproduced from SSAR Figure 2.5.2-9)

UCSS Maximum Magnitude. In order to define the largest earthquake that could be produced by the Charleston seismic source, the applicant stated that it developed a distribution for Mmax based on several post-EPRI (1989) magnitude estimates for the 1886 Charleston earthquake. The applicant modified the USGS hazard model magnitude distribution (Frankel et al. 2002), shown in SER Table 2.5.2-2, to include a total of five discrete magnitude values, each separated by 0.2 M units. The applicant's Mmax distribution included a discrete value of M 6.9 to represent the Bakun and Hopper (2004) best estimate of the 1886 Charleston earthquake magnitude, as well as a lower value of M 6.7 to capture the probability that the 1886 earthquake was smaller than the Bakun and Hopper (2004) mean estimate of M 6.9. In their study, Bakun and Hopper (2004) provide a 2-sigma range of M 6.4 to M 7.2.

Table 2.5.2-2 - Comparison of Maximum Magnitudes and Weights for the USGS and SCDOT Models with the Applicant's UCSS Model

Mmax (M)	USGS Model Weight	SCDOT Model Weight	UCSS Model Weight
6.7	—	—	0.1
6.8	0.2	—	—
6.9	—	—	0.25
7.1	0.2	0.2	0.3
7.3	0.45	0.6	0.25
7.5	0.15	0.2	0.1

UCSS Recurrence Model. Most of the available geologic data pertaining to the recurrence of large earthquakes in the South Carolina region were published after 1990. In the absence of these data, the 1989 EPRI study (EPRI NP-6395-D) estimated the recurrence of large Charleston-type earthquakes using a truncated exponential model. The 1989 EPRI study estimated the parameters of this exponential model from historical seismicity. The recurrence of Mmax earthquakes in the EPRI study was on the order of several thousand years, which is significantly greater than more recently published estimates of about 500 to 600 years that are based on paleoliquefaction data (Talwani and Schaeffer 2001).

To estimate recurrence for earthquakes with **M** less than 6.7, the applicant used an exponential magnitude distribution. The applicant estimated the parameters of this exponential distribution from the earthquake catalog. However, based on paleoliquefaction data, the applicant found that Mmax earthquakes (**M** greater than 6.7) have occurred more frequently than would be implied by extrapolation of the recurrence of smaller magnitude (**M** less than 6.7) earthquakes within the UCSS. Thus, the applicant treated Mmax events within the UCSS according to a characteristic earthquake model, which means that this source repeatedly generates earthquakes, known as characteristic earthquakes, similar in size to Mmax. The applicant estimated the recurrence of these characteristic earthquakes from paleoliquefaction data.

The applicant stated that it further reevaluated the data presented by Talwani and Schaeffer (2001) and provided an updated estimate of earthquake recurrence. Talwani and Schaeffer (2001) used calibrated radiocarbon ages with 1-sigma error bands to define the timing of past liquefaction episodes in coastal South Carolina. However, the standard practice in paleoliquefaction studies is to use calibrated ages with 2-sigma error bands (e.g., Sieh et al.

1989; Grant and Sieh 1994; Tuttle 2001) to more accurately reflect uncertainties associated with radiocarbon dating. The applicant determined that the use of 1-sigma error bands by Talwani and Shaeffer (2001) may lead to overinterpretation of the paleoliquefaction record such that more episodes are interpreted than actually occurred. For this reason, the applicant recalibrated the radiocarbon ages presented in Talwani and Schaeffer (2001) and reported the newly recalibrated ages with 2-sigma error bands.

The applicant identified six individual paleoearthquakes, including the 1886 Charleston event, from the UCSS calibrated 2-sigma data. The applicant determined that two earthquake events (C and D) identified in the Talwani and Schaeffer (2001) 1-sigma analysis are not individually distinguishable at the 95 percent (2-sigma) confidence interval, and the applicant defined these two events as a single event, C'. The applicant also suggested that Talwani and Schaeffer (2001) events F and G likely represent a single large event, defined by the applicant as event F'. The applicant interpreted the six large paleoearthquakes (1886, A, B, C', E, and F') to represent Charleston-type events that occurred within the past ~5000 years. Furthermore, the applicant determined that results of the 2-sigma analysis suggest there have been four large earthquakes in the most recent ~2000-year (yr) portion of the earthquake record (1886, A, B, and C').

The applicant calculated two different average recurrence intervals, which represent two recurrence branches on the logic tree shown in SSAR Figure 2.5.2-11. The first average recurrence interval is based on the four events (1886, A, B, and C') that the applicant interpreted to have occurred within the past ~2000 years. The applicant concluded that this time period represents a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of the expert elicitation. The applicant assigned a weight of 0.8 to the logic tree branch representing the recurrence interval calculated for the 2000-yr record. The second average recurrence interval is based on events that the applicant interpreted to have occurred within the past ~5000 years and includes events 1886, A B, C', E, and F'. This time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). Published papers and researchers questioned suggest that the older part of the record (i.e., older than ~2000 years) may be incomplete. The applicant noted, however, that it may also be possible that the older record is complete but exhibits longer inter-event times. For this reason, the applicant assigned a weight of 0.2 to the logic tree branch representing the recurrence interval calculated for the 5000-yr record. The applicant indicated that the 0.80 and 0.20 weighting of the ~2000-yr and 5000-yr paleoliquefaction records, respectively, reflect the incomplete knowledge of both the short- and long-term recurrence behavior of the Charleston source.

The applicant used the methods of Savage (1991) and Cramer (2001) to calculate the mean recurrence interval for both the ~2000-yr and ~5000-yr records. According to the applicant, these methods describe the mean recurrence interval with best estimate mean Tave and an uncertainty described as a lognormal distribution with median T0.5 and parametric lognormal shape factor σ0.5. The average recurrence interval for the ~2000-yr record, based on the three most recent inter-event times (1886–A, A–B, B–C'), has a best estimate mean value of 548 years and an uncertainty distribution described by a median value of 531 years and a lognormal shape factor of 0.25. The average recurrence interval for the ~5000-yr record, based on five inter-event times (1886–A, A–B, B–C', C'–E, E–F'), has a best estimate mean value of

958 years and an uncertainty distribution described by a median value of 841 years and a lognormal shape factor of 0.51.

The applicant modeled earthquakes in the exponential part of the distribution as point sources uniformly distributed within the source area, with a constant depth fixed at 10 kilometers. For the characteristic model, the applicant represented source zone Geometries A, B, B', and C by a series of closely spaced, vertical, northeast-trending faults parallel to the long axis of each source zone.

2.5.2.2.3 Correlation of Earthquake Activity with Seismic Sources

SSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI seismic source model. The applicant compared the distribution of earthquake epicenters from both the original EPRI historical catalog (1627–1984) and the updated seismicity catalog (1985–2005) with the seismic sources characterized by each of the EPRI ESTs. Based on this comparison, the applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure. In addition, it concluded that there are no clusters of seismicity that would suggest a new seismic source not captured by the EPRI seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require significant revision to the geometry of any of the EPRI seismic sources. The applicant further stated that the updated catalog does not show or suggest an increase in Mmax or a significant change in seismicity parameters (activity rate, b-value) for any of the EPRI seismic sources.

2.5.2.2.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

SSAR Section 2.5.2.4 presents the results of the applicant's PSHA for the ESP site. PSHA is an acceptable method to estimate the likelihood of earthquake ground motions occurring at a site (RG 1.165 and RG 1.208). The hazard curves generated by the applicant's PSHA represent generic hard rock conditions (characterized by a shear- (S-) wave velocity of 9200 feet per second (ft/s)). In SSAR Section 2.5.2.4, the applicant also described the earthquake potential for the site in terms of the most likely earthquake magnitudes and source-site distances, which are referred to as controlling earthquakes. The applicant determined the low-and high-frequency controlling earthquakes by deaggregating the PSHA at selected probability levels. Before determining the controlling earthquakes, the applicant updated the original 1989 EPRI PSHA (EPRI NP-6395 1989) using the seismic source zone adjustments, described in SER Section 2.5.2.1.2, and the new ground motion models described below.

PSHA Inputs

Before performing the PSHA, the applicant updated the original 1989 EPRI PSHA inputs using the seismic source zone adjustments described in SSAR Section 2.5.2.2. In addition, the applicant used the updated 2004 EPRI (EPRI 1009684) ground motion models instead of the EPRI NP-6395-D (1989) ground motion models, which were used in the original 1989 EPRI PSHA.

Seismic Source Model

To update the original EPRI model, the applicant removed all of the sources identified as a Charleston source from each of the six EPRI EST models. SER Table 2.5.2-1 lists these sources. The applicant then incorporated its four UCSS alternative source geometries, Mmax, and recurrence distributions into each of the six EST models. The applicant explained that in most cases, this involved replacing a single Charleston source with four alternative Charleston sources.

The applicant used an exponential magnitude distribution to model smaller earthquakes (**M** less than 6.7) within the UCSS. To calculate the activity rate and b-value for this distribution, the applicant used the same methodology and smoothing assumptions that were used in the 1989 EPRI study. However, the applicant calculated these seismicity parameters using the new geometries of the UCSS along with the updated seismicity catalog (through April 2005). Because old and new source geometries are not coincident, the applicant allowed the portions of "old" EPRI sources that fell outside of the new UCSS source geometries to default to the existing EPRI background sources. According to the applicant, this ensured that no areas in the seismic hazard model were aseismic. For the unmodified sources of the 1989 EPRI PSHA, the applicant used the original seismicity rates from the 1988 EPRI (EPRI NP-4726-A 1988) earthquake catalog (through 1984) in its seismic hazard calculations.

To determine whether the seismicity rates used in the 1989 EPRI PSHA (EPRI NP-6395-D 1989) are appropriate for the assessment of the seismic hazard at the ESP site, the applicant assessed seismicity rates for two sources in the site region: 1) a small rectangular source around the Charleston seismicity; and (2) a triangular-shaped source representing seismicity in South Carolina and a strip of Georgia that incorporates the ESP site. The applicant stated that it selected these sources because they contribute the most to the seismic hazard at the ESP site.

The applicant investigated the seismicity rates in the two sources by running the program EQPARAM (from the EPRI EQHAZARD package) first for the original EPRI catalog and then for the updated EPRI catalog (through April 2005). The applicant used the a- and b-values obtained from EQPARAM to calculate the recurrence rates for different earthquake magnitudes. For the rectangular Charleston source, the applicant concluded that the seismicity rates remain the same when the seismicity from 1985 to April 2005 is added. For the triangular South Carolina source, the applicant concluded that the seismicity rates decrease when the seismicity from 1985 to April 2005 is added.

The applicant concluded that the seismicity recorded since 1984 does not indicate that seismic activity rates have increased in those sources contributing most to the hazard at the ESP site, under the assumptions of the 1989 EPRI PSHA. Based on the review of geological and seismological data published since the 1986 EPRI Project (EPRI NP-4726), presented in SSAR Section 2.5.2, the applicant concluded that, with the exception of the Charleston seismic source, there are no significant changes to the original EPRI Mmax values. SSAR Section 2.5.2.2.2 discusses the applicant's modifications to Mmax for the Charleston seismic source.

Ground Motion Models

The applicant used the ground-motion models developed by the 2004 EPRI-sponsored study (EPRI 1009684 2004) for the updated PSHA. For general area sources, the applicant combined 9 estimates of median ground motion with 4 estimates of aleatory uncertainty, which resulted in 36 combinations. For fault sources in rifted regions (which apply to the East Coast Fault System [ECFS] fault segments), the applicant combined 12 estimates of median ground motion with four estimates of aleatory uncertainty, resulting in 48 combinations.

The applicant compared the EPRI NP-6395 (1989) ground motion model with the EPRI 1009684 (2004) ground motion models. The differences between the two models are a function of magnitude, distance, and structural frequency. The applicant stated that in general, the median ground-motion amplitudes are similar at high frequencies. At low frequencies, the EPRI 1009684 (2004) models show lower median ground motions because these models incorporate the possibility of a double-corner source model. However, the applicant stated that the EPRI 1009684 standard deviations are universally higher than those of EPRI NP-6395.

PSHA Methodology and Calculation

For the PSHA calculation, the applicant used the Risk Engineering, Inc. FRISK88 seismic hazard code. The applicant first performed a PSHA using the original 1989 EPRI primary seismic sources and ground-motion models in order to validate FRISK88 against the EPRI software EQHAZARD. The applicant compared the results from FRISK88 with the original EPRI hard rock results. The applicant determined that a comparison of the mean hazard curves for peak ground acceleration (PGA) generally agrees to within 5.1 percent for amplitudes up to 1 g.

Using the updated EPRI seismic source characteristics and new ground-motion models as inputs, the applicant performed PSHA calculations for PGA and spectral acceleration at frequencies of 25, 10, 5, 2.5, 1, and 0.5 hertz (Hz). Following the guidance provided in RG 1.165, the applicant performed PSHA calculations assuming generic hard rock site conditions (i.e., an S-wave velocity of 9200 ft/s). The applicant incorporated the effects of the ESP site geology into its calculation of the SSE spectrum, which uses the hard rock PSHA results as a starting point.

PSHA Results

To determine the low- and high-frequency controlling earthquakes for the ESP site, the applicant followed the procedure outlined in Appendix C to RG 1.165. This procedure involves the deaggregation of the PSHA results at a target probability level to determine the controlling earthquake in terms of a magnitude and source-to-site distance. The applicant chose to perform the deaggregation of the mean 10^{-4}, 10^{-5}, and 10^{-6} PSHA hazard results. SER Figure 2.5.2-3 shows the results of the applicant's high-frequency (5 to 10 Hz) 10^{-4} hazard deaggregation, while SER Figure 2.5.2-4 shows the results of the low-frequency (1 to 2.5 Hz) 10^{-4} hazard deaggregation. The staff did not show the applicant's deaggregation plots for the 10^{-5} and 10^{-6} mean hazard levels because of their similarity to the 10^{-4} deaggregation plot shown in SER Figures 2.5.2-3 and 2.5.2-4.

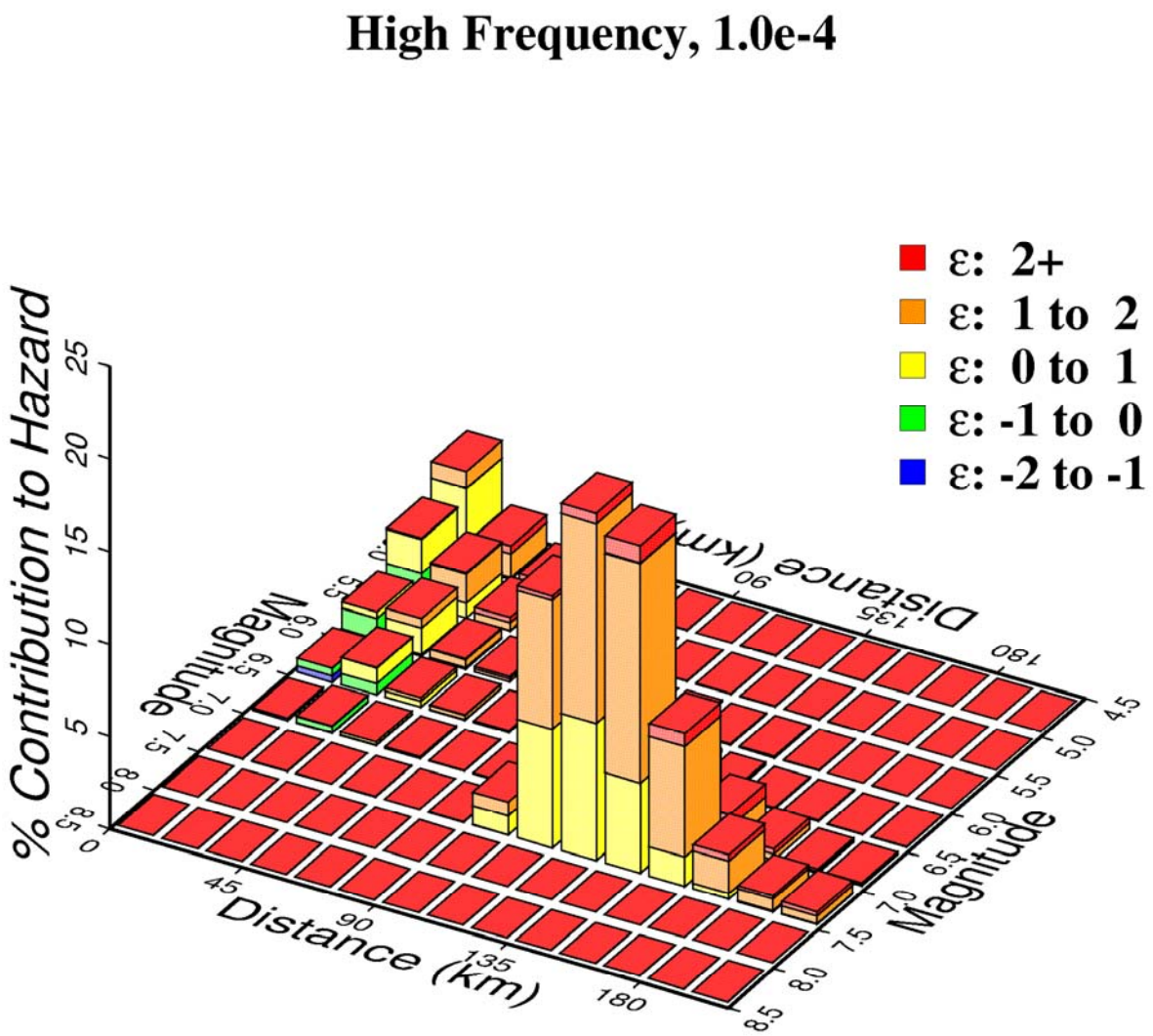


Figure 2.5.2-3 - High-frequency (5 to 10 Hz) 10^{-4} hazard deaggregation (reproduced from SSAR Figure 2.5.2-22)

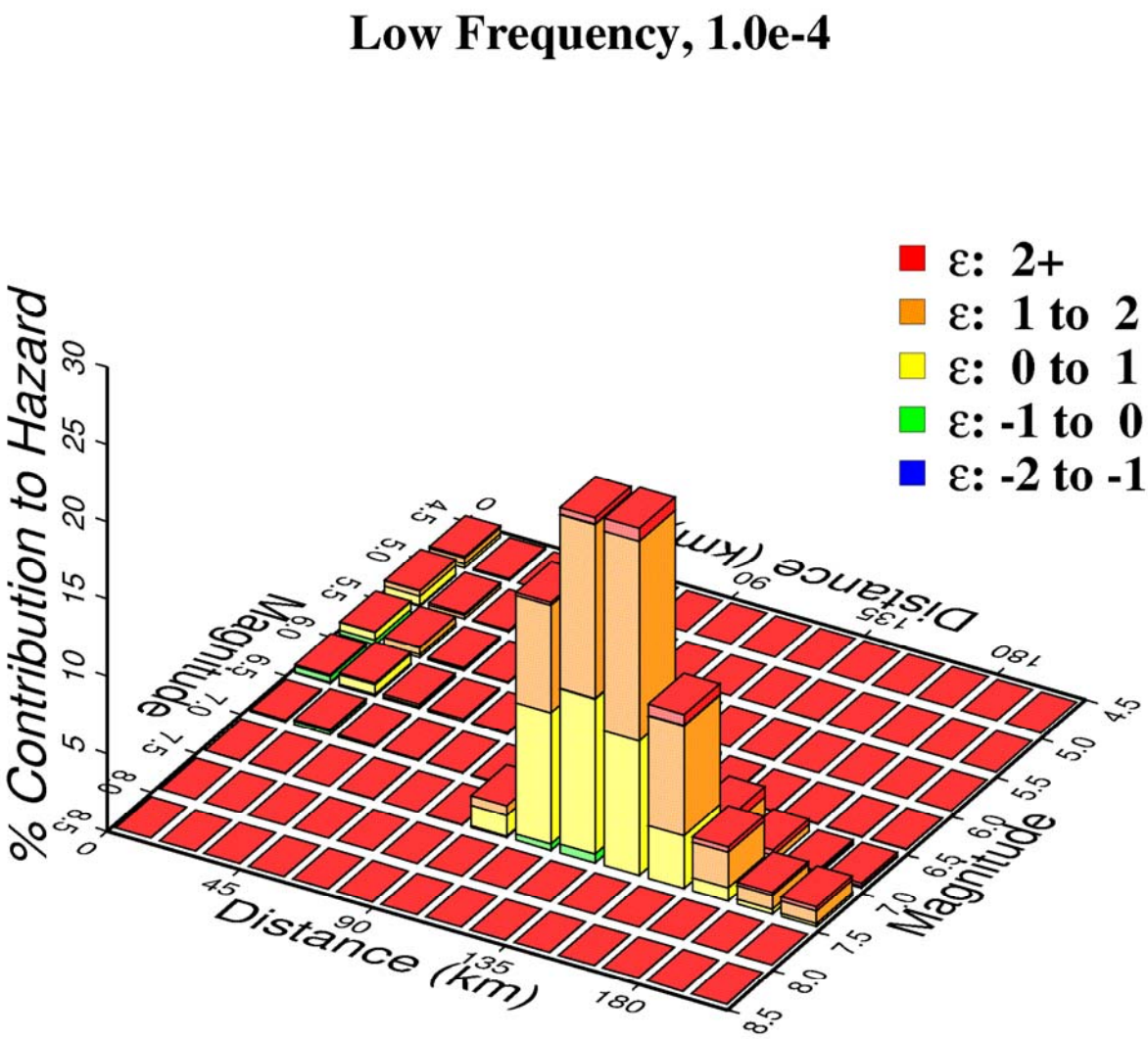


Figure 2.5.2-4 - Low-frequency (1 to 2.5 Hz) 10^{-4} hazard deaggregation (reproduced from SSAR Figure 2.5.2-23)

Because of the similarity of the mean magnitude (Mbar) and mean distance (Dbar) values for the three hazard levels, the applicant selected a single Mbar and Dbar value for each frequency range. SER Table 2.5.2-3 provides the Mbar and Dbar values for the high- and low-frequency controlling earthquakes corresponding to the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. SER Table 2.5.2-3 also provides the applicant's final Mbar and Dbar values for the high- and low-frequency controlling earthquakes. For the high-frequency mean 10^{-4}, 10^{-5}, and 10^{-6} hazard, the controlling earthquake, based on the final Mbar and Dbar pair, is an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles), corresponding to an earthquake from a local seismic source zone. For the low-frequency mean 10^{-4}, 10^{-5}, and 10^{-6} hazard, the controlling earthquake is an **M** 7.2 event and occurs at a distance of 130 kilometers (80.8 miles). This earthquake corresponds to an event in the Charleston seismic zone.

Table 2.5.2-3 - Computed and Final Mbar and Dbar Values Used for Development of High- and Low-Frequency Target Spectra (Based on the Information Provided in SSAR Table 2.5.2-17)

High Frequency (5 to 10 Hz)				
Mean Hazard Level	10^{-4}	10^{-5}	10^{-6}	Final Values
Mbar (M)	5.5	5.6	5.6	5.6
Dbar	17.7 km (11 mi)	11.5 km (7.1 mi)	9.1 km (5.7 mi)	12 km (7.5 mi)
Low Frequency (1 to 2.5 Hz)				
Mean Hazard Level	10^{-4}	10^{-5}	10^{-6}	Final Values
Mbar (M)	7.2	7.2	7.2	7.2
Dbar	136.5 km (84.8 mi)	134.3 km (83.5 mi)	132.9 km (82.6 mi)	130 km (80.8 mi)

2.5.2.2.5 Seismic Wave Transmission Characteristics of the Site

SSAR Section 2.5.2.5 describes the method used by the applicant to develop the site free-field soil uniform hazard response spectrum (UHRS). The hazard curves generated by the PSHA are defined for generic hard rock conditions (characterized by an S-wave velocity of 9200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 2000 feet below the ground surface at the ESP site. To determine the soil UHRS, the applicant: (1) developed soil/rock profile models for the ESP site; (2) selected seed earthquake time histories; and (3) performed the final site response analysis.

Site Response Model

According to the applicant, the soil profile to a depth of approximately 1049 feet at the ESP site consists of approximately 86 feet of predominantly sands, silty sands, and clayey sands, with occasional clay seams, referred to as the Upper Sand Stratum (Barnwell Group). At the base of this sand unit is a Shelly Limestone (Utley Limestone), which is characterized by solution channels, cracks, and discontinuities. Beneath the Utley limestone is the Blue Bluff Marl (Lisbon Formation), consisting of approximately 64 feet of slightly sandy, cemented calcareous clay. The Blue Bluff Marl is underlain by approximately 900 feet of fine-to-coarse sand with interbedded silty clay and clayey silt, referred to as the Lower Sand Stratum. The Lower Sand Stratum comprises the Still Branch, Congaree, Snapp, Black Mingo, Steel Creek, Gaillard/Black Creek, Pio Nono, and Cape Fear formations.

According to the applicant, the rock profile at the ESP site, below approximately 1049 feet, consists of the Dunbarton Triassic (206–24 mya) basin followed by Paleozoic (543–248 mya) crystalline rock. The Dunbarton Triassic basin rock comprises red sandstone, breccia, and

mudstone and is characterized by a weathered zone in the upper 120 feet. The Paleozoic crystalline basement is characterized by a high S-wave velocity (greater than 9200 ft/s). The Pen Branch fault forms the boundary between the Dunbarton Triassic basin and the Paleozoic basement rock. As described in SSAR Section 2.5.1, the Pen Branch fault dips to the southeast at an angle of 45 degrees below the ESP site.

The soil/rock profile model used by the applicant for its site response analysis is shown in SSAR Figure 2.5.4-7 and SSAR Table 2.5.4-11. The uppermost competent in-situ layer is the Blue Bluff Marl, which is encountered at a depth of 86 feet and characterized by an average S-wave velocity of 2354 ft/sec. Note that SSAR Figure 2.5.4-7 and SSAR Table 2.5.4-11 do not show the Barnwell Group and Utley Limestone. The applicant intends to remove the incompetent Barnwell Group (and the underlying Utley Limestone) because it is susceptible to liquefaction and dissolution-related ground deformation. Furthermore, its S-wave velocity is generally below 1000 ft/s. Thus, in its site response calculations, the applicant assumes that these layers have been replaced with 86 feet of structural backfill.

SSAR Figure 2.5.4-7 shows S-wave velocities for each of the different soil and rock layers to a maximum depth of 2275 feet. The applicant based this S-wave velocity profile on the results of suspension primary and secondary (P-S) velocity and seismic cone penetrometer tests (CPTs) performed at the ESP site, as well as deep borehole S-wave velocity data from the Savannah River Site (SRS 2005). The applicant did not determine S-wave velocity for the compacted backfill as part of the ESP subsurface investigation. Instead, the applicant relied on data for existing Units 1 and 2. To represent the variability of the depth to the top of the Paleozoic crystalline basement, where the S-wave velocity is at least 9200 ft/s, the applicant developed six alternative site response profiles, which are provided in Part B of SER Table 2.5.4-11. For the six alternative profiles, the depth to the top of the Paleozoic crystalline rock ranged from 1525 feet to 2275 feet. According to the applicant, the six alternative site response profiles also accounted for the uncertainty of the S-wave velocity gradient between the top of the unweathered section of the Dunbarton Triassic basin to the top of the Paleozoic crystalline rock. In its site response model, the applicant used the PSHA rock motions at the top of the Paleozoic crystalline rock as input.

The applicant collected additional S-wave velocity data as part of the COL site investigation. This data is described in detail in SSAR Section 2.5.4.4 and is referred to as “COL” data by the applicant. The applicant used the SASW (Spectral Analysis of Surface Waves) and cross-hole methods, and the results of Resonant Column and Torsional Shear (RCTS) tests to determine the S-wave velocity of the proposed backfill. The applicant also determined the S-wave velocity of the Blue Bluff Marl and the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum using down-hole seismic CPT tests and suspension P-S velocity tests, combined these data with two ESP profiles (located in the powerblock area of Units 3 and 4) and averaged the results. The applicant then developed an S-wave velocity profile for soil (i.e. to a depth of 1059 ft). The resulting S-wave velocity profile is presented in SSAR Table 2.5.4-11a and SSAR Figure 2.5.4-7a. Because the COL S-wave velocity measurements only extended to a maximum depth of 420 feet below ground surface, the applicant incorporated the S-wave velocity data from the ESP profile (provided in SSAR Table 2.5.4-11 and SSAR Figure 2.5.4-7) below this depth.

The applicant did not use the additional COL S-wave velocity profile as input to its site response calculations. Instead, the applicant provided justification that the use of only the ESP S-wave velocity profile is adequate. In SSAR Section 2.5.4.7.5, the applicant presented a comparison of the ESP and COL S-wave velocity profiles. Based on the comparison of the two S-wave velocity profiles shown in SSAR Figure 2.5.4-7a, the applicant concluded that there is good agreement between the two data sets. Furthermore, based on the results of site response sensitivity studies presented in SSAR Section 2.5.2.9, the applicant concluded that the difference in the amplification between the ESP and COL data is small.

The strain-dependent shear modulus and damping relationships used by the applicant for the soil units at the ESP site are based on EPRI TR-102293 (1993). The applicant also used the strain-dependent shear modulus and damping relationships developed for the nearby SRS by Lee (1996). For the Dunbarton Triassic basin and Paleozoic crystalline rocks, the applicant assumed linear behavior during earthquake shaking with 1-percent damping.

As part of the COL site investigation, the applicant also developed strain-dependent shear modulus and damping relationships based on RCTS tests performed on compacted backfill, Blue Bluff Marl, and Lower Sand samples. The resulting site-specific shear modulus reduction curves are provided in SSAR Table 2.5.4-12a and SSAR Figure 2.5.4-9a, while the site specific damping curves are provided in SSAR Table 2.5.4-12a and SSAR Figure 2.5.4-11a. Although the applicant relied only on the generic EPRI and SRS strain-dependent shear modulus and damping relationships as input to its site response calculations, the applicant presented a comparison with the site-specific relationships in SSAR Figures 2.5.4-19a through 2.5.4-20c. Specifically, SSAR Figures 2.5.4-19a, 19b, and 19c compare the normalized shear modulus reduction versus shear strain curves for the compacted backfill, Blue Bluff Marl, and Lower Sands, respectively. SSAR Figures 2.5.4-20a, 20b, and 20c compare damping versus shear strain for the same units. In SSAR Section 2.5.4.7.5, the applicant stated that generally, the figures suggest that the subsurface soils behave more linearly (i.e. provide a smaller reduction in shear modulus and less damping) than both the generic EPRI and SRS relationships. However, the applicant's site response sensitivity studies, described in SSAR Section 2.5.2.9, resulted in small differences in amplification between the ESP and COL data.

The applicant stated that once it determined the appropriate soil and rock dynamic properties, it modeled the variability present in the site data by randomizing the soil and rock S-wave velocity profiles, soil shear modulus reduction and damping relationships, and rock-damping values. For each family of degradation curves (i.e., EPRI or SRS), the applicant generated 60 randomized soil/rock profiles to account for the variability in the site properties. The applicant generated the 60 randomized soil/rock profiles using the stochastic model described in EPRI TR-102293 (1993) and Toro (1996). Inputs to the applicant's stochastic model include the base-case soil and rock profiles provided in SSAR Table 2.5.4-11, as well as the depth to bedrock, which the applicant randomized to account for the range of depths associated with the Pen Branch fault. For each randomized velocity profile, the applicant developed one set of randomized shear modulus reduction and damping curves from the EPRI family of curves and another set from the SRS family of curves.

To account for the variability in soil shear strain modulus and material-damping ratio with shearing strain amplitude, the applicant randomized the shear modulus reduction and damping

curves used for the site response analysis. For each of the randomized velocity profiles, the applicant developed one set of randomized shear modulus reduction and damping curves for each family of degradation curve (i.e., EPRI or SRS). Inputs to the applicant's model include the base-case shear modulus reduction and damping curves provided in SSAR Tables 2.5.4-12 and 2.5.4-13 and shown in SSAR Figures 2.5.4-9 to 2.5.4-12. The applicant stated that it also accounted for the uncertainty in damping ratio for the Dunbarton Triassic basin rock, which is represented by a 5- to 95-percentile range of 0.7 to 1.5 percent.

Site Response Input Time Histories

The applicant developed target spectra for two different frequency ranges, high-frequency (5 to 10 Hz) and low-frequency (1 to 2.5 Hz), as defined in RG 1.165. These high- and low-frequency target response spectra represent the Mbar and Dbar values from the deaggregation of the 10^{-4}, 10^{-5}, and 10^{-6} hazard curves. For the high-frequency cases, the applicant considered only those sources within 105 kilometers of the site to compute the Mbar and Dbar values. To compute the low-frequency Mbar and Dbar values, the applicant only considered sources at distances greater than 105 kilometers from the site. The applicant noted that this distinction was made based on the dominance of the Charleston source for low frequencies and long return periods.

Because of the similarity of the calculated Mbar and Dbar values for the three hazard levels, the applicant selected a single Mbar and Dbar pair to represent the high-frequency controlling earthquake and a single Mbar and Dbar pair to represent the low-frequency controlling earthquake. SER Table 2.5.2-3 provides the final Mbar and Dbar values used for the development of the high- and low-frequency target spectra.

Using the final high- and low-frequency Mbar and Dbar values, described above, the applicant developed target response spectra using the log-average of the single and double corner CEUS spectral shape models of NUREG/CR-6728 (Technical Basis for Revision of Regulatory Guidance of Design Ground Motions: Hazard- and Risk- Consistent Ground Motion Spectra Guidelines). The applicant scaled the low-frequency spectral shape to the corresponding UHRS (i.e., 10^{-4}, 10^{-5} or 10^{-6}) at 1.75 and scaled the high-frequency spectral shape to the corresponding UHRS at 7.5 Hz. SER Figure 2.5.2-5 shows the resulting high- and low-frequency target response spectra for the 10^{-4} mean hazard level. The applicant also developed target response spectra for the 10^{-5} and 10^{-6} hazard levels.

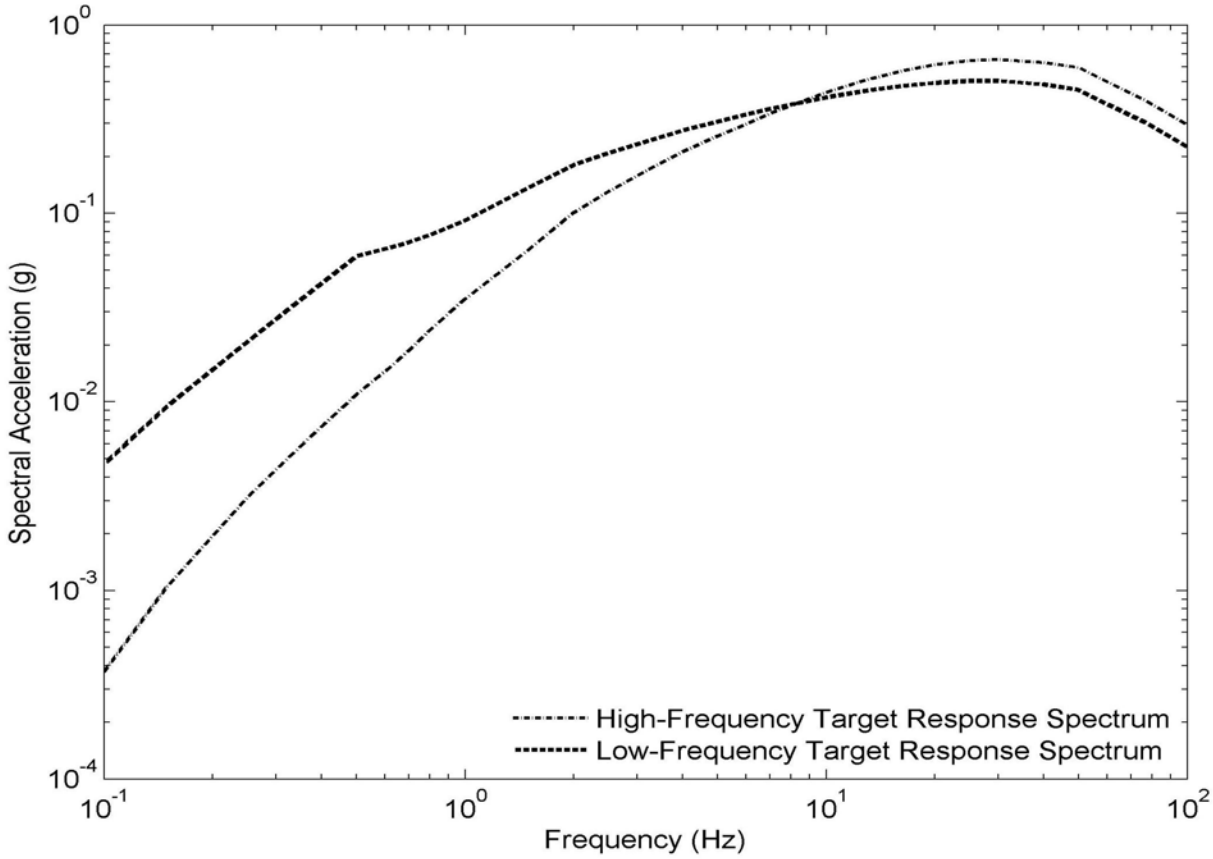


Figure 2.5.2-5 - Low- and high-frequency target response spectra representing the 10^{-4} hazard level (based on the information provided in SSAR Tables 2.5.2-20a, and 2.5.2-20b).

To determine the ESP dynamic site response, the applicant spectrally matched a suite of acceleration time histories to the six target response spectra described above. The applicant selected strong motion acceleration time histories that were recorded at rock-site locations in the Western United States (WUS), Eastern Canada, Turkey, and Japan. Specifically, the applicant selected time histories recorded at sites characterized by S-wave velocities greater than 600 meters per second (m/s) (1968.5 ft/s) in the upper 30 meters (98.4 feet) and similar magnitudes and distances to the final high- and low-frequency Mbar and Dbar values.

The applicant spectrally matched a total of 30 seed time histories to the low-frequency target response spectra corresponding to the 10^{-4}, 10^{-5}, and 10^{-6} mean hazard levels. The applicant spectrally matched a different group of 30 seed time histories to the high-frequency target response spectra representing the 10^{-4}, 10^{-5}, and 10^{-6} mean hazard levels. The applicant used the spectral matching criteria recommended in NUREG/CR-6728 to check the average spectrum from the 30 spectrally matched time histories for a given frequency range and mean hazard level.

Site Response Methodology and Calculation

To determine the final site response, the applicant used the program SHAKE to compute the site amplification functions (AFs) for each of the spectrally matched time histories. As shown in SER Table 2.5.2-4, for each hazard level (10^{-4}, 10^{-5}, and 10^{-6}) and for each deaggregation earthquake (high- and low-frequency), the applicant paired the 60 randomized soil profiles corresponding to the EPRI curves and the 60 randomized soil profiles representing the SRS curves with the 30 spectrally matched time histories. The applicant applied each time history to two of the randomized soil/rock profiles, which resulted in a total of 240 AFs for each of the three mean hazard levels.

Table 2.5.2-4 - Site Response Analyses Performed (Based on the Information Provided in SSAR Table 2.5.2-19)

Mean Hazard Level	10^{-4}		10^{-5}		10^{-6}		Total Number of Analyses
Deaggregation Earthquake	High Freq.	Low Freq.	High Freq.	Low Freq.	High Freq.	Low Freq.	
Number of Input Time Histories	30	30	30	30	30	30	
Number of Randomized Soil Profiles (EPRI)	60	60	60	60	60	60	360
Number of Randomized Soil Profiles (SRS)	60	60	60	60	60	60	360
							720

Site Response Results

To obtain the final site AFs, the applicant divided the output response spectrum (defined at the top of the backfill) by the hard rock input response spectrum for each of the cases shown in SER Table 2.5.2-4. For the 10^{-4} mean hazard level, the applicant computed the mean of the 60 individual AFs corresponding to the high-frequency input time histories and the EPRI-based randomized soil profiles. The applicant repeated this process for the SRS-based randomized soil profiles. The applicant's final high-frequency AF (shown in the lower plot of SER Figure 2.5.2-6) corresponds to the mean of these two results. The applicant developed the final low-frequency AF in a similar manner and this is also shown in SER Figure 2.5.2-6 (upper plot). According to the applicant's results, the ESP site subsurface amplifies the high-frequency input hard rock motion over the fairly wide frequency range of 0.1 to ~25 Hz, with the maximum amplification of 3.8 at a frequency of 0.6 Hz. The applicant's results also show that the low-frequency input hard rock motion is amplified over the frequency range of 0.1 to ~20 Hz, with the maximum amplification of 4.0 at a frequency of 0.6 Hz.

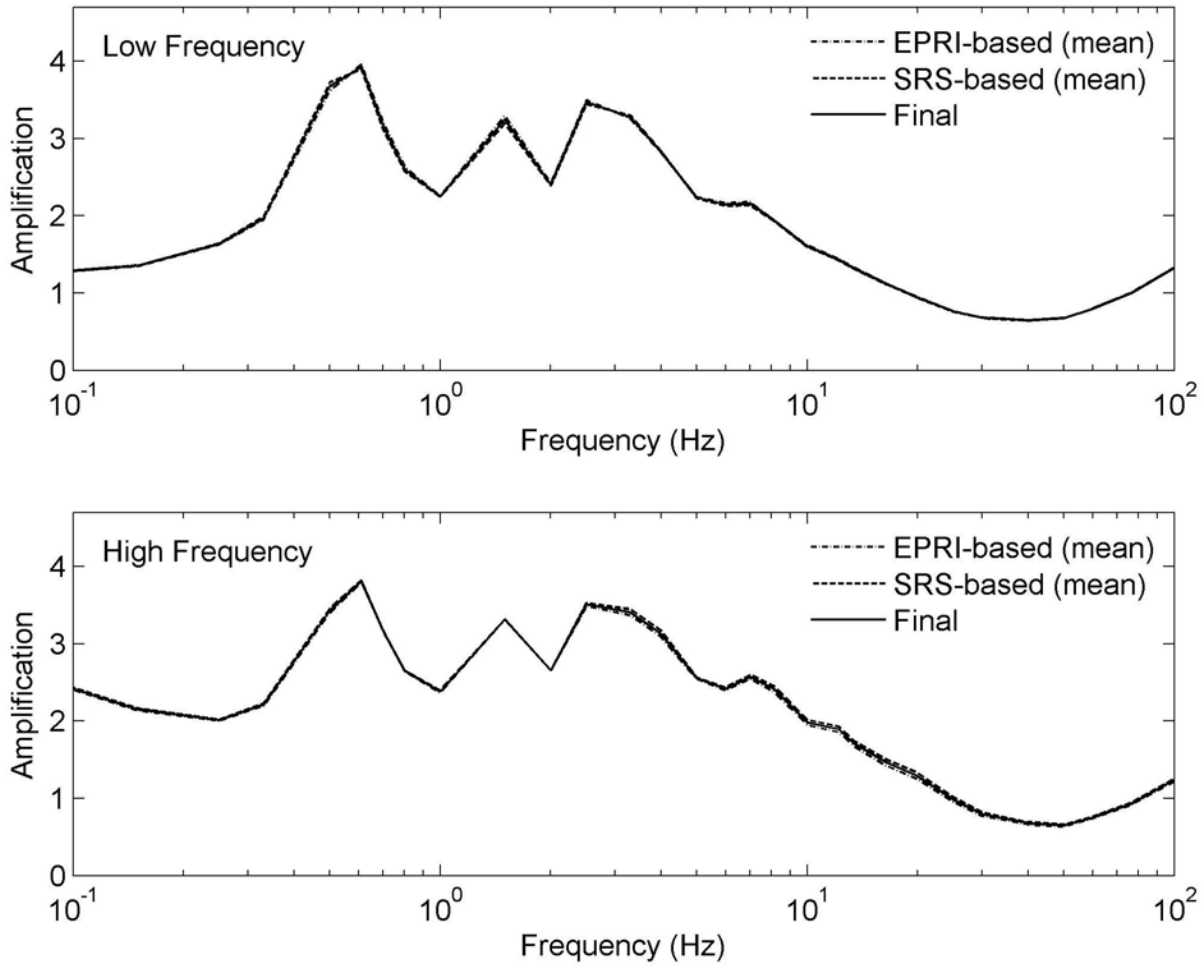


Figure 2.5.2-6 - Final EPRI and SRS high- and low-frequency AFs for the 10^{-4} hazard level (based on the information provided in SSAR Tables 2.5.2-20e and 2.5.2-20f)

The applicant determined the final 10-4 soil surface spectrum for the ESP site by scaling the hard rock UHRS (shown in SER Figure 2.5.2-5) by the final AFs (shown in SER Figure 2.5.2-6). The applicant defined each of the AFs at a total of 300 frequencies, but only defined the hard rock UHRS at 7 structural frequencies. For this reason, the applicant interpolated the hard rock UHRS at values between the 7 structural frequencies using the high- and low-frequency spectral shapes for hard rock from NUREG/CR-6728. The applicant's choice of the high- or low-frequency spectral shape for the interpolation depended on the envelope motion. The applicant defined the envelope motion as the envelope of the high- and low-frequency mean output response spectra (defined at the top of the soil column). The applicant noted that at frequencies above 8 Hz, this is always the HF motion and at frequencies below 2 Hz, this is always the LF motion. The applicant further noted that at frequencies between 2 and 8 Hz, the envelope motion depended on the frequency.

Next, the applicant multiplied the hard rock UHRS (now defined at 300 structural frequencies) by either the high- or low-frequency final amplification factors (shown in SER Figure 2.5.2-6). The applicant multiplied the hard rock UHRS by the low-frequency mean amplification factor if it used low-frequency spectral shape to interpolate the hazard rock UHRS at that structural frequency. If the applicant used the high-frequency spectral shape to interpolate the hard rock UHRS at that frequency, then it multiplied the hard rock UHRS by the high-frequency mean AF. The applicant stated that at some intermediate frequencies between 2 and 8 Hz, the high- and low- frequency AFs are weighted in order to achieve a smooth transition between HF and LF spectra.

The applicant repeated the above process for the 10^{-5} hazard level to determine the final 10^{-5} soil UHRS. SER Figure 2.5.2-7 provides the final soil UHRS for the 10^{-4} and 10^{-5} hazard levels.

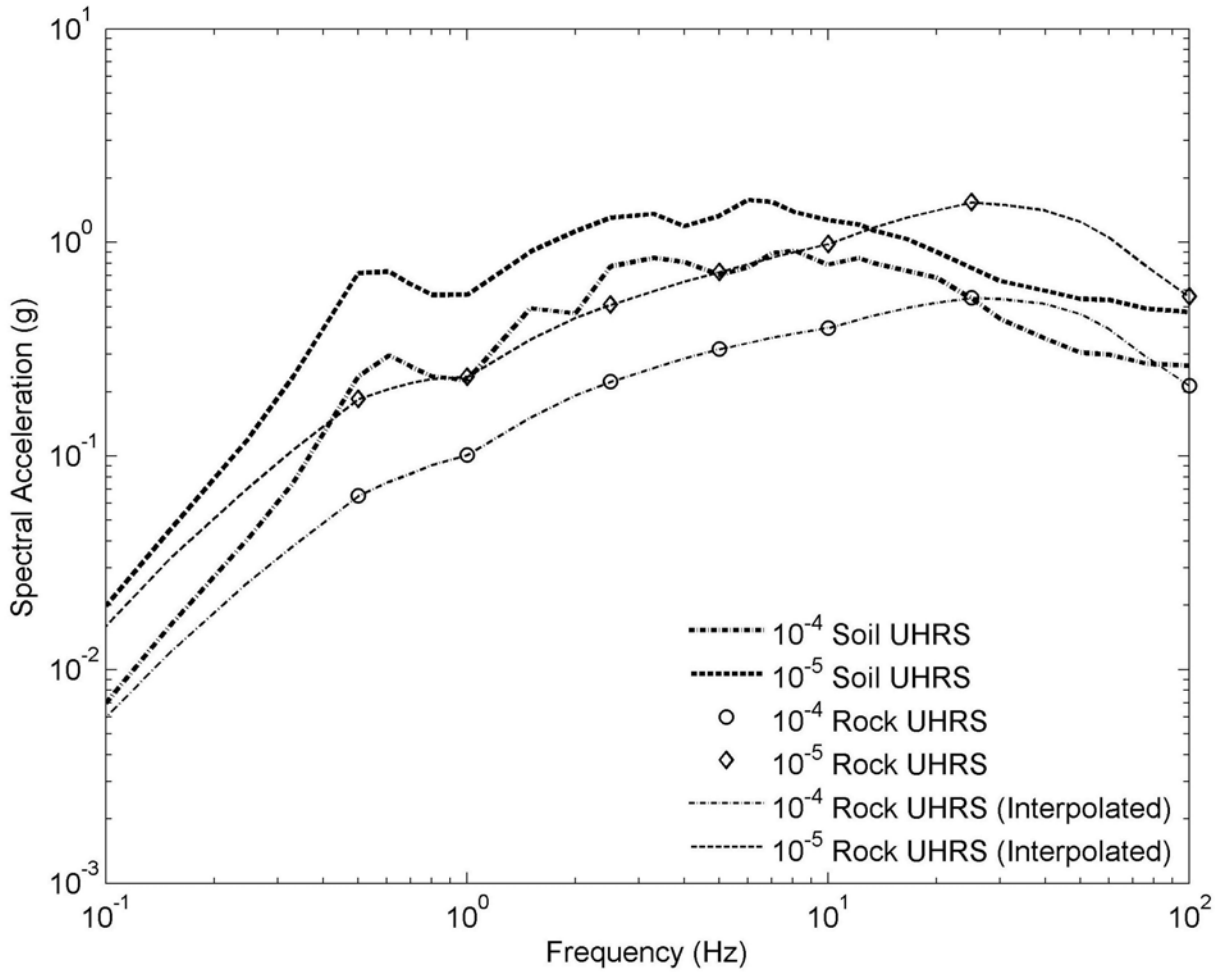


Figure 2.5.2-7 - Horizontal soil-based UHRS for the 10^{-4} and 10^{-5} hazard levels (based on the information provided in SSAR Tables 2.5.2-16 and 2.5.2-21b)

2.5.2.2.6 Ground Motion Response Spectra

SSAR Section 2.5.2.6 describes the method used by the applicant to develop the horizontal and vertical site-specific ground motion response spectra (GMRS). To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and in ASCE/SEI Standard 43-05, "Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities and Commentary." The applicant developed the vertical GMRS by applying vertical-to-horizontal response spectral (V/H) ratios, based on NUREG/CR-6728 and Lee (2001), to the horizontal GMRS.

Horizontal Ground Motion Response Spectrum

The applicant developed a horizontal, site-specific, performance-based GMRS using the method described in ASCE/SEI Standard 43-05 and RG 1.208. The performance-based method achieves the annual target performance goal (PF) of 10^{-5} per year for frequency of onset of significant inelastic deformation. This damage state represents a minimum structural damage state, or essentially elastic behavior, and falls well short of the damage state that would interfere with functionality. The horizontal GMRS, which meets the PF, is obtained by scaling the site-specific mean 10^{-4} UHRS by a design factor (DF):

$$DF = \max\left\{1.0, 0.6\left(A_R\right)^{0.8}\right\} \qquad \text{Equation (1)}$$

where the amplitude ratio, AR, is given by the ratio of the 10^{-5} UHRS and the 10^{-4} UHRS spectral accelerations for each spectral frequency.

The applicant determined the horizontal performance-based GMRS by scaling the 10^{-4} soil UHRS, shown in SER Figure 2.5.2-7, by the DF defined by Equation (1). The applicant's horizontal GMRS is shown in SER Figure 2.5.2-8, which is defined at the top of the structural backfill. The applicant smoothed the GMRS using a running average filter (above 1 Hz) constrained to go through the seven structural frequencies that define the original rock UHRS (SER Figure 2.5.2-5). The applicant made an exception for the 5-Hz structural frequency because of the trough observed in the 10^{-4} soil UHRS (refer to SER Figure 2.5.2-8) at this frequency. The smoothed 5-Hz GMRS value is based on amplitudes at adjacent frequencies. SER Figure 2.5.2-8 also shows the soil UHRS for both the 10^{-4} and 10^{-5} mean hazard levels for comparison.

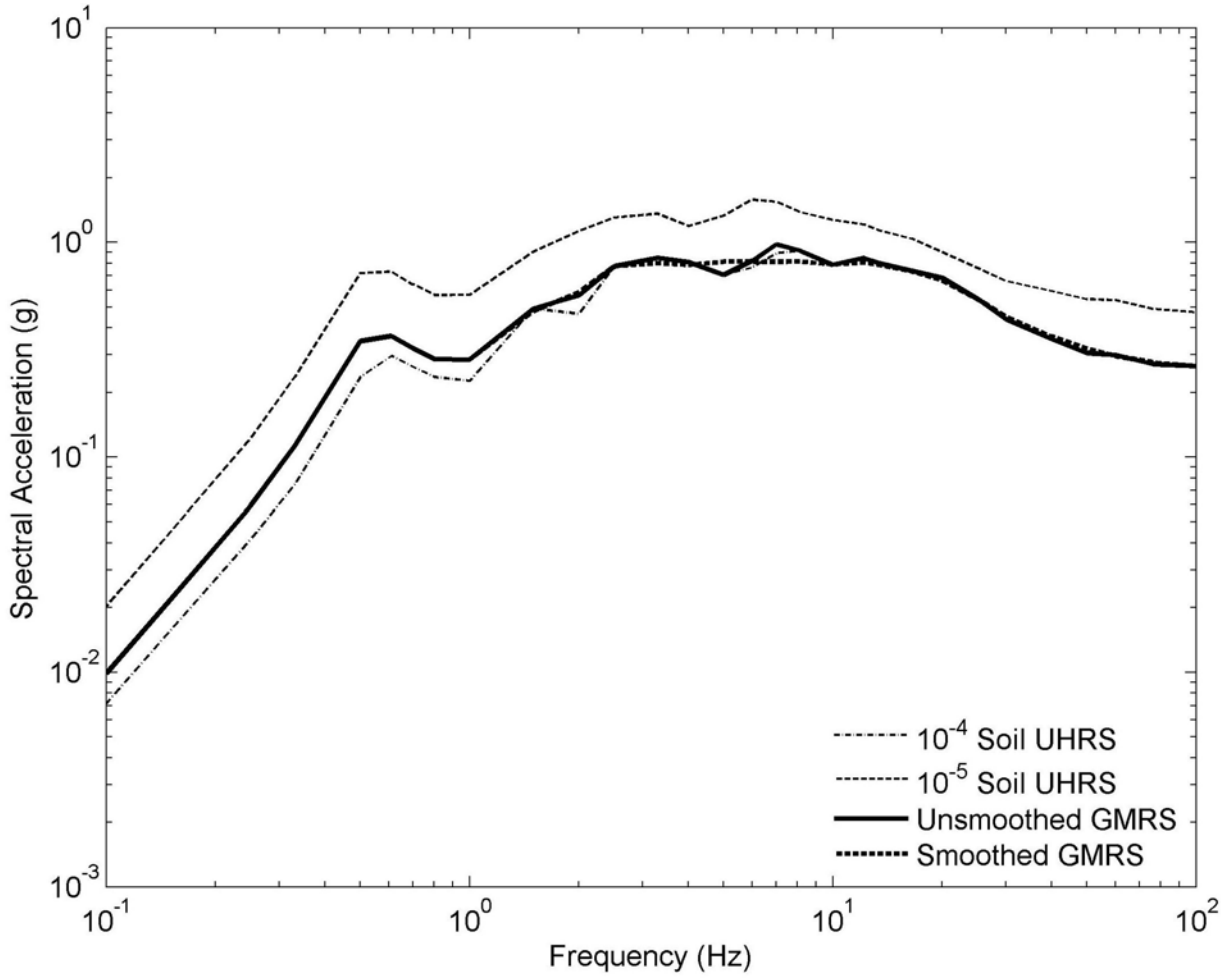


Figure 2.5.2-8 - Horizontal raw and smoothed GMRS (based on the information provided in SSAR Table 2.5.2-22b)

Vertical GMRS

To determine the vertical GMRS, the applicant applied V/H ratios, based on NUREG/CR-6728 and Lee (2001), to the horizontal smoothed GMRS shown in SER Figure 2.5.2-8. Since the V/H ratios presented in NUREG/CR-6728 and Lee (2001) are functions of magnitude, source distance, and local site conditions, the applicant developed V/H ratios corresponding to the final low- and high-frequency controlling earthquakes shown in SER Table 2.5.2-3. The low-frequency controlling earthquake corresponds to an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles), while the high-frequency controlling earthquake is represented by an **M** 5.6 event occurring at a distance of 12 kilometers (7.5 miles).

NUREG/CR-6728 presents V/H ratios for soft rock WUS sites and hard rock CEUS sites. The WUS rock V/H ratios provided in NUREG/CR-6728 are based on an empirical database of WUS strong-motion records. Due to the limited number of available CEUS ground motion recordings, NUREG/CR-6728 uses the WUS ratios and modifies them based on the results of modeling studies to obtain CEUS rock ratios. In addition, Appendix J to NUREG/CR-6728 provides a formula to develop V/H ratios for CEUS soil sites:

$$V/H_{CEUS,Soil} = V/H_{WUS,Soil,Empirical} * \left[V/H_{CEUS,Soil,Model} / V/H_{WUS,Soil,Model} \right] \qquad \text{Equation 2}$$

Because the ESP site is a soil site, the applicant used Equation (2) to determine V/H ratios. The applicant obtained the first term of Equation (2), V/HWUS,Soil,Empirical, from the ground motion model of Abrahamson and Silva (1997) which provides horizontal and vertical ground motion relationships for deep soil sites. In NUREG/CR-6728, generic soil columns were used to determine V/HWUS,Soil,Model and V/HCEUS,Soil,Model ratios, which provided results for **M** 6.5 and distances of 1, 5, 10, 20, and 40 kilometers. The applicant obtained the second term of Equation (2) using V/HCEUS,Soil,Model and V/HWUS,Soil,Model ratios corresponding to **M** 6.5 and 20 kilometers to represent the high-frequency (**M** 5.6, 12 km) controlling earthquake. In addition, the applicant used the V/HCEUS,Soil,Model and V/HWUS,Soil,Model ratios corresponding to **M** 6.5 and 40 kilometers to represent the low-frequency (**M** 7.2, 130 km) controlling earthquake. The applicant considered these magnitude and distance substitutions to be conservative because V/H ratios are observed to decrease with distance for a given magnitude. The applicant assigned a weight of approximately 1:3 to the results representing the high- and low-frequency controlling earthquakes, respectively.

Lee (2001) used the methodology outlined in NUREG/CR-6728 to develop V/H ratios for the MOX Fuel Fabrication Facility at the SRS. However, Lee (2001) developed V/HCEUS,Soil,Model ratios using a site-specific soil model for the SRS, rather than the generic CEUS profile used in Appendix J to NUREG/CR-6728. To obtain V/H ratios corresponding to the high-frequency controlling earthquake (**M** 5.6, 12 km), the applicant interpolated the results provided in Lee (2001) between **M** 5.5 at 10 kilometers and 20 kilometers and **M** 6.0 at 10 kilometers and 20 kilometers. Similarly, to obtain V/H ratios corresponding to the **M** 7.2, 130-km earthquake, the applicant interpolated the results provided in Lee (2001) between **M** 7.0 at 100 kilometers and **M** 7.2 at 100 kilometers. The distance of 100 kilometers was the largest distance considered in Lee (2001). However, the applicant considered the distance substitution of 100 kilometers for 130 kilometers to be conservative because V/H ratios are observed to

decrease with distance for a given magnitude. The applicant assigned a weight of approximately 1:3 to the results representing the high- and low-frequency controlling earthquakes, respectively.

SER Figure 2.5.2-9 plots the resulting V/H ratios obtained from NUREG/CR-6728 and Lee (2001), as well as the final V/H ratios. The V/H ratios from Lee (2001) are higher than those derived from the NUREG/CR-6728 results for frequencies greater than about 0.7 Hz. To develop the final V/H ratios, the applicant used an approximate envelope of the two results. The applicant assigned a greater weight to the V/H ratios from Lee (2001) because this study used a site-specific soil model for the nearby SRS. SER Figure 2.5.2-7 also plots V/H ratios from RG 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Revision 1, issued December 1973. The final V/H ratios are slightly less than those provided in RG 1.60 at all frequencies.

To obtain the vertical GMRS, the applicant scaled the horizontal smoothed GMRS, shown in SER Figure 2.5.2-8, by the final V/H ratio (shown in SER Figure 2.5.2-9).

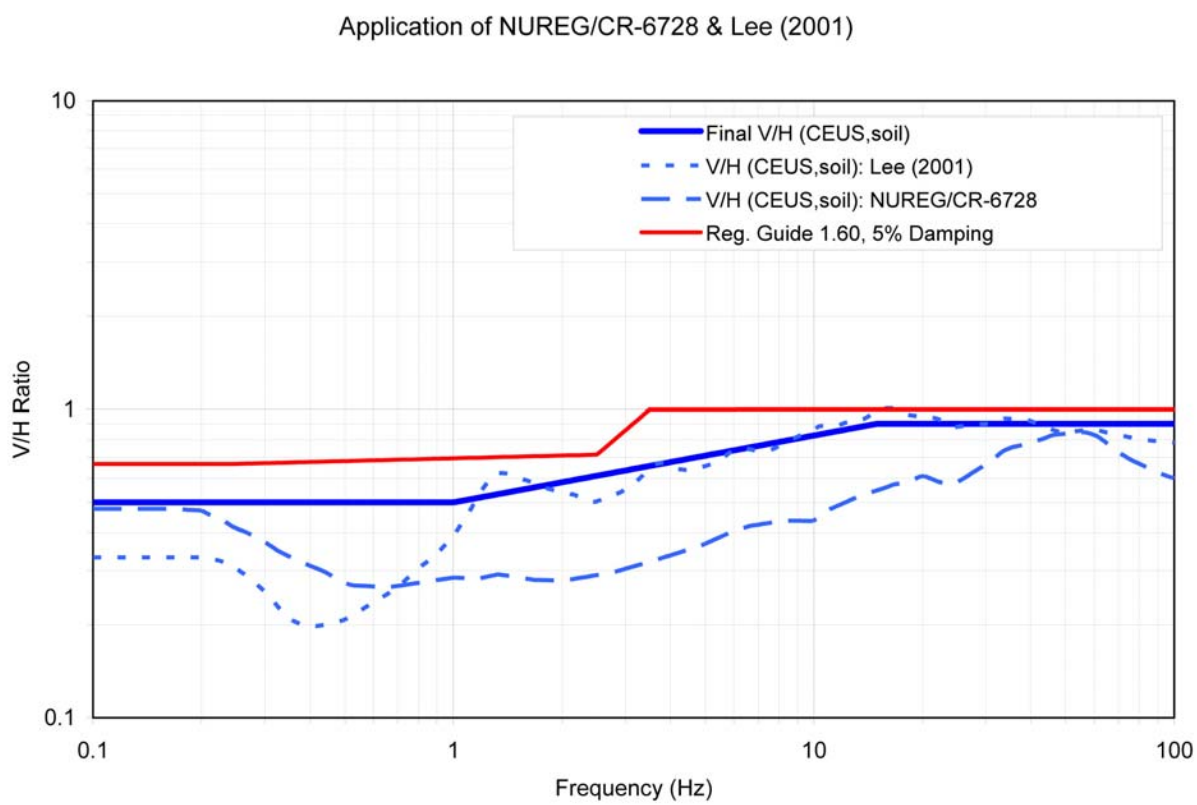


Figure 2.5.2-9 - Final V/HCEUS,Soil ratios (reproduced from SSAR Figure 2.5.2-43)

2.5.2.2.8 Operating Basis Ground Motion

The applicant did not determine the Operating Basis Earthquake (OBE) as part of the Vogtle ESP and stated that the OBE will be determined during the COL stage.

2.5.2.2.9 Sensitivity Studies

As part of its COL site investigation, the applicant collected additional S-wave velocity data and developed site-specific strain-dependent shear modulus and damping relationships based on RCTS test results. However, the applicant did not use any of this additional COL data as input to its site response calculations. Instead, the applicant relied on the SRS and generic EPRI strain-dependent shear modulus and damping curves and S-wave velocity profiles developed as part of the ESP. Rather than recalculating site amplification factors that also account for additional COL data, the applicant performed site response sensitivity calculations for a select number of cases in order to demonstrate that use of the ESP S-wave velocity data and SRS and generic EPRI strain-dependent shear modulus and damping curves is appropriate.

The applicant conducted three sets of sensitivity calculations in order to evaluate: (1) the sensitivity of the AP1000 nuclear island responses to changes in the backfill S-wave velocity; (2) the effects of the backfill geometry on the site response and on the SSI response of the nuclear island; and (3) the effects of additional COL data on site response.

In the first set of calculations, the applicant evaluated the effects of changes in the backfill S-wave velocity. A comparison of the ESP S-wave velocity profile (used for the GMRS and FIRS [foundation input response spectra] computation in SSAR Section 2.5.2.5.1.5) with the S-wave velocity profile used in the sensitivity study is provided in SSAR Figure 2.5.2-51. The staff notes that the S-wave velocity profile used in the sensitivity study did not correspond to the COL backfill data because the applicant performed the sensitivity study before conducting the Phase I test pad program. The S-wave velocity values of the sensitivity study median S-wave velocity profile are larger than both the ESP and COL profiles, which are provided in SSAR Tables 2.5.4-10 and 2.5.4-10a, respectively. The applicant's analysis involved the randomization of the entire soil column with new backfill properties and development of the new outcrop motion at the foundation level of the AP1000 nuclear island. The applicant then used the new time-history and associated strain-compatible soil properties in the SSI analysis of the AP1000. The results of this sensitivity study are provided in Appendix 2.5E (Vogtle Site Specific Seismic Evaluation Report) to the SSAR. The applicant concluded that, even with significant variation of the backfill S-wave velocity, the AP1000 design is applicable to the Vogtle site with a large margin.

In the second sensitivity study, the applicant evaluated the effects of the backfill geometry. Due to the large volume of excavation and the large lateral extent of the backfill at the Vogtle site, the applicant modeled the backfill layers as free-field soil layers for both the soil amplification for development of the ground motion (GMRS and FIRS) and the site-specific seismic SSI analysis of the AP1000. To confirm this assumption, the applicant performed a two-dimensional site response analysis (Part I) followed by a two-dimensional SSI analysis (Part II) of the AP1000 model in order to evaluate the extent of backfill on the site response and on the SSI response of

the Nuclear Island. For the 2D analysis, the applicant used the cross section shown in the East-West direction provided in SSAR Figure 2.5.2-53. In Part I of the analysis, the applicant performed a 2D site response analysis. The applicant's 2D model for the site response analysis is provided in SSAR Figure 2.5.2-54, which is based on the cross section shown in SSAR Figure 2.5.2-53. The applicant used the same properties for backfill, Blue Bluff Marl, the lower sand layers and layers extending to the rock at the base as those that it used to develop the GMRS and FIRS. The computation of the GMRS and FIRS (described in SSAR Section 2.5.2.5), however, involved 60 randomized soil profiles, 30 high-frequency and 30 low-frequency input time histories). Thus, for its 2D analysis, the applicant only considered a subset of the soil profiles (i.e. the upper, mean, and lower bound soil profiles) and input time histories (i.e. three high-frequency and three low-frequency input time histories). The applicant compared the resulting site amplification factors with those calculated from the 1D SHAKE results for the same set of input motions and soil properties, which are shown in SSAR Figures 2.5.2-55, 2.5.2-56, and 2.5.2-57 for locations (presented as "in-column" motions) at depths of 0 ft (GMRS), 40 ft (FIRS horizon), and at 86 ft depth (Top of Blue Bluff Marl), respectively, at the centerline of the backfill (shown in SSAR Figure 2.5.2-54). The applicant concluded that the differences are very small. The applicant further concluded that the geometry of the backfill has an insignificant effect on GMRS and FIRS.

In Part II, the applicant developed a Vogtle 2D SASSI model of the nuclear island (NI) to include the backfill as part of the structural model shown in Figure 2.5.2-58. This model is similar to the model in Part I except that the applicant included the AP1000 NI model using only the mean soil profile and a single time history from the analysis performed in Part I (i.e. the input motions for the two SSI analyses are obtained from the respective 1D SHAKE analysis in Part I). The applicant compared the SSI responses for the 2D SASSI NI model (referred to as Bathtub Model-d5) at key locations in the NI are compared with the SSI results of the 2D SASSI (referred to as 2D-AP-d5) that assumes backfill extends to infinity in lateral directions. These comparisons are shown in SSAR Figure 2.5.2-59 through 2.5.2-64. The applicant concluded that the response spectra are similar and it considered the differences to be negligible. The applicant also plotted the generic AP1000 standard design response spectra for comparison for the purpose of demonstrating that a significant margin exists between the AP1000 generic response and the Vogtle 2D results.

Finally, the applicant performed sensitivity studies to evaluate the effects of the additional COL S-wave velocity and the strain dependent shear modulus and damping relationships based on RCTS test results. As input, the applicant selected three high-frequency and three low-frequency rock time histories representing the 10^{-4} annual exceedance frequency level from the suite of motions used for the GMRS computation in SSAR Section 2.5.2.5. The applicant also used three soil profiles representing the best estimate COL velocity profile (shown in SSAR Figure 2.5.4-7a) as well as the upper and lower bounds. In addition, the applicant used the associated COL strain-dependent soil properties presented in SSAR Figures 2.5.4-9a and 2.5.4-11a and in SSAR Table 2.5.4-12a. The applicant performed two sets of analyses in order to consider the high and low PI (Plasticity Index) cases of the Blue Bluff Marl as illustrated in SSAR Figures 2.5.4-9a and 2.5.4-11a. The applicant then averaged the results using the three high-frequency input time histories, three soil profiles, and the high and low PI cases of the Blue Bluff Marl, then divided this average response spectrum (corresponding to a depth of 40 ft) by the 10^{-4} high-frequency input response spectrum to obtain site amplification factors. The

applicant repeated this process for the low-frequency input time histories. The applicant then enveloped the resulting high-frequency and low-frequency amplification factors, which is represented by the green dashed curve in SSAR Figure 2.5.2-65c. The blue solid curve in SSAR Figure 2.5.2-65c corresponds to the amplification factors based on a limited number of ESP soil profiles. From the ESP set of runs described in SSAR Section 2.5.2.5.1, the applicant used the strain compatible velocity and damping profiles to obtain the median and upper bound profiles (using one standard deviation as the variation) to use as input to the analysis. The applicant used the same three high-frequency and three low-frequency time histories used for the analysis of the COL data above. In SSAR Figure 2.5.2-65c, the applicant also plotted (depicted by the red dashed curve) the amplification factors resulting from the fully randomized ESP soil profiles and entire group of input time histories (described in SSAR Section 2.5.2.5). The applicant concluded that the comparison of the two sets of results based on the ESP data shows good agreement and thus that the limited number of profiles and time histories are adequate for the purpose of the evaluation of the inpact of the COL data. Furthermore, the applicant concluded that the difference in amplification between the ESP and COL data is small.

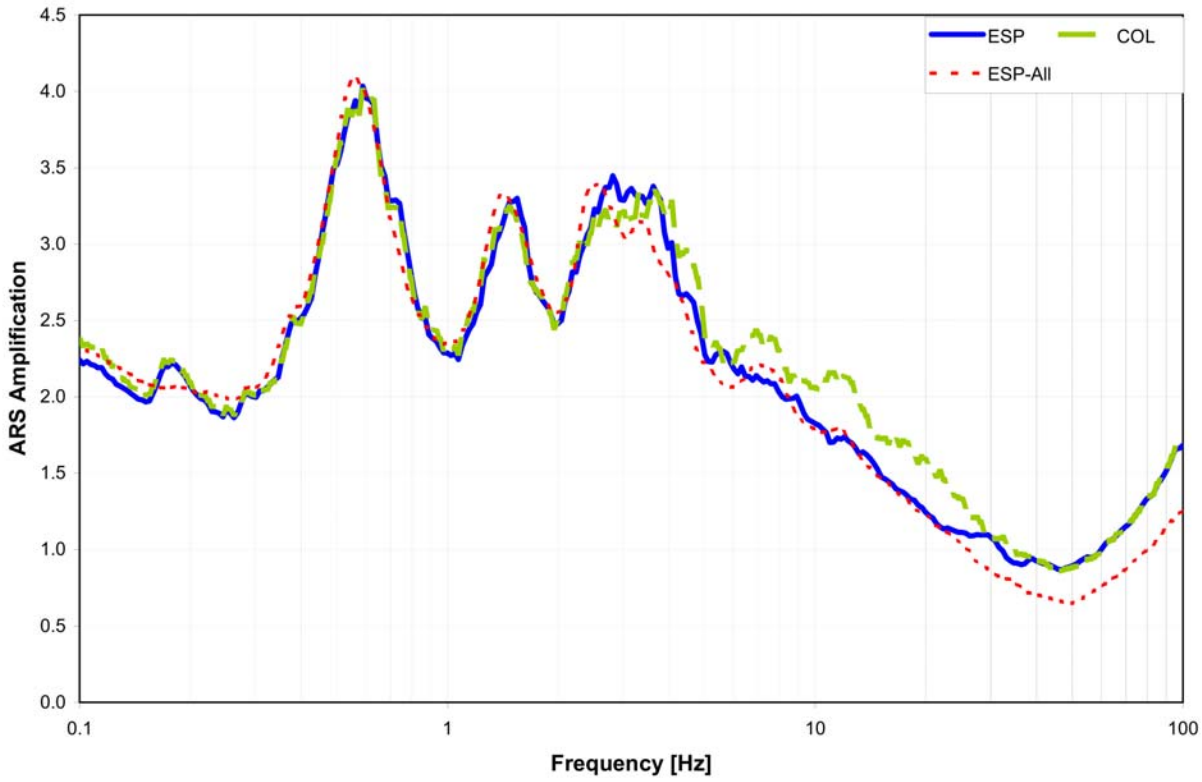


Figure 2.5.2-10 – Comparison of Amplification Factors from Sensitivity Analyses (reproduced from SSAR Figure 2.5.2-65c)

2.5.2.3 Regulatory Basis

SSAR Section 2.5.2 presents the applicant's determination of ground motion at the ESP site from possible earthquakes that might occur in the site region and beyond. In SSAR Section 1.8, the applicant stated that it had developed the geological and seismological information used to determine the seismic hazard in accordance with regulations listed in SSAR Table 1-2, which includes 10 CFR 50.34; Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," to 10 CFR Part 50; and 10 CFR 100.23. The applicant further stated in SSAR Table 1-2 that it developed this information in accordance with the guidance presented in Section 2.5.2 of Revision 3 of NUREG-0800 and RG 1.165. The staff reviewed this portion of the application for conformance with the regulatory requirements and guidance applicable to the determination of the SSE ground motion for the ESP site, as identified below. The staff notes that the application of Appendix S to 10 CFR Part 50 in an ESP review, as referenced in 10 CFR 100.23(d)(1), is limited to defining the minimum SSE for design.

In its application review, the staff considered the regulatory requirements of 10 CFR 52.17(a)(1)(vi) and 10 CFR 100.23(c) and (d), which require that the applicant for an ESP describe the seismic and geologic characteristics of the proposed site. In particular, 10 CFR 100.23(c) requires that an ESP applicant investigate the geological, seismological, and engineering characteristics of the proposed site and its environs with sufficient scope and detail to support estimates of the SSE ground motion and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site. In addition, 10 CFR 100.23(d) states that the SSE ground motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface. Section 2.5.2 of Revision 3 of NUREG-0800 and RG 1.208 provide guidance concerning the evaluation of the proposed SSE ground motion, and RGs 1.165 and 1.208 provide guidance regarding the use of PSHA to address the uncertainties inherent in the estimation of ground motion at the ESP site.

2.5.2.4 Technical Evaluation

This section of the SER provides the staff's evaluation of the seismological, geological, and geotechnical investigations that the applicant conducted to determine the GMRS for the ESP site. The technical information presented in SSAR Section 2.5.2 resulted from the applicant's surface and subsurface geological, seismological, and geotechnical investigations performed in progressively greater detail as distance to the ESP site decreases. The GMRS is based upon a detailed evaluation of earthquake potential, taking into account regional and local geology, Quaternary (1.8 mya–present) tectonics, seismicity, and specific geotechnical characteristics of the site's subsurface materials.

SSAR Section 2.5.2 characterizes the ground motions at the ESP site from possible earthquakes that might occur in the site region and beyond to determine the site GMRS. According to RG 1.208, applicants may develop the GMRS for a new nuclear power plant using either the EPRI or LLNL PSHAs for the CEUS. However, RG 1.208 recommends that applicants perform geological, seismological, and geophysical investigations and evaluate any relevant research to determine whether revisions to the EPRI or LLNL PSHA databases are

necessary. As a result, the staff focused its review on geologic and seismic data published since the late 1980s that could indicate a need for changes to the EPRI or LLNL PSHAs.

2.5.2.4.1 Seismicity

SSAR Section 2.5.2.1 describes the development of a current earthquake catalog for the ESP site. The applicant started with the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is complete though 1984. To update the earthquake catalog, the applicant used information from the ANSS and SEUSS.

The staff focused its review of SSAR Section 2.5.2.1 on the adequacy of the applicant's description of the historical record of earthquakes in the site region. In Request for Additional Information (RAI) 2.5.2-1, the staff asked the applicant to provide electronic versions of the EPRI seismicity catalog (EPRI NP-4726-A 1988) for the region of interest (30degrees to 37degrees N, 78 degrees to 86 degrees W), as well as its updated EPRI seismicity catalog. The staff used the catalog data that the applicant provided in response to RAI 2.5.2-1 to compare with its own compilation of recent earthquakes for the site region. The applicant's updated catalog consisted of a total of 61 events. Of these 61 events, there were 56 mb 3 events and 5 mb 4 events. In comparison, the staff's list of earthquakes, based entirely on the ANSS earthquake catalog, consisted of 50 mb 3 events and 3 mb 4 events.

Because the applicant used the EPRI historical earthquake catalog (EPRI NP-4726-A 1988), which is part of the 1989 EPRI seismic hazard study that the NRC endorsed in RG 1.165, the staff concludes that the seismicity catalog used by the applicant is complete and accurate for the time period 1777–1985. The staff compared the applicant's update of the regional seismicity catalog with its own listing of recent earthquakes and, as a result, concludes that the earthquake catalog used by the applicant is complete and provides a conservative estimate of earthquake magnitudes and locations for the ESP site region.

To determine whether the seismicity rates used in the EPRI study (EPRI NP-6395-D 1989) are appropriate for the assessment of the seismic hazard at the ESP site, the applicant used two areas in the site region: (1) a small rectangular area around the Charleston seismicity; and (2) a triangular-shaped area that envelops the seismicity in South Carolina and a strip of Georgia. The applicant concluded that, for the rectangular Charleston source, the updated catalog indicates that the seismicity rates are the same. For the triangular South Carolina source, the updated catalog indicated that seismicity rates decreased when the seismicity from 1985 to April 2005 was added. In RAI 2.5.2-18, the staff asked the applicant to provide a justification for the selection of the geometries used to represent the Charleston source and the South Carolina source. In response to RAI 2.5.2-18, the applicant assessed the seismicity in two additional areas within the site region. The applicant concluded that any region in South Carolina that would affect the seismic hazard at the ESP site would have estimated activity rates that stay constant or decrease, if the new regional earthquake catalog were added to the analysis.

Based on the applicant's evaluation of multiple areas and its determination that seismicity rates in the region have not increased since 1985 for any of these selected areas, the staff concludes that the applicant's use of the EPRI seismicity rates is appropriate and that these rates are appropriate for the assessment of the seismic hazard at the ESP site.

2.5.2.4.2 Geologic and Tectonic Characteristics of the Site and Region

SSAR Section 2.5.2.2 describes the seismic sources and seismicity parameters used by the applicant to calculate the seismic ground motion hazard for the ESP site. Specifically, the applicant described the seismic source interpretations from the 1986 EPRI Project (EPRI NP-4726), relevant post-EPRI seismic source characterization studies, and its updated EPRI seismic source zone for the Charleston area. The staff focused its review of SSAR Section 2.5.2.2 on the applicant's update of the Charleston seismic source zone. The staff also reviewed the applicant's basis for not updating the other EPRI source zones that contribute to the seismic hazard at the ESP site.

Summary of EPRI Seismic Sources

Section 2.5.2.2.1 summarizes the seismic sources and seismicity parameters used in the 1986 EPRI Project and subsequently implemented in the 1989 PSHA (EPRI NP-D 1989). The 1989 EPRI PSHA study expressed Mmax values in terms of mb. The applicant noted that most modern seismic hazard analyses describe Mmax in terms of **M** and used the arithmetic average of the conversion relations presented in Atkinson and Boore (1995), Frankel et al. (1996), and EPRI TR-102293 (1993) to convert from mb to **M**. In RAI 2.5.2-5, the staff asked the applicant to provide its converted **M** values. In response to RAI 2.5.2-5, the applicant provided a table that listed a range of mb values and the corresponding converted **M** values.

To confirm the applicant's magnitude conversions, the staff compared the applicant's converted **M** values with the **M** values it obtained using the conversion relations of Frankel et al. (1996) and Johnston (1994), which were provided in Chapman and Talwani (2002). The staff found that the conversion provided in Chapman and Talwani (2002) yields slightly larger **M** values in the mb 4.0 to 7.5 range. However, based on the uncertainties associated with magnitude conversions and the applicant's use of the average of three different conversion relations to account for this uncertainty, the staff concludes that the applicant's converted **M** values are adequate.

SSAR Sections 2.5.2.2.1.1 through 2.5.2.2.1.6 provide a summary of the primary seismic sources developed in the 1980s by each of the six EPRI ESTs. Each EST described its set of seismic source zones for the CEUS in terms of source geometry, probability of activity, recurrence, and Mmax. Each EPRI EST identified one or more seismic source zones that include the ESP site. Although some of the ERPI ESTs assigned Mmax values as high as **M** 7.5 for the source zones that make up the Atlantic coastal region, the Mmax values for the seismic source zones that include the site have a weighted mean of about **M** 6.0. In RAI 2.5.2-6, the staff asked the applicant to explain whether it considered more recent studies on large worldwide earthquakes by Johnston (1994) and Kanter (1994) as possible updates of the earlier EPRI seismic source models.

In response to RAI 2.5.2-6, the applicant stated that the final versions of the Johnston (1994) and Kanter (1994) assessments (included in Volume 1 of the Johnston et al. 1994 study) do not constitute new information that would require an update of the Mmax values used for the EPRI seismic source models. In its response, the applicant stated that the initial results of the

Johnston et al. (1994) study were available to the EPRI ESTs, and that the final results of the Johnston et al. (1994) study generally support the initial findings of the study.

The staff reviewed the applicant's response to RAI 2.5.2-6 and concluded that, although many of the EPRI ESTs assigned Mmax values that reflect the studies of Johnston and Kanter, the applicant did not provide an adequate justification to support the low weights for some of the larger Mmax values. In particular, the Dames and Moore EST gave fairly low weights to some of its seismic source zones. For example, the two Mmax values assigned by the Dames and Moore EST for the "Southern Appalachian Mobile Belt" are mb 5.6 with a weight of 0.8 and 7.2 with a weight of 0.2. These two Mmax values and weights are similar to those for the other ESTs for the Atlantic coastal margin; however, the Dames and Moore EST also assigned a probability of activity of only 0.26 for this source. Similarly, for its "Southern Cratonic Margin," the Dames and Moore EST assigned a probability of activity of only 0.12. The combined effect of these low probabilities of activity and low weights for the larger magnitudes results in a lower hazard for the ESP site. This result is shown in SER Figures 2.5.2-17 and 2.5.2-18, which are plots of the 1- and 10-Hz PSHA hazard curves for each of the EPRI ESTs. As shown in these two figures, the Dames and Moore seismic hazard curves are substantially lower than those for the other ESTs.

In response to RAI 2.5.2-6, the applicant also stated that the Vogtle ESP site is located within Kanter's (1994) Piedmont domain 223 in nonextended crust and, as a result, large magnitude earthquakes are not expected in this domain. The staff, however, believes that the ESP site is located within the Mesozoic passive margin. Specifically, the site is on the hanging wall of the southeast-dipping Pen Branch fault (SSAR Figures 2.5.1-2, 2.5.1-29, and 2.5.1-34), which is the main border fault of the Dunbarton Triassic basin (SSAR Figures 2.5.1-2 and 2.5.1-10). In turn, the Dunbarton Triassic basin is a subbasin within the much larger South Georgia basin complex (SSAR Figures 2.5.1-2 and 2.5.1-7). Therefore, the site is in Kanter's Eastern Seaboard domain 218. The rocks beneath the site are Triassic strata of domain 218's rift basins (SSAR Figures 2.5.1-34 and 2.5.1-38). Beneath the Triassic rocks is the Piedmont domain, but the Piedmont rocks have been cut by the Mesozoic extensional faults that bound the rifts. The distinction between the Eastern Seaboard and Piedmont domains depends on the presence or absence of Mesozoic extensional faults, rather than the age of the rocks cut by those faults. Accordingly, the staff believes that the site is subject to the higher Mmax of the Eastern Seaboard domain of Kanter (1994). The site is in one of the regions that Johnston et al. (1994) found to have hosted all earthquakes of **M** 7.0 and larger in the world's stable continental regions (SCRs).

SER Figure 2.5.2-11 shows a histogram of magnitudes of the 30 earthquakes that had **M** 6.5 and larger in the world's extended margin, which is based on the compilation of the largest earthquakes in the world's SCRs by Johnston et al. (1994). The histogram has a large peak at **M** 6.6 and 6.7. The earthquakes making up the peak come from various SCRs, continents, and plate tectonic settings, indicating that values of 6.6 and 6.7 occur widely in diverse geologic and tectonic settings. This implies that Mmax is unlikely to be less than these values anywhere in the extended margin of North America. As such, the low weights and low probability of activities assigned by the Dames and Moore EST to larger Mmax values do not reflect worldwide earthquake activity in extended margins.

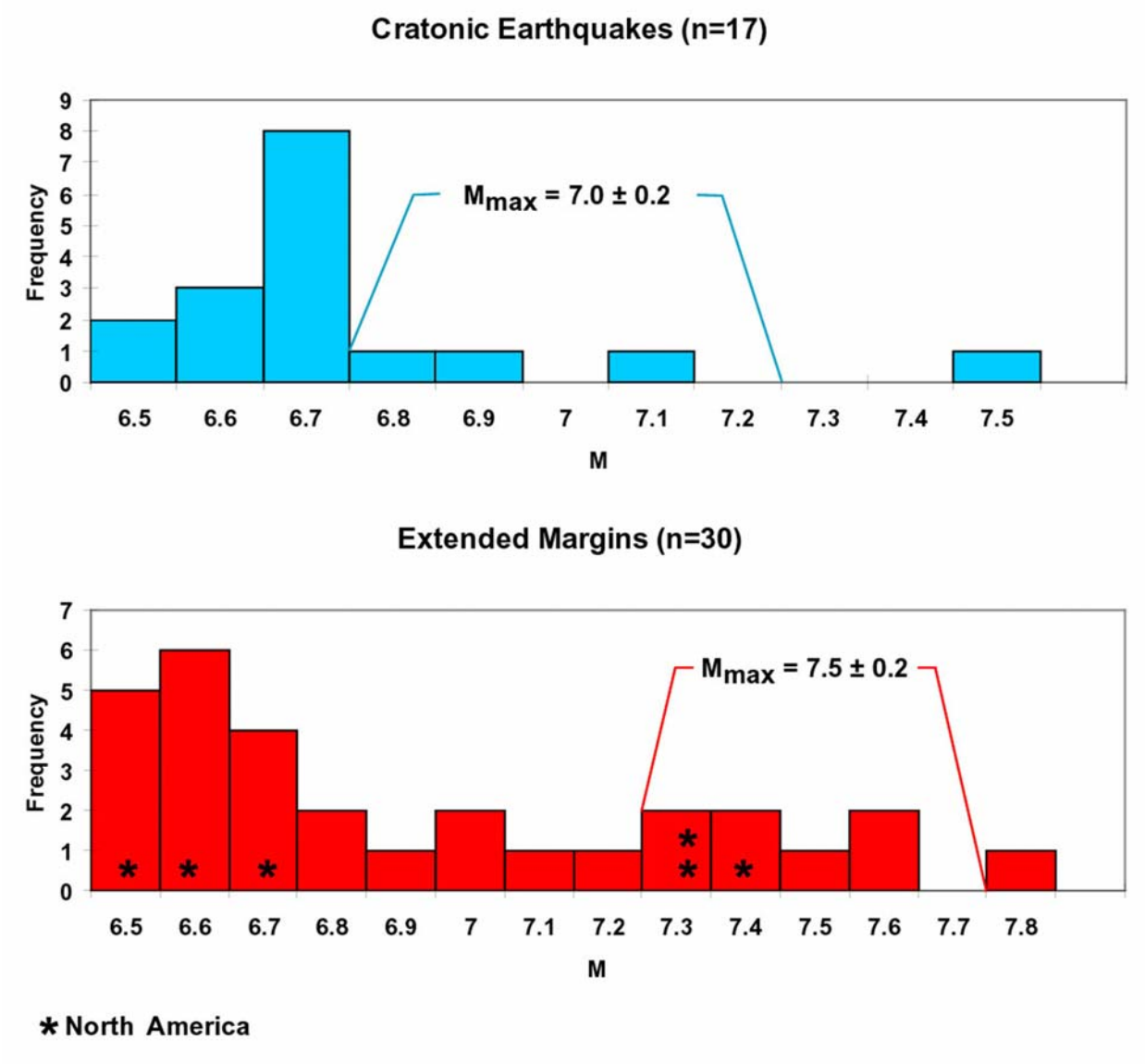


Figure 2.5.2-11 - Histogram showing magnitudes of the 30 earthquakes that had M 6.5 and larger in the world's extended margins (Source: USGS)

In summary, the staff concluded that the applicant did not provide an adequate justification to support the low weights for the larger Mmax values for the EPRI source zones that include the site. In particular, the staff was concerned that the low weights and low probability of activities assigned by the Dames and Moore EST to some of its seismic source zones result in hazard curves for the ESP site that may not adequately characterize the regional seismic hazard. In addition, the staff concluded that the site is located within the Mesozoic passive margin, rather than the Piedmont unextended province as stated in the applicant's response. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-1.

As noted above, Open Item 2.5-1 related to the staff's concern that the low weights and low probability of activities assigned by the Dames and Moore EST to some of its seismic source zones resulted in hazard curves for the ESP site that may not adequately characterize regional seismic hazard. In response to Open Item 2.5-1, the applicant stated the following:

> As pointed out in the DSER, the Dames & Moore team assigned low probabilities of activity (PA) to some of its sources, such as source zones 41 and 53. Zone 53 (Southern Appalachian Mobile Belt) is a default zone for several Triassic rift basin sources, represents a host zone for the Vogtle site, and has a PA = 0.26. The lack of a background zone beneath the region covered by source 53 results in a source-less area when 53 is "turned off." While the implementation of this aspect of the Dames & Moore source model has been the subject of debate, this is not an "error" or misinterpretation in their model. Statements in both the Dames & Moore EPRI report (1986) as well as recent discussions with James McWhorter, an original member of the Dames & Moore EST, indicate that Dames & Moore intended to represent the earthquake process in this fashion.

The applicant provided the following discussion from page 5-3 of the Dames and Moore report (1986, Volume 6), which indicates that Dames and Moore believes earthquake occurrence can be explained by tectonic reasons and that they do not use background zones as in other traditional seismic hazard assessments:

> "In our model, uniform seismicity is a consequence of a reasonable tectonic explanation for earthquake occurrence in the zone. To avoid muddling the tectonic aspect, our team does not use backgrounds. There is either a tectonic reason for a block of the earth's crust to be seismically active or there is not. So what we formerly called a "global background" no longer exists; the sources replacing it have a PA reflecting our confidence in a tectonic reason for earthquake activity there."

The applicant stated that although the Dames and Moore seismic source zone implementation is different from the other ESTs, it still represents the range of expert opinion in the EPRI SSHAC Level 4 study. The applicant further stated that "from a process standpoint, it is not the responsibility of the applicant to defend the original rationale or implementation of the EPRI study, which has been approved by the NRC in Regulatory Guide 1.165 and forms the basis for evaluating sites across the CEUS. The individual teams were given latitude as to how to model seismic hazard in order to capture the full range of opinion for the poorly understood earthquake process in the CEUS. Without new data to invalidate the model, an individual team or model should not be reinterpreted or disregarded simply because their resultant hazard is less than the other EST source models."

In addition, the applicant subsequently provided supplemental information regarding Open Item 2.5-1 in a letter dated December 11, 2007. This letter addressed additional concerns that the staff had about the Dames and Moore model regarding a quotation in the 1992 DOE Standard "Guidelines for Use of Probabilistic Seismic Hazard Curves at Department of Energy Sites for Department of Energy Facilities" (DOE-STD-1024-92). The purpose of the DOE Standard was to provide guidance in the use of the seismic hazard curves developed by the

LLNL and the EPRI. The Standard based its recommendations on the evaluations of the LLNL and EPRI seismic hazard method performed by LLNL, Jack Benjamin and Associates, and Risk Engineering Inc. The following quotation is from one of the issues identified by Risk Engineering, Inc.:

> "Risk Engineering, Inc. has also found that the EPRI team of Dames and Moore does not fully account for historic seismicity near the Savannah River Site (SRS). One reason for this is the fact that the SRS host source zone was given a low probability of activity. Risk Engineering, Inc. recommended that the Dames and Moore seismic source input not be used to calculate the seismic hazard at SRS."

The applicant's December 11, 2007 supplemental information contained a letter enclosure from Dr. Robin K. McGuire of Risk Engineering, Inc., which provided additional background regarding the above quotation. In his letter, Dr. McGuire stated that "the quote from my 1991 report was taken from a study that had the purpose of reconciling hazard curves from the EPRI and LLNL reports. In my role as a seismic-hazard analyst in that project (rather than an expert in seismic source characterization), I achieved the project goal by giving credibility only to those interpretations that were consistent with historical seismicity at all magnitude levels. Interpretations that were high or low relative to historical seismicity were given zero weight. The remaining interpretations gave hazard that was relatively consistent (as one would expect), which achieved the purpose of the study. Thus the down-weighting of the Dames & Moore source model was not made on the basis of its geologic or technical merits."

With respect to the quotation in the DOE report, Dr. McGuire stated the following:

> "Examining historical earthquakes from the EPRI catalog in Dames & Moore source 53, one event occurred in 1966 with mb=4.7, and all other historical earthquakes had mb≤4.3. A search of the PDE and ISC catalogs indicates that the 1966 event was an offshore explosion, and if so the largest historical earthquake in source 53 was mb~4.3. In any case the quotation in the 1st paragraph is accurate relative to historical earthquakes with mb≤4.7, but the Dames & Moore interpretation is not inconsistent with the occurrence of earthquakes with mb>5. Stated another way, no earthquakes with mb>5 have occurred historically in the Dames & Moore source 53, and Dames & Moore said there is a 26 percent chance that earthquakes with mb>5 will occur there in the future."

In its supplemental response, the applicant also provided a letter from Dr. Robert Kennedy, which demonstrated that the Dames and Moore model contribution is not significant at the Vogtle ESP site. Dr. Kennedy looked at the 10 Hz total mean hazard curve together with the contributing mean hazard curves from the updated Charleston source and each of the six ESTs source models. He noted that at any spectral acceleration, the total mean annual frequency of exceedance, H, is given by combining the Charleston source mean annual frequency of exceedance with the mean of the 6 ESTs mean annual frequency of exceedance:

$$H = H_C + \left(H_R + H_{WC} + H_{We} + H_L + H_B + H_{DM}\right)/6 \qquad \text{Equation (3)}$$

Where HC is the mean annual frequency of exceedance from the updated Charleston source, and HR, HWC, HWe, HL, HB, HDM, are the mean annual frequencies of exceedance from the Rondout, Woodward-Clyde, Weston, Law, Bechtel, and Dames and Moore teams, respectively. At a spectral acceleration of 0.42 g, Dr. Kennedy found that deleting the Dames and Moore input (HDM) increased the total mean annual frequency of exceedance by only approximately 5 percent. He further concluded that similar results exist at a spectral acceleration corresponding to a mean annual frequency of exceedance of 10^{-5}.

In reviewing the response to Open Item 2.5-1 and supplemental information provided by the applicant, the staff concluded that the applicant did not provide adequate justification for the low probabilities of activity that Dames and Moore team assigned to several of its source zones. The staff is concerned because the Dames and Moore model states that there is only a 26 percent and 12 percent chance that earthquakes larger than mb 5.0 can occur in source zones 53 and 42, respectively. The Dames and Moore team's interpretation differs significantly from the other ESTs interpretations as well as other recent seismic hazard studies including USGS, SCDOT, and TIP studies. The staff, however, agrees with the applicant's determination that the Dames and Moore team does not contribute significantly to the hazard at the Vogtle site. The staff performed a similar comparison to the one performed by Dr. Kennedy, but instead compared percentage changes in spectral acceleration rather than annual exceedance frequency. The results showed that the percentage increase in the 10 Hz total mean hazard spectral acceleration at the 10^{-4} annual exceedance frequency is 2.07 percent if the Dames and Moore team's contribution is removed. At the 10^{-5} annual exceedance frequency, the percentage increase in spectral acceleration is 3.44 percent. The staff concludes that the percentage increase is even less for the 1 Hz hazard curve. The percentage increase in spectral acceleration at the 10^{-4} annual exceedance frequency is 0.39 percent when the Dames and Moore team's contribution is removed. At the 10^{-5} annual exceedance frequency, the percentage increase in spectral acceleration is 0.38 percent. Thus, in spite of the staff's concerns that the Dames and Moore team did not adequately characterize the regional seismic hazard, the staff considers open Item 2.5-1 to be resolved because the Dames and Moore team's contribution to the total mean hazard at the Vogtle ESP site is not significant.

Post-EPRI Seismic Source Characterization Studies

SSAR Section 2.5.2.2.2 describes three PSHA studies that were completed after the 1989 EPRI PSHA and which involved the characterization of seismic sources within the ESP site region. These three studies include the USGS National Seismic Hazard Mapping Project (Frankel et al. 1996, 2002), the SCDOT seismic hazard mapping project (Chapman and Talwani 2002), and the NRC TIP study (NUREG/CR-6607, "Guidance for Performing Probabilistic Seismic Hazard Analysis for a Nuclear Plant Site: Example Application to the Southeastern United States"). The applicant provided a description of both the USGS and SCDOT [South Carolina Department of Transportation] models, as well as the impact of these more recent studies on the EPRI PSHA models. The applicant did not, however, consider the TIP study to be a relevant source of information. The TIP study implemented the PSHA guidelines developed by the SSHAC (NUREG/CR-6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts") and focused on the development of seismic zonation and earthquake recurrence models for the Watts Bar (Tennessee), and Vogtle sites.

The applicant stated that it did not explicitly incorporate the results of the TIP study into the SSAR because "the study was more of a test of the methodology rather than a real estimate of the seismic hazard." Because part of the TIP study focused on the Vogtle site, the staff, in RAI 2.5.2-7, asked the applicant to explain why it concluded that the TIP study was more of a test of the methodology rather than a real estimate of the seismic hazard and why it did not use the TIP study results. In response, the applicant stated the following:

> The TIP study focuses primarily on implementing the Senior Seismic Hazard Advisory Committee (SSHAC) PSHA methodology (SSHAC 1997), however, and was designed to be as much of a test of the methodology as a calculation of seismic hazard. For example, as part of the test of the methodology, Committee members were asked to present opposing arguments, regardless of whether they agreed with the position they were asked to present. As a disclaimer, Kevin Coppersmith prefaced his discussion of the Pen Branch fault with the following statement:
>
> The following white paper—much like a lawyers (sic) legal argument—presents a particular position and seeks only to support that position. I have intentionally tried to present an unbalanced case, giving only lip service to counter arguments…Further, I have done a poor job of citing references and providing supporting data to many of my arguments (p. A-51).
>
> The TIP study provides useful discussions, including speculations regarding the Charleston seismic source, seismic hazards of the South Carolina–Georgia region, and Eastern Tennessee. However, the TIP study focuses primarily on methodology. The process-oriented focus of the TIP study is also illustrated in the report presentation, which is very thorough on methodology, but significantly lacking in presenting a summary of seismic source model parameters. For these reasons, the TIP study results are not explicitly incorporated into the VEGP ESP application.

The staff reviewed the applicant's response to RAI 2.5.2-7, as well as the TIP report, and disagrees with the applicant's conclusion that the TIP report was more of a test of the methodology rather than a real estimate of the seismic hazard.

The disclaimer provided in the applicant's response to RAI 2.5.2-7 accompanied a white paper titled, "Include the Pen Branch and Other Local Faults in the PSHA," written by Kevin Coppersmith after the first TIP workshop, which involved a panel of five expert evaluators, the technical facilitator/integrator (TFI) team, and expert proponents and presenters. The workshop comprised a series of technical sessions, which included presentations of recent research and interpretations by the presenters. Each of the technical sessions was followed by a discussion moderated by the TFI team in which key outstanding technical issues were defined. These key issues were then assigned to evaluators as the topics of "white papers" to be written after the workshop. For example, Kevin Coppersmith was assigned to write the white paper in support of "Discrete local fault sources for Vogtle," while Pradeep Talwani was assigned to present a case against "Discrete local fault sources for Vogtle." The TIP report states that "the objective of these papers is to clarify the arguments for and against key interpretations having direct bearing

on seismic source characterization in a way that will stimulate interaction among the evaluators." The TIP report also states that "the experts were asked to act as proponents of a certain scientific position and since the issues selected involved dichotomous positions they had to argue for a position that they do not necessarily defend. This has an advantage of forcing the experts, and all the participants, into discovering the positive aspects of scientific concepts other than their own." Thus, Kevin Coppersmith's disclaimer that accompanied his white paper merely reflects his assigned role to provide supporting arguments for a key workshop issue.

The staff concludes that, while the primary objective of the TIP study was to implement the SSHAC PSHA methodology, there is nothing to suggest that the project's final hazard results are not valid. In fact, the seismic hazard results from the TIP triggered a followup NRC-sponsored study, documented in Appendix G to NUREG/CR-6607, which involved a comparison of the TIP hazard results with NUREG-1488, "Revised Livermore Seismic Hazard Estimates for 69 Sites East of the Rocky Mountains." Therefore, although portions of the TIP report may have been focused on implementing the SSHAC methodology, much of the data and results contained in the report are applicable to the ESP site. Thus, in the SER with open items, the staff did not concur with the applicant's disposition of the TIP study. The staff requested that the applicant provide an evaluation of any information contained in the TIP study that is relevant to the seismic source characterization of the ESP site. The staff considered this information necessary in order to determine whether the applicant provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-2.

In response to Open Item 2.5-2, the applicant reiterated its position that the Trial Implementation Project (TIP) study was primarily an exercise in implementation of the SSHAC process. The applicant also stated the following:

> The fact that all final seismic source model parameters and weights are not presented in the TIP report also support that this study focused primarily on implementation of the SSHAC process as opposed to the development and publication of a new source model for the southeastern US. The absence of a complete set of parameters and weights in the TIP study also makes it difficult to replicate the entire source model and directly compare with some of the specific EPRI model parameters. The TIP report provides tables and figures that illustrate how the individual EVA's (experts) evaluated or weighted certain issues or parameters, but the report does not provide a final tabulation of all source parameters and weights that were used in the computation of hazard in the TIP study.

The applicant noted, however, that "the TIP report does present logic trees, tables, and plots that summarize different aspects of their seismic source characterization and uncertainty in several key parameters". The applicant also stated the following in support of the TIP study:

> However, the TIP study does address some key issues and provides assessments of these issues by the five experts assembled (Bollinger, Chapman, Coppersmith, Jacob, and Talwani) that can be evaluated and compared, in a more general sense, to the EPRI EST source model parameters. The TIP study

> included multiple workshops to define, clarify, and elicit expert opinion on several critical issues relating to the source characterization process and specific technical questions on seismic sources that were judged to be significant to the hazard at the Vogtle and Watts Bar sites.

As requested by the staff in Open Item 2.5-2, the applicant also presented an evaluation of information in the TIP study relevant to the seismic source characterization of the ESP site, including the ETSZ. The applicant stated that "Several of the key issues addressed in the TIP study support the wide range of uncertainty expressed in the EPRI EST seismic source characterizations for the ESP site."

In summary, the applicant acknowledged that the TIP study is a valid study and also provided an evaluation of information relevant to the seismic source characterization of the ESP site (see Open Item 2.5-3 for the applicant's discussion of the TIP study report with respect to the ETSZ). Therefore, the staff considers Open Item 2.5-2 to be closed.

Northwest of the ESP site, at a distance just beyond 200 miles, is the ETSZ zone. As shown in SER Figure 2.5.2-1, the ETSZ covers a cluster of earthquakes in eastern Tennessee. In SSAR Section 2.5.2.2.2.5, the applicant stated that, despite being one of the most active seismic zones in Eastern North America, the largest recorded earthquake recorded in the ETSZ is a magnitude 4.6, and no evidence for larger prehistoric earthquakes, such as paleoliquefaction features, has been discovered. The applicant also stated that, with the exception of the Law source 17 (Eastern Basement), none of the EPRI EST sources that included the ETSZ contributed more than 1 percent of the total hazard at the ESP site. For this reason, the applicant's hazard calculations did not include the sources that accounted for ETSZ seismicity, with the exception of Law source 17. The applicant also concluded that no new information regarding the ETSZ has been developed since 1986 that would require a significant revision to the original EPRI seismic source model, specifically with regards to Mmax for the ETSZ.

In RAI 2.5.2-16, the staff asked the applicant to provide the Mmax distributions and geographic coordinates defining the geometry of each EST-identified ETSZ. In response to RAI 2.5.2-16, the applicant provided the staff with the requested information and also stated the following:

> None of the EPRI-SOG teams specifically defined a zone identified as "Eastern Tennessee Seismic Zone." Each EPRI-SOG team did define one or more zones that encompass seismicity in eastern Tennessee and, in most cases, the surrounding regions.

The staff concludes that the information provided by the applicant, in response to RAI 2.5.2-16, is complete. SER Table 2.5.2-5 shows the Mmax distributions for the EPRI EST seismic sources that encompass seismicity in eastern Tennessee, provided by the applicant in its response to RAI 2.5.2-16.

Table 2.5.2-5 - Mmax Values Corresponding to the EPRI EST Seismic Source Zones That Encompass Seismicity in Eastern Tennessee (Provided by the Applicant In Response to RAI 2.5.2-5)

EPRI EST	Source	Description	Probability of Activity	Mmax (M) and Weights
Bechtel	24	Bristol Trends	0.25	5.31 [0.10] 5.66 [0.40] 6.06 [0.40] 6.49 [0.10]
	25	NY-AL Lineament	0.3	4.97 [0.10] 5.31 [0.40] 5.66 [0.40] 6.49 [0.10]
	25A	NY-AL Lineament (Alternative)	0.45	4.97 [0.10] 5.31 [0.40] 5.66 [0.40] 6.49 [0.10]
Dames & Moore	4	Appalachian Fold Belt	0.35	5.66 [0.80] 7.51 [0.20]
	4A	Kinks in Appalachian Fold Belt	0.65	6.82 [0.80] 7.51 [0.20]
Law Engineering	17	Eastern Basement	0.62	5.31 [0.20] 6.82 [0.80]
Rondout	13	Southern NY-AL Lineament	1	4.78 [0.30] 6.06 [0.55] 6.34 [0.15]
	24	Southern Appalachians	0.99	6.49 [0.30] 6.82 [0.60] 7.16 [0.10]
	27	TN-VA Border	0.99	4.78 [0.30] 6.06 [0.55] 6.34 [0.15]
Weston	24	NY-AL Clingman	0.9	4.97 [0.26] 5.66 [0.58] 6.49 [0.16]
Woodward-Clyde	31	Blue Ridge Combo	0.024	5.54 [0.33] 6.06 [0.34] 7.16 [0.33]
	31A	Blue Ridge Combo (Alternative)	0.211	5.54 [0.33] 6.06 [0.34] 7.16 [0.33]

In RAI 2.5.2-17, the staff asked the applicant to justify its rationale for not updating the ETSZ as characterized by the EPRI ESTs and to discuss how the Mmax distributions developed by each EST compare with more recent Mmax estimates for the ETSZ included in the USGS hazard model (Frankel et al. 2002) and Bollinger (1992). In addition, the staff asked the applicant to explain whether the contribution to the hazard would change if the EST source zones representing the ETSZ were assigned a single Mmax of M 7.5, or alternatively, to explain why it believes an Mmax value of M 7.5 with a weight of 0.5 or higher is not warranted for the ETSZ.

In response, the applicant concluded that the majority of the seismicity that defines the ETSZ is beyond the 200-mi site region. The applicant also noted that its update of the Charleston seismic source model (based on recent paleoliquefaction studies) has increased the relative contribution of the Charleston source to the ESP site and thus served to decrease the relative contribution of more distant sources such as the ETSZ. Furthermore, the applicant stated that there is no historic or prehistoric evidence for large magnitude events occurring in the eastern Tennessee area. In support of the low weights assigned by the EPRI ESTs for this region, the applicant stated the following:

> While the lack of evidence for past large events in ETSZ does not preclude large events from occurring in the future, this fact should influence the weighting of the Mmax distribution. It is therefore logical that the Mmax distribution for the ETSZ should have lower weights assigned to the largest magnitudes, in contrast to the Charleston and New Madrid sources, where there is a high confidence that those sources are capable of producing large events since they have occurred in the past.

In response to RAI 2.5.2-17, the applicant concluded that the EPRI EST maximum magnitude distributions for the ETSZ span the range of more recent assessments. The applicant's discussion focused on Bollinger's (1992) source model for the SRS. The applicant stated that Bollinger's (1992) Mmax of **M** 6.3, which was given a weight of 95 percent, is close to the mean maximum magnitude of ~**M** 6.2 of the EPRI study. The applicant also noted that Bollinger (1992) assigned a low weight of 5 percent to an Mmax of **M** 7.8, which was calculated based on a low probability that the dimensions of seismogenic structures within the zone may extend along the entire 300-km northeast-trending axis of the zone. The applicant also concluded that the TIP study (NUREG/CR-6607) provided a similarly broad Mmax magnitude distribution as did the EPRI distribution of M 4.8 to M 7.5 for the ETSZ. The applicant stated that the magnitude distributions for all TIP Study ETSZ source zone representations ranged from as low as **M** 4.5 to as high as **M** 7.5, with the mode of about **M** 6.5 for almost each distribution (NUREG/CR-6607, pages F-12 to F-19 of Appendix F).

In summary, the applicant concluded the following in its response to RAI 2.5.2-17:

> The ETSZ is characterized by abundant seismicity, but has yet to produce a recorded event greater than **M** 5, which is about the minimum magnitude used to characterize seismic sources in modern PSHA studies. In our opinion, we believe that there is sufficient uncertainty in the Mmax potential of the ETSZ that a broad range of magnitudes is appropriate and that the EPRI model sufficiently captures the range of more recent Mmax distributions for this source. While the

ETSZ may be capable of producing a **M** 7.5, we do not believe that a weight of 0.5 to 1.0 for this magnitude represents the range of expert opinion reflected in the post-EPRI studies by Bollinger (1992) and Savy et al. (2002). The exception, of course, is the USGS model that assigns a single magnitude of **M** 7.5.

The staff reviewed the applicant's response to RAI 2.5.2-17 and disagrees with the applicant that the ETSZ EPRI EST Mmax values adequately represent the ETSZ. Rather, the staff concludes that even though these EPRI EST sources have Mmax values as large as **M** 7.5, the corresponding weights are very low. In addition, the probabilities of activities of many of the ETSZ EPRI EST sources are also low. For example, in SER Table 2.5.2-5, the Dames and Moore Appalachian Fold Belt source has an Mmax value of **M** 7.5 and a weight of 0.20, and the probability of activity of this source is only 0.35.

SER Table 2.5.2-6 shows recent Mmax values for the ETSZ including Frankel et al. (2002), Chapman and Talwani (2002), and Bollinger (1992). A comparison of the two results shows that the EPRI Mmax values shown in SER Table 2.5.2-5 are significantly lower than more recent studies, as shown in SER Table 2.5.2-6. For example, Chapman and Talwani (2002) assigned a single Mmax of **M** 7.0 to the ETSZ. They noted that epicentral locations of the earthquakes define a major northeast-trending seismic zone, over 300 kilometers in length, suggesting the possibility of a major shock, if the zone is viewed as defining a through-going basement fault. Chapman and Talwani (2002) also stated that "focal mechanisms and the spatial locations of seismicity have revealed much information concerning this important issue, but the seismic hazard posed by this seismic zone remains uncertain."

Table 2.5.2-6 - Mmax Values for the ETSZ for Recent Studies

Study	Mmax (M) and Weights
Bollinger (1989)	6.2 [1.0]
Johnston and Chiu (1989)	7.2 [1.0]
Bollinger (1992)	5.7 [0.158] 6.1 [0.158] 6.2 [0.317] 6.5 [0.158] 7.2 [0.158] 7.8 [0.050]
Frankel et al. (2002)	7.5 [1.0]
Chapman and Talwani (2002)	7.0 [1.0]

Furthermore, as stated in the applicant's response above, none of the EPRI ESTs specifically defined a zone identified as the "Eastern Tennessee Seismic Zone." Each EPRI EST did define one or more zones that encompass seismicity in eastern Tennessee and, in most cases, the surrounding regions. In more recent studies, the seismicity within the ETSZ is explicitly developed into source geometries to account for the ETSZ (e.g., Frankel et al. 2002; Chapman and Talwani 2002; Bollinger 1992; and NUREG/CR-6607).

To validate the applicant's claim that the ETSZ hazard results are insignificant compared to the Charleston seismic source, the staff did a confirmatory analysis. The staff performed hazard

calculations using maximum magnitudes for the ETSZ that ranged from **M** 6.0 to **M** 7.8. This magnitude range reflects more recent Mmax values assigned to the ETSZ, as shown in SER Table 2.2.5-6. SER Figure 2.5.2-12 shows the staff's 1-Hz hazard curves for the ETSZ using this range of Mmax values. SER Figure 2.5.2-12 also shows the applicant's total mean hazard curve and the Charleston seismic source zone contribution for comparison. The staff's results show that, although the Charleston seismic source zone clearly dominates the 1-Hz hazard, the contribution from the ETSZ for some of the larger Mmax values (greater than 7.0) may contribute significantly more than 1 percent to the total hazard for the ESP site.

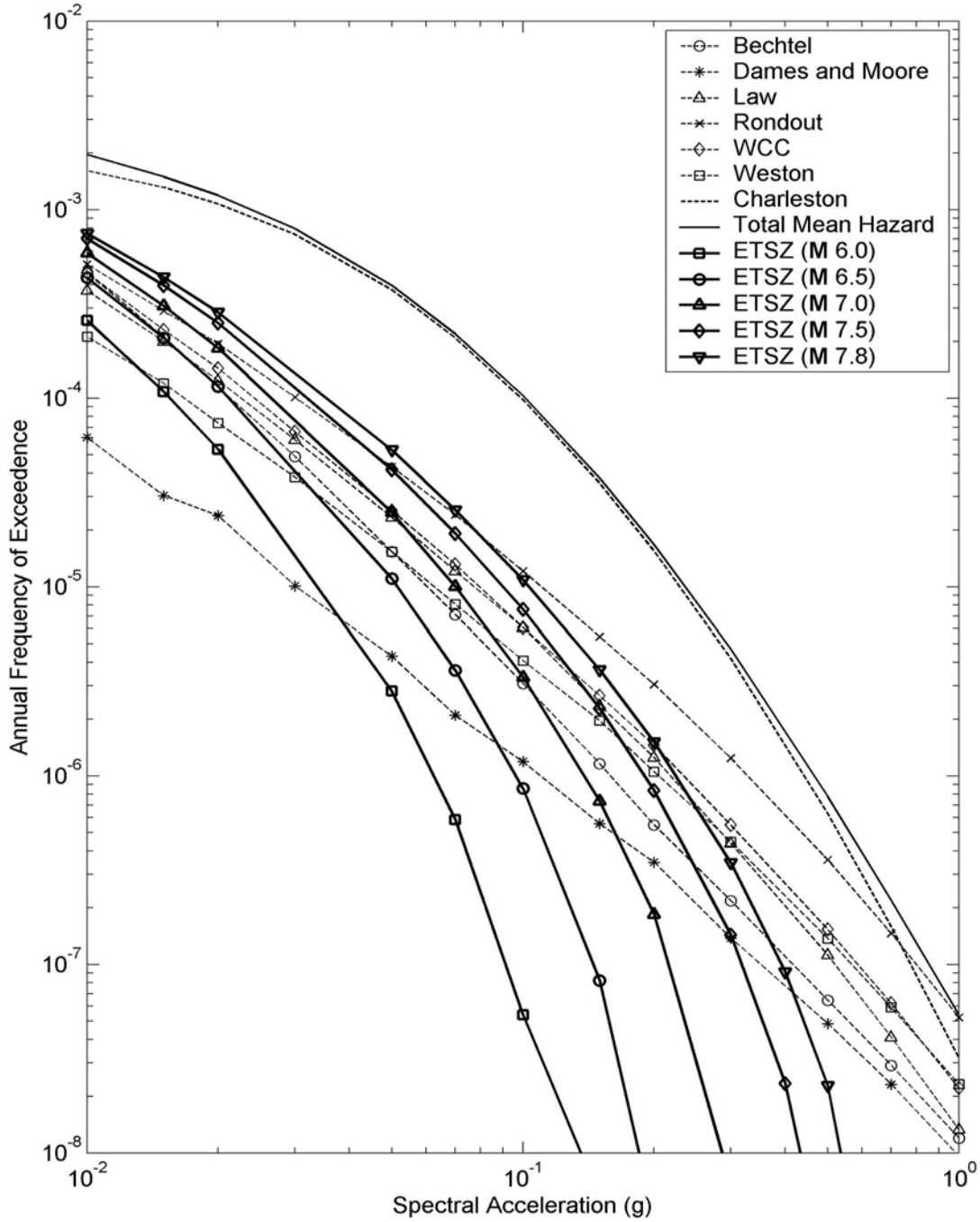


Figure 2.5.2-12 - Comparison of the staff's 1-Hz hazard curves for the ETSZ for magnitudes ranging from M 6.0 to M 7.8

The staff concluded that, despite the uncertainty regarding the potential for large earthquakes within the ETSZ, the results of post-EPRI source characterizations for the ETSZ suggest that the EPRI EST characterization of the ETSZ needs to be updated. The results of the staff's confirmatory analysis confirmed the applicant's assertion that the Charleston seismic source dominates the 1-Hz hazard. However, the staff concluded in the SER with open items that the contribution of the ETSZ at the ESP site may be significant enough to warrant inclusion in the applicant's PSHA, if larger Mmax values are considered. Accordingly, in the SER with open items, this issue was identified as Open Item 2.5-3.

In response to Open Item 2.5-3, the applicant stated the following:

> The Eastern Tennessee seismic zone (ETSZ) lies between the New York-Alabama and Ocoee aeromagnetic anomalies in what Kanter (1994) has classified as non-extended crust. Wheeler (1995; 1996) has defined this region associated with Eastern Tennessee seismicity as Late Proterozoic/early Paleozoic Iapetan extended crust. Based on the Johnston et al. (1994) study of stable continental cratons, the global seismicity database indicates that the largest historic earthquakes (**M**>7) are limited to Mesozoic extended crust. The Johnston et al. (1994) data base shows that Paleozoic non-extended crust has a mean Mmax of **M**6.4. Therefore, based on the global database, there is no analog to suggest that the ETSZ portion of the crust should fail in large (**M**>7) events.

As requested by the staff in Open Item 2.5-2, the applicant also provided an evaluation of the TIP study relevant to the seismic source characterization of the ESP site. In response to Open Item 2.5-3 (as well as in response to the staff's request in Open Item 2.5-2) the applicant provided the following evaluation of the ETSZ based on the TIP study:

> The Trial Implementation Project (TIP) study (Savy et al., 2002) identified the ETSZ as a key issue in assessing hazard for the Watts Bar site in Tennessee. While this study was primarily a trial implementation of the SSHAC process, the NRC has requested in Open Item 2.5-2 that we more closely examine information contained in the TIP study that is relevant to the seismic source characterization of the ESP site. The TIP study defined eight source zones to represent uncertainty in the geometry of the ETSZ and defined composite Mmax distributions for each source zone using the weighting schemes from each of the five experts. The composite Mmax distributions are presented graphically (pages F-12 through F-19 of the TIP study) for each of the ETSZ source zones, and are summarized in the table below with values of the minimum, maximum, and mode of the distributions.

Source Zone	Min	Mode	Max
4a1	4.5	6.5	7.5
4a1+2	5.0	6.5	7.5
4a1+2+3	5.0	6.5	7.5
4b1	5.0	6.5	7.5
4b2	5.0	6.5	7.5
4c	5.0	6.5	7.5
4d	5.0	6.5	7.5
4e	5.0	6.5	7.5

The magnitude distributions for all ETSZ source zone representations in the TIP study ranged from as low as **M**4.5 to as high as **M**7.5, with a mode of either **M**6.3 or **M**6.5 for each distribution. The modal values represent the greatest weight of the distributions, indicating that the experts participating in the trial implementation of the SSHAC Level 4 process felt that the majority of the weight belonged in the moderate magnitude events as opposed to the largest magnitudes. The broad distribution of the TIP study is similar to the distribution of **M**4.8 to **M**7.5 in the EPRI source zones.

The modal Mmax value for each of the TIP characterizations of the ETSZ is either **M**6.3 or **M**6.5. Even though the TIP study does not present discrete magnitudes and weights, the modal magnitudes suggest a mean magnitude on the order of ~**M**6.5 or less for the ETSZ.

In summary, the applicant concluded that ”Since no new data or evidence has been developed to imply large magnitude earthquakes in the ETSZ since the EPRI study, there is no basis for rejecting the Mmax interpretations of the EPRI teams, which cover the range of Mmax employed in more recent seismic source characterizations. Therefore additional calculations of seismic hazard with larger Mmax values for the ETSZ would be purely speculative and could not form a basis for conclusions.”

The staff disagrees with the applicant’s conclusions that additional calculations of seismic hazard with larger Mmax for the ETSZ are not warranted. The staff notes that there are more recent seismic hazard studies, such as the LLNL TIP study and the Geomatrix TVA Dam safety study, which provide new information on the seismic hazard of the area. Furthermore, the staff does not agree with the applicant’s conclusion that the EPRI team’s Mmax composite distribution for the ESTZ is similar to that of more recent studies. The applicant only compared the range of the Mmax values of the EPRI study rather than the actual weighted values. SER Figure 2.5.2-13 clearly shows that more recent studies place a significantly higher probability on larger maximum magnitude earthquakes than the EPRI study. The mean Mmax for the TIP (i.e. Savy et al., 2002) and Geomatrix studies are approximately **M**6.55 and **M**6.58, respectively.

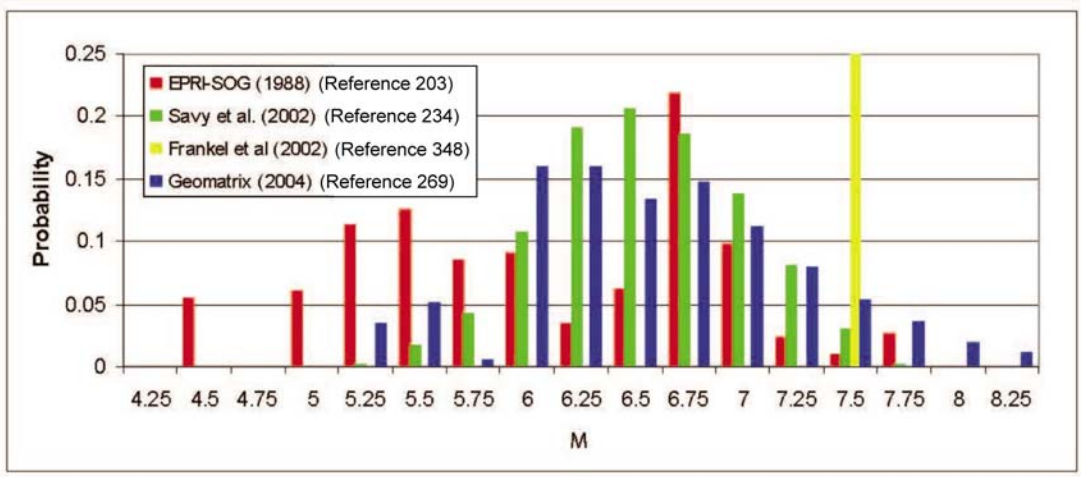


Figure 2.5.2-13. Composite EPRI-SOG distribution in terms of M compared to more recent assessments (reproduced from the Bellefonte RCOL application)

The staff concludes, however, that the contribution of the ESTZ at the Vogtle ESP site is insignificant, even when Mmax values comparable to the mean Mmax values for the TIP and Geomatrix studies are considered. Based on the staff's sensitivity study, presented in SER Figure 2.5.2-12, a mean magnitude of **M**6.5 for the ETSZ contributes to less than 1 percent of the total hazard at 1 Hz for ground motions critical for design levels (0.1 g and higher). Therefore, the staff considers Open Item 2.5-3 to be resolved.

Updated EPRI Seismic Sources

Based on the results of several post-EPRI PSHA studies (Frankel et al. 2002; Chapman and Talwani 2002) and the recent availability of paleoliquefaction data (Talwani and Schaeffer 2001) for the Charleston source zone, the applicant updated the EPRI characterization of the Charleston seismic source zone as part of the ESP application. The applicant referred to its update as the UCSS model. The staff focused its review on the applicant's UCSS geometry, Mmax values, and recurrence model. The staff also reviewed the methodology that the applicant used to perform this update.

SSHAC Update of the Charleston Seismic Source. In SSAR Section 2.5.2.2.2.4, the applicant noted that the UCSS model is described in detail in a 2006 Bechtel engineering study report. In order to review the applicant's UCSS model, the staff, in RAI 2.5.2-2, requested a copy of the Bechtel (2006) report. In response to RAI 2.5.2-2, the applicant provided the staff with a copy of Bechtel (2006). Based on its review of the Bechtel (2006) report, the staff gained additional insight regarding the applicant's UCSS model.

As described in Bechtel (2006), the applicant performed an SSHAC Level 2 study to incorporate current literature and data, as well as the understanding of experts, into an update of the Charleston seismic source model. An SSHAC Level 2 study uses an individual, team, or

company to act as a Technical Integrator (TI), who is responsible for reviewing data and literature and contacting experts who have developed interpretations of or who have specific knowledge about the seismic source. The TI for the update of the Charleston seismic source model consisted of a team of six William Lettis & Associates, Inc. (WLA) personnel (Scott Lindvall, Ross Hartleb, William Lettis, Jeff Unruh, Keith Kelson, and Steve Thompson). The WLA TI team first compiled and reviewed all new information developed since 1986 regarding the 1886 Charleston earthquake and the seismic source that may have produced this earthquake and then compared this new information with the 1986 EPRI EST assessments of the Charleston seismic source. Following the literature review, the TI conducted interviews with experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. The TI consulted the following seismic and geologic experts:

- Dr. David Amick, Science Applications International Corporation
- Dr. Martin Chapman, Virginia Polytechnic Institute
- Dr. Chris Cramer, U.S. Geological Survey
- Dr. Art Frankel, U.S. Geological Survey
- Dr. Arch Johnston, Center for Earthquake Research and Information, University of Memphis
- Dr. Richard Lee, Los Alamos National Laboratory
- Dr. Joe Litehiser, Bechtel Corporation (original team leader of the 1986 Bechtel EST)
- Dr. Stephen Obermeier, U.S. Geological Survey (retired)
- Dr. Pradeep Talwani, University of South Carolina
- Dr. Robert Weems, U.S. Geological Survey

The TI next integrated this information to develop an updated characterization of the Charleston seismic source that captures the composite representation of the informed technical community.

In RAI 2.5.2-4, the staff asked the applicant to justify its rationale for selecting an SSHAC Level 2 methodology for the UCSS update, as opposed to a higher level update. To support its rationale for using the SSHAC Level 2 methodology, the applicant stated the following:

> SSHAC (1997) describes four levels of study (Levels 1 through 4), in increasing order of sophistication and effort. The choice of the level of a PSHA is driven by two factors: (1) the degree of uncertainty and contention associated with the particular project, and (2) the amount of resources available for the study (SSHAC 1997). SSHAC (1997, Table 3-1) suggests that a Level 2 study is appropriate for issues with “significant uncertainty and diversity,” and for issues that are “controversial” and “complex.” In a SSHAC Level 2 study, a Technical Integrator (TI) is responsible for reviewing data and literature and contacting experts who have developed interpretations or who have specific knowledge of the seismic source. The TI interacts with experts to identify issues and interpretations, and to assess the range of informed expert opinion. In Level 3 studies, the TI goes a step further by bringing together experts and focusing dialog and interaction between them in order to evaluate relevant issues. In Level 4 studies, a Technical Facilitator/Integrator (TFI) is responsible for aggregating the judgments of a panel of experts to develop a composite distribution of the informed technical community. In a meeting held on

> July 7, 2005, VEGP ESP Technical Advisory Group (TAG) members Dr. Martin Chapman, Dr. Robert Kennedy, Dr. Carl Stepp, and Dr. Robert Youngs agreed that a Level 2 study is appropriate for updating the Charleston seismic source model.

In RAI 2.5.2-4, the staff also asked the applicant to describe its implementation of the SSHAC Level 2 methodology. Specifically, the staff asked the applicant to describe in more detail how the expert's opinions were integrated into the development of the final UCSS model, how any conflicting opinions between the experts were dealt with, and how the final source model represents the informed consensus of the community beyond those queried for the UCSS update. In response, the applicant stated that, as part of the SSHAC process, the TI contacted 10 experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. The applicant stated the following:

> These experts were asked a series of questions pertaining to key issues regarding the Charleston seismic source. This was not a formal process of expert interrogation to obtain from each expert all of the specific parameters and weights to be used in the model. Instead, we allowed the experts to speak to their own areas of expertise. It was then the TI's responsibility to combine these responses with data from the published literature to capture the range of expert opinion and judgment regarding parameters and weights to be used in the UCSS model.

Regarding the TI integration of the expert's opinion into the development of the final UCSS model, the applicant provided the following information:

> This activity included a two-day workshop held on September 13–14, 2005 to develop the UCSS model at the WLA office in Valencia, California after several weeks of literature and data review. The workshop included the TI team, who integrated Charleston area data and expert interpretations, discussed uncertainties and conflicting expert interpretations, and developed UCSS geometries and the logic tree.

The applicant also stated the following regarding the review of the UCSS model by the TAG panel:

> A Technical Advisory Group (TAG) panel was convened in April 2006 in Frederick, Maryland to critically review the UCSS model and to provide feedback regarding the process and the results of the study. TAG members Chapman, Kennedy, Stepp, and Youngs were in attendance. In addition, Dr. Carl Stepp and Dr. Martin Chapman reviewed written copies of the Engineering Report describing the UCSS and provided written comments on, and approval of, the document.

With regard to how the final source model represents the informed consensus of the community beyond those queried for the UCSS update, the applicant stated, "for the VEGP ESP study, a Senior Seismic Hazard Analysis Committee (SSHAC) Level 2 study was performed to

incorporate current literature and data and the understanding of experts into an update of the Charleston seismic source model," and that "the intent of the SSHAC process is to represent the range of current understanding of seismic source parameters by the informed technical community."

Based on its review of SSHAC (1997) and the Bechtel (2006) report provided by the applicant in response to RAI 2.5.2-2, as well as the applicant's response to RAI 2.5.2-4, the staff concludes that the applicant's overall implementation of the SSHAC Level 2 process is adequate. In accordance with an SSHAC Level 2 study, the applicant established a TI, comprising six WLA personnel, to conduct a literature review and contact experts and researchers familiar with geologic/seismologic data and recent characterizations of the Charleston seismic source. As defined in the SSHAC report, a TI is "a single entity (individual, team, or company, etc.) who is responsible for ultimately developing the composite representation of the informed technical community." Also in accordance with SSHAC, the applicant selected a peer review panel to "critically review the UCSS model and to provide feedback regarding the process and results of the study." The applicant referred to its peer review panel as the VEGP ESP TAG. The TAG consisted of Dr. Martin Chapman, Dr. Robert Kennedy, Dr. Carl Stepp, and Dr. Robert Youngs. According to the 1997 SSHAC report, the purpose of the peer review panel is to "assure that the process followed was adequate and to ensure that the results provide a reasonable representation of the diversity of views of the technical community."

The staff also concludes that the applicant's selection of an SSHAC Level 2 study is appropriate for the update of the Charleston seismic source zone. As shown in SER Table 2.5.2-7 (reproduced from Table 3-1 of the 1997 SSHAC report), the SSHAC criteria for deciding on the level of the study is rather subjective. The 1997 SSHAC report suggests that Level 2 studies are appropriate for issues with "significant uncertainty and diversity," and for issues that are "controversial" and "complex," while Level 3 and 4 studies are appropriate for issues that are "highly contentious; significant to hazard; and highly complex." SSHAC (1997) also states that Level 3 and 4 studies "are resource-intensive and are, therefore, most appropriate for large-scale studies for critical facilities." Thus, based on the guidance provided in SSHAC (1997), and because the applicant's study involved the update of a single seismic source zone, the staff agrees with the applicant's decision to use an SSHAC Level 2 study.

Table 2.5.2-7 - Degrees of PSHA Issues and Levels of Study (from SSHAC (1997), Table 3-1, p. 23)

ISSUE DEGREE	DECISION FACTORS	STUDY LEVEL
A Noncontroversial and/or insignificant to hazard	Regulatory concern Resources available Public perception	1 TI evaluates/weights models based on literature review and experience; estimates community distribution
B Significant uncertainty and diversity; controversial; and complex		2 TI interacts with proponents and resource experts to identify issues and interpretations; estimates community distribution
C Highly contentious; significant to hazard; and highly complex		3 TI brings together proponents and resource experts for debate and interaction; TI focuses debate and evaluates alternative interpretations; estimates community distribution
		4 TFI organizes panel of experts to interpret and evaluate; focuses discussions; avoids inappropriate behavior on part of evaluators; draws picture of evaluators' estimate of the community's composite distribution; has ultimate responsibility for project

Although the staff concurs with the applicant's selection and overall implementation of an SSHAC Level 2 method to update the Charleston seismic source model, its review of Bechtel (2006) resulted in several additional questions. For example, the staff was unable to determine the actual questions that each of the experts involved in the SSHAC Level 2 study were asked, the range of expert opinions related to key aspects of the UCSS model (i.e., recurrence, geometry, and maximum magnitude), or the specific process used to combine the expert's opinions and resolve any differing opinions. On June 18, 2007, the applicant supplemented its response to RAI 2.5.2-4 with additional information regarding its SSHAC Level 2 study. Because the staff received this information late in the review process, the staff identified this as Open Item 2.5-4 in the SER with open items, to allow additional time to complete the review. The staff also requested the applicant to explain why only two of the four members of the TAG panel reviewed and approved written copies of the engineering report describing the UCSS, as stated in response to RAI 2.5.2-4.

In its supplemental response to RAI 2.5.2-4, the applicant provided the staff with the list of questions that the technical integrator developed and used as its basis for communicating with

researchers by telephone. These questions covered the main issues involving the Charleston earthquake process, geometry, maximum magnitude (Mmax), and recurrence. The applicant also provided the responses given by each of the experts. The applicant noted that some of the experts limited their responses to their own specific area of expertise. For example, Stephen Obermeier (USGS, retired) provided comments and insight on paleoliquefaction data, but did not wish to comment on specific questions regarding source geometry modeling and other parameters. In addition, the applicant also stated that in some interviews, selected questions were not asked if the topic was outside the expert's research area or if the interview was limited on time.

The applicant's supplemental response to RAI 2.5.2-4 also describes how the expert's opinions were integrated into the development of the final UCSS model, and how any conflicting opinions between the experts were dealt with. The applicant stated that "because the SSHAC Level 2 process does not involve bringing the experts together, there was not a forum for experts to directly question or challenge each other's assumptions or results and formally resolve any conflicting opinions." The applicant noted that "in the compilation of literature and expert opinions, there were instances where one expert's opinions differed from others." The applicant further noted that "in these cases, it is the responsibility of the Technical Integrator (TI) to "evaluate the viability and credibility of the various hypotheses with an eye toward capturing the range of interpretations, their credibilities, and uncertainties" (SSHAC 1997). The applicant stated that "conflicting opinions were included in the model parameters in an effort to capture the range of opinion and uncertainty."

In Open Item 2.5-4, the staff also requested the applicant to explain why only two of the four members of the Technical Advisory Group (TAG) panel reviewed and approved written copies of the engineering report describing the Updated Charleston Seismic Source (UCSS), as stated in its response to RAI 2.5.2-4. In response to Open Item 2.5-4, the applicant stated the following:

> The Updated Charleston Seismic Source (UCSS) model was presented to the entire Technical Advisory Group (TAG) panel in meetings on April 12-13, 2006. As such, the TAG performed participatory peer review of the UCSS, including reviewing the approach (i.e., SSHAC Level 2), data, and results of the updated model. The TAG panel consisted of three seismologists and one structural engineer. It was decided that it would be in the best interest of the project to also have a detailed review of UCSS engineering report by members of the TAG. The two seismologists most familiar with the tectonics and seismicity of the southeastern US, Dr. Martin Chapman and Dr. Carl Stepp, were requested to review written copies of the engineering report and provide comments.

The staff reviewed the applicant's responses to RAI 2.5.2-4 and Open Item 2.5-4. Based on its review, the staff concludes that the applicant adequately performed a SSHAC Level 2 study to update the Charleston seismic source zone. The staff concludes that the list of questions that the TI asked the experts generally addressed the key aspects of the UCSS model, and that the applicant's UCSS adequately captured the range of expert's input, when provided. The staff further concludes that the TI adequately integrated the range of expert's responses, where appropriate, into the final UCSS along with its findings based on its review of current literature

and paleoliquefaction data. In addition, the staff considers it appropriate that only two of the TAG panel members performed a detailed review the final UCSS because these members had the most familiarity with the tectonics and seismicity of the southeastern US.

Paleoliquefaction features of the Charleston seismic source zone. Abundant soil liquefaction features induced by the 1886 Charleston earthquake, in addition to other large prehistoric earthquakes (dating back to the mid-Holocene), are preserved in geologic deposits at numerous locations within the 1886 meizoseismal area and along the South Carolina coast. SSAR Section 2.5.2.2.2.4.1 states that the characteristics of the 1886 Charleston earthquake, combined with the greatest density of prehistoric liquefaction features, "show that future earthquakes having magnitudes comparable to the Charleston earthquake of 1886 most likely will occur within the area defined by Geometry A. A weight of 0.7 is assigned to Geometry A". Additionally, SSAR Figure 2.5.2-9 indicates no likelihood that an 1886-sized earthquake has occurred inland from the coastal region, except along Geometry C, and then only with a probability of 0.1. In RAI 2.5.2-8, the staff asked the applicant to summarize the age, liquefaction susceptibility, and geographic distribution of liquefiable deposits in the zone that is 50 to 150 kilometers (31 to 93 miles) inland from the coast and explain whether this information supports a negligible probability of large inland earthquakes. In addition, in RAI 2.5.2-8, the staff requested that the applicant reconcile the negligible probability of large inland earthquakes, as indicated in SSAR Figure 2.5.2-9, with the discovery of prehistoric liquefaction features as much as 100 kilometers (62 miles) inland in fluvial deposits of the Edisto River (Obermeier 1996). In response to RAI 2.5.2-8, the applicant stated the following:

> Liquefaction susceptibility is a function of numerous variables included to, sediment grain size and sorting, degree of compaction and/or cementation, deposit thickness, depth below ground surface, degree of saturation, and sediment age. Obermeier (1996) suggested that South Carolina Coastal Plain deposits older than about 250 ka have negligible potential for liquefaction due to the effects of chemical weathering. Obermeier (1996) observed that, in general, the region within 30 mi (~50 km) of the coast is highly susceptible to liquefaction. The liquefiable deposits of the about 100 ka Princess Anne Formation, however, are mapped greater than 65 mi inland (McCartan et al. 1984).
>
> Numerous liquefaction features caused by the 1886 Charleston earthquake and paleoliquefaction features from prehistoric Events A, B, C', E and F' are distributed along a 115 mi stretch of coastal South Carolina from Bluffton in the south to Georgetown in the north. The inland extent of 1886 liquefaction is less well-constrained.
>
> There is no structural, geomorphic, paleoseismic (other than the cited sparse liquefaction data), or historic (i.e., 1886) evidence to suggest a source zone geometry that trends northwest-southeast or extends significantly inland from the 1886 meizoseismal area. The sparse liquefaction features along the Edisto River cited by Seeber and Armbruster (1981), Amick et al. (1990), and Obermeier (1996) likely reflect strong ground shaking in deposits susceptible to liquefaction, and not a localized, inland source.

The staff agrees that the applicant's response adequately summarized the age, liquefaction susceptibility, and geographic distribution of liquefiable deposits in the zone 50–150 kilometers (31–93 miles) inland from the South Carolina coast. However, it is the staff's opinion that the applicant, in its RAI response, did not provide substantial evidence to rule out the occurrence of large inland earthquakes, especially given the presence of liquefiable deposits greater than 100 kilometers (65 miles) inland from the coast. The occurrence of a large earthquake inland from the coast would necessitate a different Charleston source zone model. Accordingly, in the SER with open items, the staff identified this issue as Open Item 2.5-5. In Open Item 2.5-5, the staff asked the applicant to provide supporting evidence to rule out the occurrence of large inland earthquakes.

In response to Open Item 2.5-5, the applicant explained that it would be difficult to provide direct evidence that large earthquakes have not occurred inland from Charleston. The applicant described liquefaction and paleoliquefaction features that have been documented by a number of researchers along the Edisto River as far as 70 km (45 mi) inland from the coast. The applicant considered these sites to represent liquefaction and paleoliquefaction features documented farthest inland from the coast. The applicant explained that most researchers do not document negative findings for inland liquefaction features and provided the following statement:

> Various researchers (e.g., Amick et al. 1990, Obermeier 1996) have published maps depicting the geographic distribution of 1886 liquefaction and paleoliquefaction sites in coastal South Carolina and along the eastern seaboard. These researchers do not, however, thoroughly document their reconnaissance of the rivers and drainage ditches that lack features indicative of strong ground shaking inland from the Charleston meizoseismal area, other than to say none was observed inland.

The applicant also provided additional supporting information in the form of documented expert opinion regarding the likelihood of large inland earthquakes. The following statement by the applicant details the opinions of Stephen Obermeier (U.S. Geological Survey, retired), an expert in eastern U.S. liquefaction and paleoliquefaction:

> Obermeier discussed the areas reconnoitered as part of his and others' research into South Carolina coastal plain liquefaction sites. There are no published maps that show in detail those areas studied but in which no liquefaction features were recognized. According to Obermeier, Figure 7.6 from Obermeier (1996) represents the best published approximation of the areas of investigation. This figure indicates that, with the exception of the Edisto River, the search for liquefaction features extended roughly 12 to 30 mi (20 to 50 km) inland throughout South Carolina. Reconnaissance along the Edisto River extended to roughly 45 mi (70 km) from the coast and represents the inland-most extent of the search for liquefaction features. Reconnaissance was conducted inland along the Edisto River in part because the banks of this river and its associated drainage ditches, more so than most in South Carolina, provide relatively good geologic exposure in which liquefaction features may be recognized.

The applicant compared the geographic distribution of the inland Edisto River liquefaction features to those found along the coast and made the following statement:

> It is instructive to note that these Edisto River liquefaction sites are closer to the Charleston meizoseismal area (<40 miles) than are the liquefaction sites up and down the coast that experienced liquefaction during the 1886 event (~100 miles). These observations indicate that the local Charleston source is capable of producing the observed inland liquefaction features along the Edisto River.

The applicant also provided the following statement contained in the TIP study (Savy et al., 2002) to further support a local Charleston source rather than an inland source for producing large earthquakes:

> The hazard at the Vogtle plant will be sensitive to the northwestern and western extents of the Charleston source. There appears to be no compelling reason to extend the source to the northwest from the 1886 epicentral area by connecting the Summerville-Middleton Place and Bowman zones of microseismicity. Dave Amick has found no paleoliquefaction evidence for strong ground shaking in the Bowman area, and the microseismicity there is much shallower than in the epicentral area. (p.19)

The applicant stated that while it is difficult to provide conclusive evidence that a large earthquake would not occur inland from the coast, many large areal source zones contained in the EPRI source model allow for potential large earthquakes to occur throughout the southeastern U.S. and thus would account for the possibility of a large inland earthquake outside of the local Charleston source.

While the applicant's position for supporting a negligible probability of large inland earthquakes does not rule out the potential for large inland earthquakes to occur, the staff believes that the applicant provided adequate documentation to support the likelihood of a local Charleston source rather than a source inland from the coast. The staff found the applicant's submittal of expert opinion regarding previous documentation of inland historic and prehistoric liquefaction features to be sufficient to support the applicant's evaluation. Only a handful of sites inland from Charleston along the Edisto River provide evidence for earthquake-induced liquefaction and most researchers do not document a lack of evidence in their observations. While numerous factors contribute to the liquefaction susceptibility at a site, liquefiable sediments are known to be present greater than 100 km (65 mi) inland from the coast, with minimal evidence for liquefaction observed.

The lack of more abundant earthquake-induced liquefaction features observed farther inland coupled with the presence of features extending more than 100 miles along the coast, and mostly equidistant from Charleston, does not prove large inland earthquakes have not occurred but rather suggests a more likely centralized earthquake source closer to Charleston. The staff concurs with the applicant that it would be difficult to provide direct evidence against the occurrence of large inland earthquakes. Furthermore, the staff concludes that the information provided by the applicant in support of a localized Charleston earthquake source, rather than an

inland earthquake source, is adequate based on evidence in the existing literature as well as expert opinion regarding actual observed liquefaction features. Therefore, the staff considers Open Item 2.5-5 to be resolved.

With regard to the size and quantity of earthquakes that produced the Charleston area liquefaction features, SSAR Section 2.5.2.2.2.4.3 suggests that the liquefaction features attributed by researchers to a single large, prehistoric earthquake might actually have been produced by several moderate magnitude earthquakes that are closely spaced in time (SSAR, page 2.5.2-26). In RAI 2.5.2-9, the staff asked the applicant to determine whether Talwani or Obermeier, two recognized experts, have data on the sizes of prehistoric liquefaction craters and whether these or any related data might constrain the possible magnitudes of the prehistoric earthquakes.

In response to RAI 2.5.2-9, the applicant explained that it is possible to compare the 1886 earthquake liquefaction features with liquefaction features attributed to pre-1886 events. The applicant further explained that some pre-1886 features suggest an earthquake magnitude similar to the 1886 Charleston earthquake. The applicant provided the following evidence:

> Obermeier (1996) noted "almost all craters that predate 1886 have a morphology and size comparable to the 1886 craters" (p.345). Moreover, the sizes of individual craters formed during the 600 and 1,250 years BP events are at least as large as those formed during the 1886 earthquake, both in the vicinity of Charleston and farther away (Obermeier 1996). These observations suggest that some prehistoric earthquakes have been at least as large as the 1886 earthquake.

The applicant cited a number of references, including Talwani and Schaeffer (2001), Hu et al. (2002a, 2002b), Leon (2003), and Leon et al. (2005), each of which attempted in some degree to estimate earthquake magnitudes associated with liquefaction features over the extended, as well as more limited, areas in the Charleston vicinity. According to the applicant, the magnitude estimates based on these studies vary widely, from **M** 7+ (Talwani and Schaeffer 2001) to **M** 6.8–7.8 (Hu et al. 2002b) to M 6.9–7.1 and **M** 5.6–7.2 (Leon et al. 2005) for earthquakes associated with widespread liquefaction features. Magnitude estimates for earthquakes producing liquefaction features over more limited areas vary similarly from M 6+ (Talwani and Schaeffer 2001) to **M** 5.5–7.0 (Hu et al. 2002b) to **M** 5.7–6.3 and **M** 4.3–6.4.

The applicant concluded that, even with the large uncertainties attached to estimating magnitudes from paleoliquefaction data, and in turn reflecting broad magnitude estimates for prehistoric earthquake events, the studies cited suggest that at least some of the prehistoric earthquakes have been similar in magnitude to the 1886 Charleston earthquake. Specifically, the applicant's response indicates that pre-1886 liquefaction craters "have a morphology and size comparable to the 1886 craters." This statement indicates that 1886 and pre-1886 liquefaction craters have similar maximum sizes, with ground conditions and hypocentral depths being similar, which implies similar historic and prehistoric earthquake magnitudes.

While the applicant's reasoning does not rule out the occurrence of numerous smaller earthquakes, the staff believes that the applicant made an accurate assumption that earthquake

magnitudes for pre-1886 earthquakes in the Charleston area are similar to the magnitude range attributed to the 1886 event based on the documentation of large liquefaction craters induced by both 1886 and pre-1886 earthquakes. As such, the staff concludes that the applicant conservatively assumed that the pre-1886 earthquakes were similar in magnitude to the 1886 event.

In RAI 2.5.2-10, the staff asked the applicant to summarize, for each of the pre-1886 events, the number of liquefaction features and sites that have been documented, the areal extent of liquefaction (i.e., the number of square kilometers affected), the number of dates that have been collected, and how well the features correlate from one site to the next.

In response to RAI 2.5.2-10, the applicant summarized the methods used in the application to constrain the timing of liquefaction-inducing earthquakes and referenced SSAR Table 2.5.2-13 to provide an age comparison of Charleston liquefaction events (Talwani and Schaeffer 2001). The applicant provided the following background information:

> Talwani and Schaeffer (2001) used calibrated radiocarbon ages with 1-sigma error bands in order to define the timing of past liquefaction episodes in coastal South Carolina. The standard in paleoseismology, however, is to use calibrated ages with 2-sigma (95.4 percent confidence interval) error bands (e.g., Sieh et al. 1989; Grant and Sieh 1994). Likewise, in paleoliquefaction studies, in order to more accurately reflect the uncertainties in radiocarbon dating, the use of radiocarbon dates with 2-sigma error bands (as opposed to narrower 1-sigma error bands) is advisable (Tuttle 2001).

Because Talwani and Schaeffer used calibrated ages with 1-sigma error bands, the applicant recalibrated Talwani and Schaeffer's (2001) radiocarbon data using 2-sigma error bands and presented the new data in the application. The applicant stated that the use of 1-sigma error bands by Talwani and Schaeffer (2001) possibly led to an overinterpretation of the paleoliquefaction record such that Talwani and Shaeffer (2001) may have interpreted more episodes than what actually occurred. The applicant used the 2-sigma recalibrated data to obtain broader age ranges for pre-1886 earthquake-induced liquefaction events. The applicant provided the following additional information:

> Paleoearthquakes were distinguished based on grouping paleoliquefaction features that have contemporary radiocarbon samples with overlapping calibrated ages. The event ages were then defined by selecting the age range common to each of the samples. For example, an event defined by overlapping 2-sigma sample ages of 100 to 200 cal yr BP and 50 to 150 cal yr BP would have an event age of 100 to 150 cal yr BP. We consider the "trimmed" ages to represent the ~ 95 percent confidence interval, with a "best estimate" event age as the midpoint between the ~ 95 percent age range.
>
> The 2-sigma analysis identified six earthquakes (including 1886) in the data presented by Talwani and Schaeffer (2001). As noted by that study, events C and D are indistinguishable at the 95 percent confidence interval, and together they compose Event C'. Additionally, our 2-sigma analysis suggests that Talwani

and Schaeffer's (2001) events F and G may have been a single, large event, which we name Event F'.

The applicant provided a summary of the approximate number of documented liquefaction features, the areal extent of those features, and the number of radiocarbon dates collected for each of the prehistoric earthquake events (A, B, C', E, F') as well as for the 1886 event. SER Figure 2.5.1-11, in response to RAI 2.5.1-10, provides a means of visually correlating liquefaction features from one site location to the next and from one event to another.

Based on its review of the applicant's response to RAI 2.5.1-10, the staff concludes that the applicant adequately summarized the documented liquefaction features associated with 1886 and pre-1886 earthquake events. The data provided by the applicant are useful in evaluating the uncertainty associated with each of the prehistoric earthquake events and in correlating similarities between events in order to better estimate possible magnitudes and source location.

SSAR Section 2.5.2.2.2.4.3 states that paleoliquefaction Event C is defined by features north of Charleston, while Event D is defined by sites south of Charleston. Events C and D are combined into a single large event, C'. In RAI 2.5.2-11, the staff requested the applicant to provide any information on liquefaction features, geographically located between these two areas, that have similar radiocarbon ages, which would support the characterization of these events as a single large event rather than two separate events. The staff also asked the applicant to provide justification that there is enough paleoliquefaction data to support a single large event C' from a single source.

In response to RAI 2.5.2-11, the applicant stated that using 2-sigma calibration for evaluating radiocarbon dates associated with Talwani and Schaeffer (2001) events C and D, based on timing alone, provides evidence that these events are indistinguishable at the 95 percent confidence interval. The applicant combined the two events into a single event, C'. Talwani and Schaeffer (2001) themselves interpreted an alternate scenario for these two events, also based on 2-sigma calibration of the data, and referred to a possible single event, C'.

The applicant provided a visual depiction of this information (SER Figure 2.5.2-14) to allow a comparison of liquefaction features associated with Talwani and Schaeffer (2001) events C and D to determine any overlap that could provide further evidence that these two events should be combined into a single event, C'. The applicant stated that liquefaction features associated with events C and D are localized and do not show any spatial overlap and "therefore do not provide definitive geographic evidence for combining these events into a single, large event C'." However, the applicant chose to include a single, large event C' (as opposed to two smaller events C and D) into the updated Charleston seismic source model based on the following three reasons:

1. The two-sigma reanalysis of Talwani and Schaeffer's (2001) age data performed for the VEGP ESP application indicates that the age data constraining the timing of Events C and D overlap one another and therefore the two events are indistinguishable. This observation is consistent with the interpretation of a single, large Event C'.

2. The incorporation of a single, large Event C' into the updated Charleston seismic source model is, in effect, a conservative approach. In developing a recurrence interval for large, characteristic earthquakes in the updated Charleston seismic source model, it was desirable to include the possibility that Events C and D represent a single, large earthquake. Talwani and Schaeffer's (2001) moderate-magnitude (~M 6) earthquakes C and D would be eliminated from the record of large (Mmax) earthquakes in the updated Charleston seismic source model, thereby increasing the calculated Mmax recurrence interval and lowering the hazard without sufficient justification.

3. The distribution of paleoliquefaction sites for Event C' is very similar to the coastal extent of liquefaction features from the 1886 earthquake. Moreover, the distribution and number of paleoliquefaction sites for Event C' are very similar to those for Events A and B, the two best documented prehistoric events (SER Figure 2.5.2-15).

Based on its review of the applicant's response to RAI 2.5.2-11, the staff acknowledges that recalibration of radiocarbon ages shows that the ages of events C and D are indistinguishable at a 95.4 percent confidence interval and that the applicant's decision to combine the two events into a single larger event, C', is justified. Geographic distribution of liquefaction features associated with a single large event C' is comparable to distribution of features associated with the 1886 Charleston earthquake and prehistoric earthquake events A, B, E and F'. The effect is to decrease the average recurrence interval of 1886-sized earthquakes from what the interval would be if events C and D were two moderate earthquakes. Thus, combining C and D is conservative with respect to seismic hazard.

Charleston Seismic Source Zone Geometries. For its update of the Charleston seismic source zone, the applicant developed new source zone boundaries. Specifically, as described in SSAR Section 2.5.2.2.4, the applicant developed four, mutually exclusive source zone geometries, referred to as A, B, B', and C, to represent the Charleston seismic source. These four source zones are shown in SER Figure 2.5.2-2 (reproduced from SSAR Figure 2.5.2-9). SSAR Section 2.5.2.2.2.4.1 states that the width of Geometry B is 80 kilometers (50 miles). However, SSAR Figure 2.5.2-9 (and SER Figure 2.5.2-2) show that the width of Geometry B is 100 kilometers (62 miles). In RAI 2.5.2-14, the staff asked the applicant to provide the actual dimensions of Geometry B used for the UCSS. In response, the applicant stated that the width of UCSS Geometry B is 100 kilometers and not 80 kilometers, as stated in SSAR Section 2.5.2.2.2.4.1. Based on the applicant's clarification of the width of source zone B, the staff concludes that the source referred to as Geometry B in SSAR Figure 2.5.2-9 is accurate.

SSAR Section 2.5.2.4.4 states that "the new interpretation of the Charleston source indicates that a source of the large earthquakes in the Charleston area exists with weight 1.0…." Although the UCSS update of the Charleston source zone covers a fairly large area, the weighting and source geometries give the largest hazard only inside Zone A (either 0.9 (A, B, B') or 1.0 (A, B, B', C)), which is a relatively small zone. In view of this result, the staff asked the applicant, in RAI 2.5.2-13, to provide justification for the UCSS source geometries and weighting scheme and define what is meant by the "Charleston area." In its response, the applicant concluded that the Charleston source area is "stationary in space and is confined to a relatively restricted area," which it referred to as Geometry A. The applicant provided the following

information to support its conclusion that the source area that produced 1886 Charleston-type large magnitude earthquakes is likely relatively restricted in area:

> The updated Charleston seismic source model includes four potential geometries (A, B, B', and C) to represent the source area for the Charleston seismic source zone. The greatest weight is given to a localized zone (Geometry A) that completely incorporates the 1886 earthquake Modified Mercalli Intensity (MMI) X isoseismal (Bollinger 1977), the majority of identified Charleston meizoseismal-area tectonic features and inferred fault intersections, and the majority of reported 1886 liquefaction features. Outlying liquefaction features are excluded because liquefaction occurs as a result of strong ground shaking that may extend well beyond the areal extent of the tectonic source. Data describing the size and spatial distribution of paleoliquefaction features suggest prehistoric earthquakes (Events A, B, C', E, and F') were of similar magnitude and location to the 1886 Charleston earthquake, which produced liquefaction at significant distances northeast and southwest from the meizoseismal area. Lower weights are given for source geometries that envelop specific postulated tectonic features (i.e., Geometry C for the southern segment of the East Coast fault system), or for broader areal distributions that also envelop the localized zone to allow for greater uncertainty in the location and lateral extent of a fault that may have produced the 1886 Charleston earthquake.

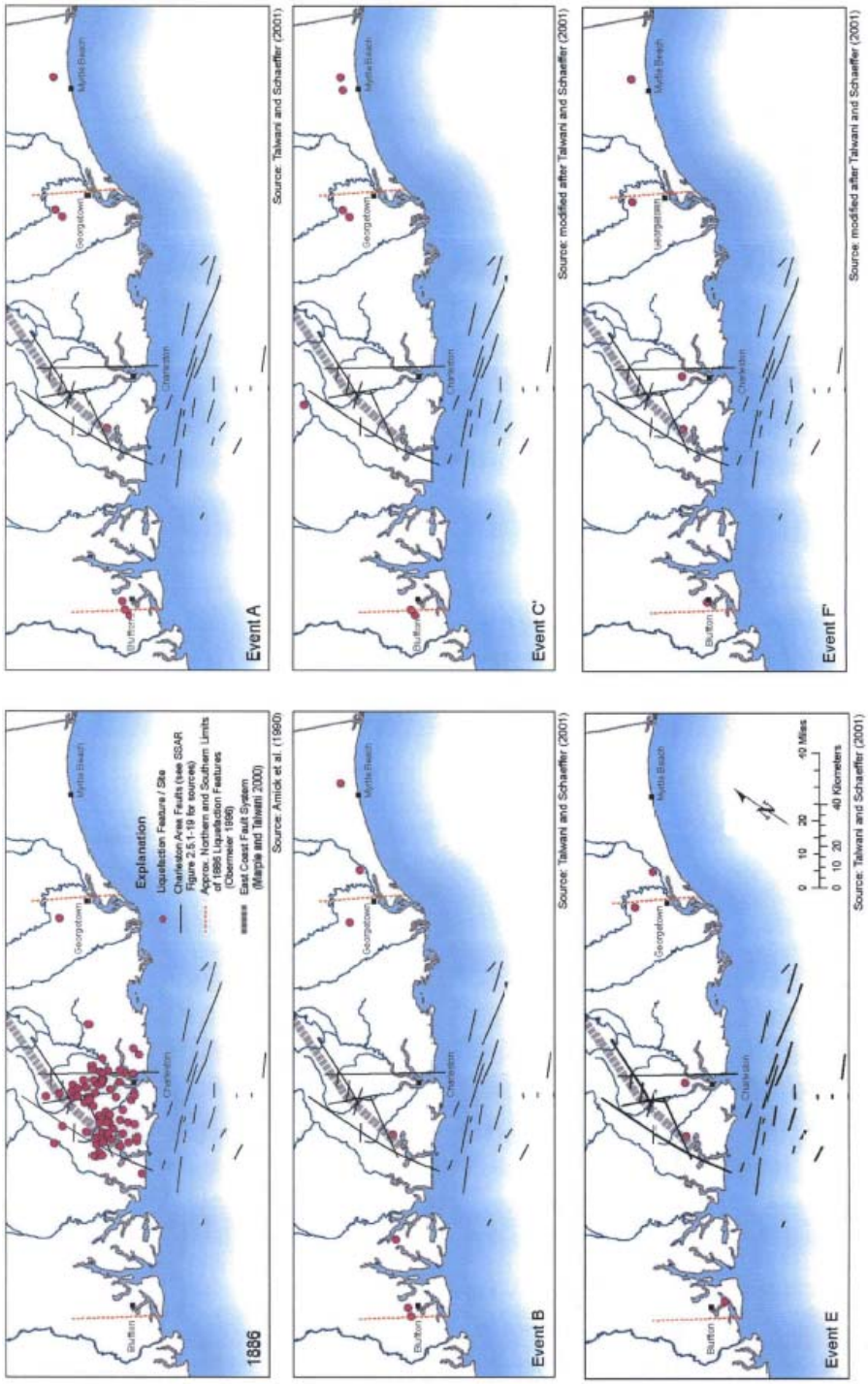


Figure 2.5.2-14 – Geographic Distribution of Liquefaction Features Associated with Charleston Earthquakes (SSAR Figure 2.5.2-12a)

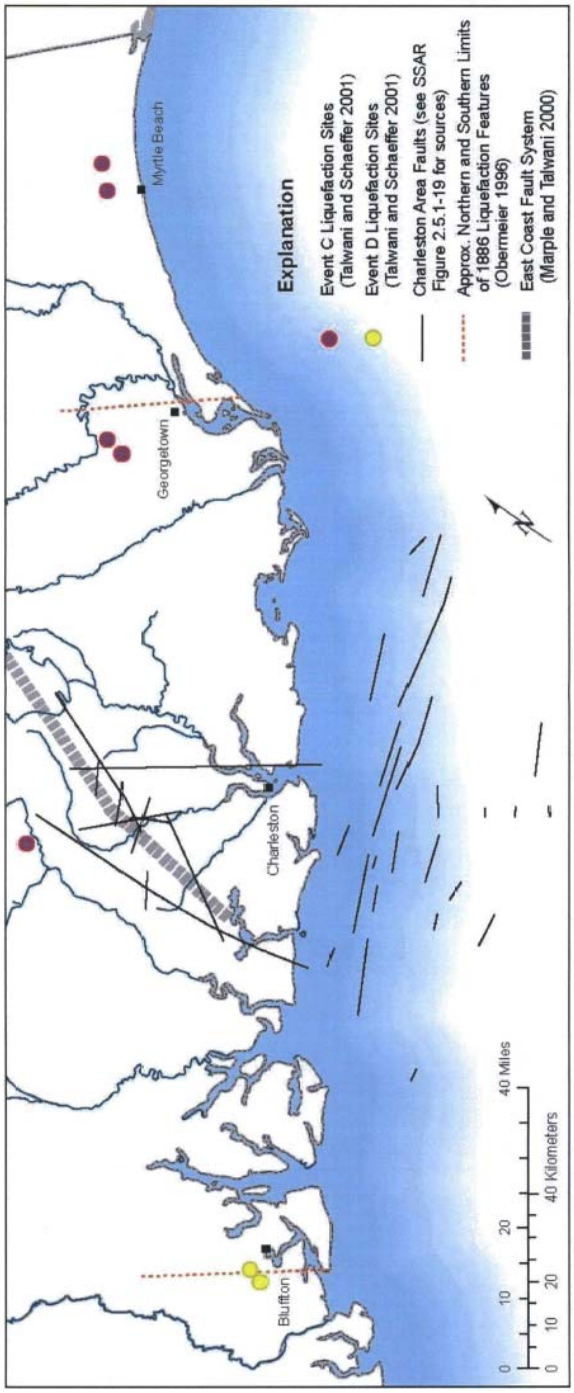


Figure 2.5.2-15 – Liquefaction Sites for Events C, C, and D (Applicant Response to RAI 2.5.2-11, Figure 2.5.2-11)

The applicant provided the following revision for the term “Charleston area” as used in the third sentence of the first paragraph of SSAR Section 2.5.2.4.4:

> The new interpretation of the Charleston source (see Section 2.5.2.2.2) indicates that a unique source of large earthquakes exists with weight 1.0 and that large magnitude events occur with a rate of occurrence unrelated to the rate of smaller magnitudes.

The applicant’s response states that the SSHAC Level 2 TI concluded that the Charleston source area is stationary in space and is confined to a relatively restricted area. Geometry A represents the preferred small source area and it is given a high weight of 0.7 (SSAR 2.5.2.2.2.4.1). According to the applicant geometry A is based on (1) the 1886 meizoseismal area and greatest density of liquefaction features; (2) the concentration of known and hypothesized tectonic features, mainly faults; (3) the concentration of historical seismicity, chiefly in the Middleton Place-Summerville seismic zone; and (4) the greatest density of prehistoric liquefaction features.

The staff focused its review on the density of prehistoric liquefaction features in relation to Geometry A because the use of a small source area to represent the sources of the 1886 and all previous large earthquakes depends crucially on a demonstration that the largest liquefaction craters of all ages concentrate near Charleston. The staff also reviewed the information presented in Bechtel (2006). Bechtel (2006) briefly references recent studies regarding the geographic distribution, density, and size of liquefaction features produced by the 1886 and prehistoric earthquakes in the Charleston region, specifically Obermeier et al. (1989, 1990, 2001) and Amick et al. (1990).

The staff also reviewed the study of Obermeier et al. (1989). Obermeier et al. (1989) conclude that, “Both the size and relative abundance of pre-1886 craters are greater in the vicinity of Charleston (particularly in the 1886 meizoseismal zone) than elsewhere, even though the susceptibility to earthquake-induced liquefaction is approximately the same at many places throughout this coastal region.” Figure 4 of Obermeier et al. (1989), reproduced as SER Figure 2.5.2-16, depicts the sizes of various prehistoric liquefaction features and demonstrates that the largest craters of all ages concentrate near Charleston. The staff notes that the figure cannot exclude the possibility that one (or more) of the large prehistoric earthquakes created its (or their) largest liquefaction features elsewhere. However, Obermeier’s (1989) figure shows four size classes of craters, with the largest prehistoric craters (wider than 3 meters) present only in the 1886 meizoseismal area. Only smaller craters are known farther south and north. Obermeier (1989) favors attributing some of these distant, small-to-medium-sized craters to infrequent moderate earthquakes at two separate sources far north and south of Charleston. The epicentral regions of 1886-sized earthquakes should have abundant craters wider than 3 meters, and they have been found only near Charleston. Sparse exposures preclude saying much about crater sizes between Beaufort and the Edisto River, south of Charleston (Obermeier et al. 1989) and south of Geometry A. Thus, it is unlikely, but possible, that the paleoliquefaction record of a large earthquake’s meizoseismal region could be concealed south of Geometry A. However, this small probability is accounted for by Geometries B and B’, which span most of the length of South Carolina’s coast. The absence of known abundant

paleoliquefaction features in North Carolina and Georgia, despite searches there (Amick and Gelinas 1991), suggests that Geometries B and B' need not extend beyond South Carolina.

Accordingly, the staff concludes that the applicant's use of a small area to represent the sources of the 1886 and all previous large earthquakes is adequate. Available evidence suggests it is likely that 1886-sized earthquakes occurred mostly or entirely within a small area like Geometry A. Evidence provided by the applicant in response to previous Open Item 2.5-5, further supports a localized source contained within Geometry A.

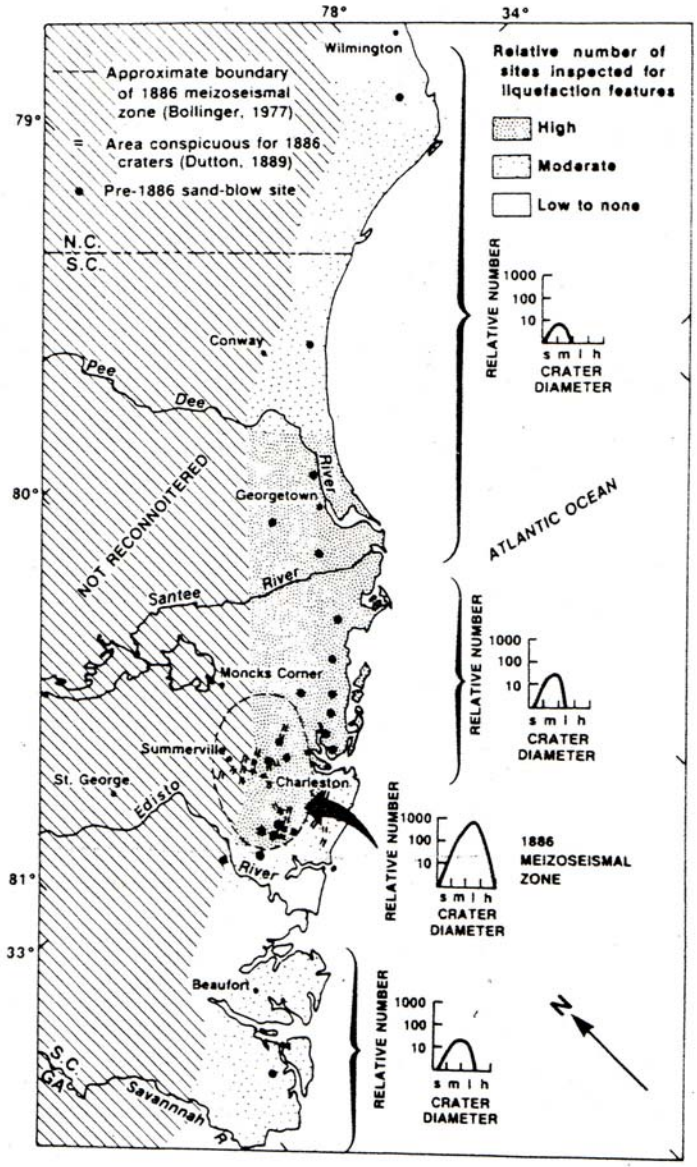


Figure 2.5.2-16 - Relative number of filled craters and crater diameters for pre-1886 sand blows at sites on marine-related sediments. The relative number is a scaling based on comparison with the abundance of craters in the 1886 meizoseismal zone, which has an arbitrary value of 1000. Crater diameters are small (s, less than 1 m), medium (m, 1–2 m), large (l, greater than 3 m) (reproduced from Obermeier et al. 1989).

Offshore of the South Carolina coast in the Charleston area there are several smaller faults (SER Figure 2.5.2-2). These faults correspond to the Helena Banks fault zone. In SSAR Section 2.5.2.2.2.4.1, the applicant concluded that, although the Helena Banks fault zone is clearly shown by multiple seismic reflection profiles and has demonstrable Late Miocene offset (Behrendt and Yuan 1987), there is no evidence to demonstrate the activity of this fault zone. In

RAI 2.5.2-15, the staff asked the applicant to explain why the two seismic events (mb 3.5 and 4.4) in 2002, which occurred in the vicinity of the Helena Bank fault zone, cannot be positively correlated with the fault zone. The association of these two events with the Helena Banks fault zone would indicate that this fault zone is currently active. In response, the applicant stated that it could not positively correlate the two earthquakes with the Helena Banks fault zone for the following reasons:

The lack of detailed information on these two 2002 offshore earthquakes (poor location, no focal mechanisms) and the lack of additional seismic activity in this offshore area, make it difficult to assign the Helena Banks fault zone as the causative fault. It is possible that the two 2002 earthquakes indicate reactivation of the Helena Banks fault zone, but the fact that these events cannot be positively correlated to the fault suggests otherwise. There are numerous faults in the central and eastern United States located close to a few or more poorly located, small earthquakes, but this simple and very limited spatial association has not typically led researchers to positively correlate them to specific faults and classify these faults as reactivated seismogenic structures.

Based on its review of the applicant's response to RAI 2.5.2-15, the staff concurs with the applicant's conclusion that it could not positively correlate the recent offshore earthquakes with the Helena Banks fault zone because of the uncertainties regarding the exact locations of these two events. However, even though these two events cannot be directly correlated with the Helena Banks fault zone, the applicant's UCSS source zone Geometry B encompasses both the Helena Banks fault zone and the epicenters of these two events.

Recurrence intervals for the Charleston seismic source. In SSAR Section 2.5.2.2.2.4.3, the applicant describes its calculation of recurrence intervals for the updated Charleston seismic source, which is largely based on paleoliquefaction data compiled by Talwani and Schaeffer (2001). The applicant calculated two different average recurrence intervals, which represent two recurrence branches on the logic tree. The first average recurrence interval is based on the four events (1886, A, B, and C') that the applicant interpreted to have occurred within the past ~2000 years. The applicant considered this time period to represent a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of its expert elicitation. This branch of the logic tree was given a weight of 0.8. The applicant's second average recurrence interval is based on events that the applicant interpreted to have occurred within the past ~5000 years and includes events 1886, A, B, C', E, and F'. This time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). Published papers and researchers questioned by the applicant suggest that the older part of the record (i.e., older than ~2000 years) may be incomplete. The applicant noted, however, that it may also be possible that the older record is complete and exhibits longer inter-event times. For this reason, the average recurrence interval calculated for the ~5000-yr record (six events) is given a weight of 0.20 on the logic tree.

In RAI 2.5.2-12, the staff asked the applicant to provide more detail regarding its rationale for the weighting of the two recurrence branches on the logic tree. The staff also asked the applicant to justify its use of these two scenarios rather than another case study (e.g., 10 large-magnitude earthquakes occurring at approximately regular intervals during the past

5000 years), including its impact on the hazard calculation. The applicant provided the following response to justify its weighting of the 2000-yr and 5000-yr logic tree branches:

> The relative weighting of these two branches of the logic tree is based on a SSHAC level 2 assessment of completeness of the geologic record of paleoliquefaction events over these two time intervals. Earthquakes in the paleoliquefaction record do not occur at regular intervals, and this may be the result of "temporal clustering of seismicity, fluctuation of water levels, or their evidence having been obliterated" (Talwani and Schaeffer 2001; p. 6640). Talwani and Schaeffer (2001) consider the paleoliquefaction record to be complete for the past 2,000 yrs. Moreover, Prof. Pradeep Talwani (University of South Carolina, pers. comm. 9/8/05) and Dr. Steve Obermeier (U.S. Geological Survey [retired], pers. comm. 9/2/05) consider the 2,000-yr record to represent a complete portion of the paleoseismic record. For these reasons, the average recurrence interval calculated for the most-recent ~2,000 yr portion of the paleoseismologic record is given a relatively high weight of 0.80.
>
> The degree of completeness for the entire ~5,000-yr record of paleoliquefaction events is uncertain. It is possible that all paleoliquefaction events in this time period have been preserved and recognized in the geologic record. Alternatively, it is possible that events are missing from the ~5,000-yr record. Average Mmax recurrence interval calculated from the entire ~5,000-yr record is greater (i.e., larger average interevent time) than that calculated for the ~2,000-yr record. The decision to give less weight (0.20) to this recurrence estimate is therefore conservative.

Regarding its use of these two scenarios rather than another case study (e.g., 10 large-magnitude earthquakes occurring at approximately regular intervals during the past 5000 years), the applicant stated the following:

> We also considered other scenarios from which to calculate earthquake recurrence, but ultimately decided not to incorporate those that included non-conservative assumptions. For example, Talwani and Schaeffer (2001) include a scenario in which their events C and D are moderate-magnitude, local earthquakes. These moderate-magnitude earthquakes would be eliminated from the record of large (Mmax) earthquakes, thereby increasing the calculated recurrence interval. This and other permutations of the paleoliquefaction record (and resulting recurrence intervals) could be included, but, if based on nonconservative assumptions, would increase the recurrence interval and lower the hazard without sufficient justification. The given example of "ten large-magnitude earthquakes occurring at approximately regular intervals during the past 5,000 years" was not included in the model because: (1) it is permissible only if events are assumed to be missing from the geologic record; and (2) the resulting recurrence interval would be very similar to the branch of the logic tree using the ~2,000-yr paleoliquefaction record.

In summary, the applicant assigned the largest weight of 0.8 to the average recurrence interval calculated for the most recent ~2000-yr portion of the paleoseismologic record. The applicant considered this time period to represent a complete portion of the paleoseismic record based on published literature (e.g., Talwani and Schaeffer 2001) and feedback from those researchers questioned (Talwani 2005; Obermeier 2005) by the applicant as part of the expert elicitation. The applicant stated that the 5000-yr time period represents the entire paleoseismic record based on available liquefaction data (Talwani and Schaeffer 2001). However, the applicant only assigned a weight of 0.2 to the 5000-yr branch of the logic tree because the completeness of the ~5000-yr paleoseismic record is uncertain.

Based on its review of the applicant's response to RAI 2.5.2-12, and the information presented by the applicant in SSAR Section 2.5.2.2, the staff concurs with the applicant's logic tree weighting for earthquake recurrence because it reflects all of the available data and uncertainties. Specifically, the applicant assigned the largest weight of 0.8 to the 2000-yr logic tree branch because there is a greater certainty that this portion of the paleoseismologic record is complete. The applicant also used the entire ~5000-yr record to calculate earthquake recurrence. The applicant calculated a recurrence interval of 958 years from the ~5000-yr record. This value is less conservative than the mean recurrence interval of 548 years calculated from the ~2000-yr record. However, the applicant assigned a significantly lower weight of 0.2 to this logic tree branch because there is a greater uncertainty that the ~5000-yr record is complete.

In summary, the staff focused its review of SSAR Section 2.5.2.2 on the applicant's update of the Charleston seismic source model and its basis for not updating the other EPRI seismic source zones that contribute to the seismic hazard at the ESP site. The staff concludes that the applicant's update of the 1986 EPRI PSHA sources adequately characterizes the seismic hazard in the region surrounding the site.

2.5.2.3.3 Correlation of Earthquake Activity with Seismic Sources

SSAR Section 2.5.2.3 describes the correlation of updated seismicity with the EPRI seismic source model. The applicant compared the distribution of earthquake epicenters from both the original EPRI historical catalog (1627–1984) and the updated seismicity catalog (1985–2005) with the seismic sources characterized by each of the EPRI ESTs. The applicant concluded that there are no new earthquakes within the site region that can be associated with a known geologic structure and that there are no clusters of seismicity suggesting a new seismic source not captured by the EPRI seismic source model. The applicant also concluded that the updated catalog does not show a pattern of seismicity that would require significant revision to the geometry of any of the EPRI seismic sources. The applicant further concluded that the updated catalog does not show or suggest an increase in Mmax or a significant change in seismicity parameters (activity rate, b-value) for any of the EPRI seismic sources. The applicant based its conclusions on a comparison of the distribution of earthquake epicenters from both the original EPRI historical catalog and from its updated seismicity catalog with the seismic sources characterized by each of the EPRI ESTs.

In Parts A and B of RAI 2.5.2-1, the staff requested electronic versions of the EPRI seismicity catalog and the applicant's updated EPRI seismicity catalog for the region of interest. In Part C

of RAI 2.5.2-1, the staff requested the geographic coordinates of the primary source zones developed by each of the six EPRI ESTs. The staff used the information provided in response to Parts A and B of RAI 2.5.2-1 to compare the applicant's update of the regional seismicity catalog with its own listing of recent earthquakes. Based on this comparison, the staff concurs with the applicant's assertion that the rate of seismic activity has not increased in the ESP region since 1985. Using the information provided in response to Part C of RAI 2.5.2-1, the staff compared the updated earthquake catalog with each of the primary seismic sources developed by each EPRI EST. Based on the comparision of earthquakes in the updated catalog with each of the EPRI EST seismic sources, the staff concurs with the applicant's conclusion that revisions to the existing EPRI sources are not warranted. However, additional worldwide earthquake data may indicate the need for an update of some of the EPRI seismic source models. In addition, recent paleoliquefaction studies predict shorter recurrence intervals for large Charleston-type earthquakes compared to predictions based on the historical seismicity catalog. These paleoliquefaction data also provide information regarding the locations of large prehistoric Charleston-type earthquakes. SER Section 2.5.2.3.2 describes the staff's conclusions with respect to the applicant's update of the Charleston seismic source.

2.5.2.3.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

SSAR Section 2.5.2.4 presents the earthquake potential for the ESP site in terms of the controlling earthquakes. The applicant determined the high- and low-frequency controlling earthquakes by deaggregating the PSHA results at selected probability levels. Before determining the controlling earthquakes, the applicant updated the 1989 EPRI PSHA using the seismic source zone adjustments described in SER Section 2.5.2.1.2 and the new ground motion models described in SER Section 2.5.2.1.4.

The staff focused its review of SSAR Section 2.5.2.4 on the applicant's updated PSHA and the ESP site controlling earthquakes determined by the applicant after completion of its PSHA. While the staff's review of the applicant's update of the EPRI seismic source model is described in SER Section 2.5.2.3.2, this SER section focuses on the review of the application of the updated seismic source model to the hazard calculation at the ESP site.

PSHA Inputs

As input to its PSHA, the applicant used its updated version of the 1989 EPRI seismic source model. The staff's evaluation of the applicant's update is described in SER Section 2.5.2.3.2. The applicant also used the ground motion models developed by the 2004 EPRI-sponsored study (EPRI 1009684 2004) as input to its PSHA. The ESP applications for the Clinton (Illinois), Grand Gulf (Mississippi) and North Anna (Virginia) sites also used the updated EPRI ground motion models. The staff's final SERs for Clinton (ADAMS Accession No. ML0612204890), Grand Gulf (ADAMS Accession No. ML061070443), and North Anna (ADAMS Accession No. ML063170371) provide an extensive review of the EPRI 2004 ground motion models. Thus, the staff considers the applicant's use of the EPRI 2004 ground motion model to be appropriate.

PSHA Results

In order to determine the adequacy of the PSHA results, the staff, in RAI 2.5.2-1, requested that the applicant to provide the 1- and 10-Hz mean hazard curves for each of the six EPRI ESTs, as well as the 1- and 10-Hz mean hazard curves for the UCSS model. In response to RAI 2.5.2-1, the applicant provided the requested hazard curves. SER Figures 2.5.2-17 and 2.5.2-18 show the applicant's 1-Hz and 10-Hz total mean hazard curves, as well as the hazard curves corresponding to each of the six EPRI EST seismic source model inputs. Both figures also show the hazard curves corresponding to the applicant's UCSS model.

The total mean hazard curves, shown in SER Figures 2.5.2-17 and 2.5.2-18, comprise the mean of the six EPRI EST total hazard curves plus the contribution of the UCSS.

As shown in SER Figure 2.5.2-17, for the 1-Hz hazard curves, the Charleston source dominates the overall hazard at the ESP site. In SER Figure 2.5.2-18, for the 10-Hz hazard curves, the contributions from each of the six ERPI seismic source models have a more significant contribution to the overall hazard.

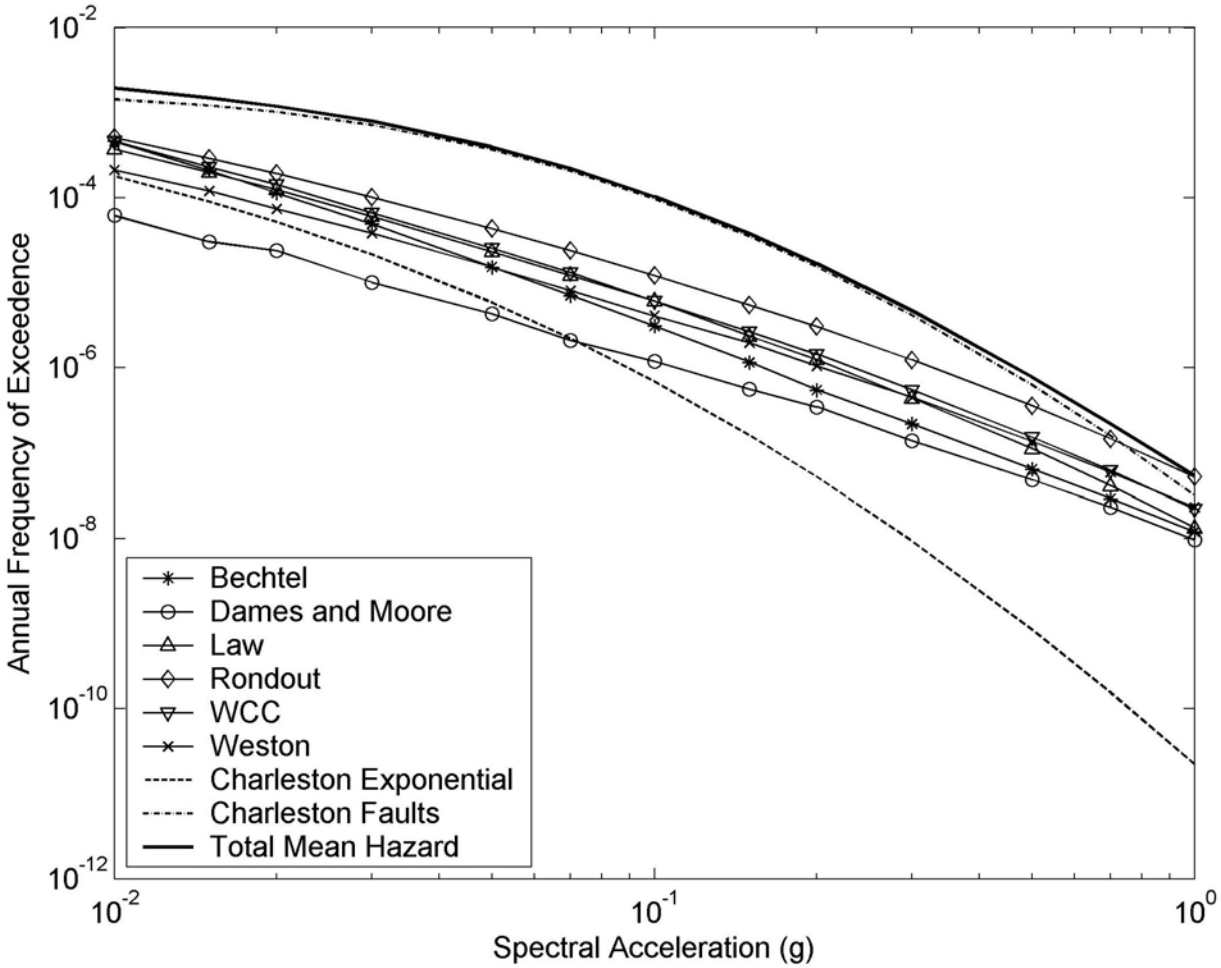


Figure 2.5.2-17 - Plot showing the applicant's 1-Hz total mean hazard curve for the ESP site. This figure also shows the contributions of the applicant's UCSS model, which consists of "Charleston Faults" and "Charleston Exponential," as well as the contributions from each of the six EPRI EST seismic source models.

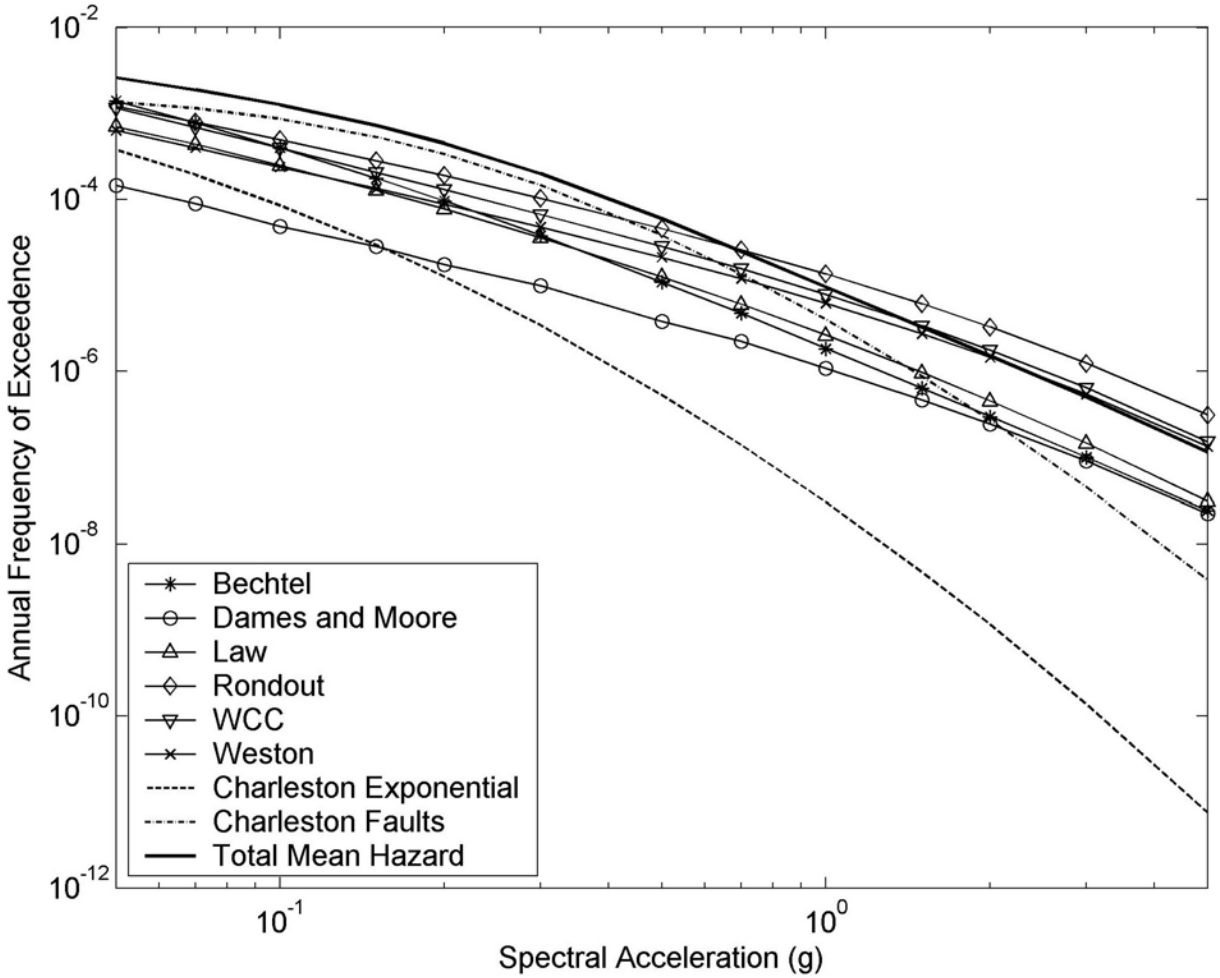


Figure 2.5.2-18 - Plot showing the applicant's 10-Hz total mean hazard curve for the ESP site. This figure also shows the contributions of the applicant's UCSS model, which consists of "Charleston Faults" and "Charleston Exponential," as well as the contributions from each of the six EPRI EST seismic source models.

Controlling Earthquakes. To determine the low- and high-frequency controlling earthquakes for the ESP site, the applicant followed the procedure outlined in Appendix C to RG 1.165. This procedure involves the deaggregation of the PSHA results at a target probability level to determine the controlling earthquakes in terms of magnitude and source-to-site distance. The applicant chose to perform the deaggregation of the mean 10^{-4}, 10^{-5}, and 10^{-6} PSHA results. SER Table 2.5.2-8 shows the low- and high-frequency controlling earthquakes. Because of the similarity of Mbar and Dbar values for the three hazard levels, the applicant selected a single recommended Mbar and Dbar value for each frequency range. For the high-frequency mean 10^{-4} and 10^{-5} and 10^{-6} hazard levels, the controlling earthquake has a magnitude of M 5.6 event occurring at a distance of 9.0 kilometers (5.6 miles), corresponding to an earthquake from a local seismic source zone. In contrast, for the low-frequency mean 10^{-4} and 10^{-5} and 10^{-6}

hazard levels, the controlling earthquake has a magnitude of M 7.2 at a distance of 130 kilometers (80.8 miles). This controlling earthquake corresponds to an event in the Charleston seismic source zone.

Table 2.5.2-8 - Computed and Final Mbar and Dbar Values Used for Development of High-and Low-Frequency Target Spectra (Based on Information Provided In SSAR Table 2.5.2-17)

High Frequency (5 to 10 Hz)				
Mean Hazard Level	**10^{-4}**	**10^{-5}**	**10^{-6}**	**Final Values**
Mbar (M)	5.6	5.6	5.7	5.6
Dbar	17.6 km (10.9 mi)	11.4 km (7.1 mi)	9.0 km (5.6 mi)	9.0 km (5.6 mi)
Low Frequency (1 to 2.5 Hz)				
Mean Hazard Level	**10^{-4}**	**10^{-5}**	**10^{-6}**	**Final Values**
Mbar (M)	7.2	7.2	7.2	7.2
Dbar	136.5 km (84.8 mi)	134.3 km (83.5 mi)	133.0 km (82.6)	130 km (80.8 mi)

In RAI 2.5.2-21, the staff asked the applicant to explain how it calculated the final Dbar and Mbar values. In its response to RAI 2.5.2-21, the applicant stated that the final low-frequency distance value of 130 kilometers (80.8 miles) is based on the source-to-site distance for the Charleston source, while the final high-frequency value of 9 kilometers (5.6 miles) is equal to the log-average of the three computed values rounded to the nearest kilometer. The applicant also stated that the final magnitude values for the respective high- and low-frequency cases are equal to the linear average of the three magnitude values rounded to the nearest tenth of a magnitude unit. In addition, the applicant provided a comparison between the high-frequency spectral shape using the final magnitude and distance values and the computed magnitude and distance values. The applicant also provided a comparison between the low-frequency spectral shape using the final magnitude and distance values and the computed magnitude and distance values. Based on its comparison, the applicant concluded that the use of the recommended magnitude and distance values in place of the computed magnitude and distance values for each of the three annual probability levels would not significantly change the results of the site response analysis.

The staff concurs with the applicant's final high- and low-frequency Mbar and Dbar values because these final values, and the corresponding spectral shapes, are very similar to the calculated values for the three annual probability levels.

Based on its review of the ESP site controlling earthquake magnitudes and distances as discussed above, the staff concludes that the applicant's PSHA adequately characterized the overall seismic hazard of the ESP site. The staff also concludes that the applicant's controlling earthquakes for the ESP site (**M** 5.6 at 9 km (5.6 miles), **M** 7.2 at 130 km (80.8 miles)) are generally consistent with both the historical earthquake record and paleoliquefaction studies in the Charleston seismic source zone. In addition, the staff finds that the ground motions developed by the applicant from the controlling earthquakes are consistent with the most recent CEUS ground motion evaluations. Accordingly, the staff concludes that the applicant followed

the guidance in RG 1.165 and RG 1.208 for evaluating regional earthquake potential and determining the ground motion resulting from controlling earthquakes.

2.5.2.4.5 Seismic Wave Transmission Characteristics of the Site

SSAR Section 2.5.2.5 describes the method used by the applicant to develop the ESP site free-field ground motion spectrum. The seismic hazard curves generated by the applicant's PSHA are defined for generic hard rock conditions (characterized by a S-wave velocity of 9200 ft/s). According to the applicant, these hard rock conditions exist at a depth of more than 2000 feet below the ground surface at the ESP site. To determine the site free-field ground motion, the applicant performed a site response analysis. The output of the applicant's site response analysis is site AFs, which are then used to determine the UHRS for three hazard levels (10^{-4}, 10^{-5}, and 10^{-6}). The 10^{-4} and 10^{-5} UHRS are then used to calculate the GMRS for the site.

In SSAR Section 2.5.2.5.1.1, the applicant describes the methodology it used to develop the soil UHRS for the 10^{-4}, 10^{-5}, and 10^{-6} hazard levels. The applicant's site free-field soil UHRS is defined at the top of the Blue Bluff Marl. According to the applicant, the top of the Blue Bluff Marl is characterized by an average S-wave velocity of 2354 ft/s. In RAI 2.5.2-19, the staff asked the applicant to provide a detailed step-by-step description of the methodology it used to develop the site AFs and the 10^{-4} and 10^{-5} soil UHRS. In response to RAI 2.5.2-19, the applicant more completely explained Steps 1 through 6. However, after reviewing the applicant's response, the staff concluded that the applicant's description of Steps 5 and 6 did not provide sufficient detail for the staff to completely evaluate the site response method. In particular, the staff was not clear on the enveloping motion used in Step 5, and the applicant's description in Step 6 appeared to differ from that described in SSAR Section 2.5.2.5.1.1. On June 18, 2007, the applicant supplemented its RAI response with additional detail on each of the steps used in the site response analysis; however, the staff had not been able to completely evaluate the applicant's supplemental information. As such, the staff was not able to reach a conclusion in the SER with open items on the adequacy of the applicant's methodology. Accordingly, in the SER with open items, the staff identified Open Item 2.5-6 to reflect the additional review time needed by the staff to review the applicant's supplemental response to RAI 2.5.2-19, as well as the staff's request for further clarification of Step 6 of the applicant's site response methodology.

Based on the applicant's response to RAI 2.5.2-19 and Open Item 2.5-6, a summary of the applicant's site response methodology is provided below:

The applicant determined the final 10^{-4} soil surface spectrum for the ESP site by scaling the hard rock UHRS (shown in SER Figure 2.5.2-5) by the final AFs (shown in SER Figure 2.5.2-6). The applicant defined each of the AFs at a total of 300 frequencies, but only defined the hard rock UHRS at 7 structural frequencies. For this reason, the applicant interpolated the hard rock UHRS at values between the 7 structural frequencies using the high- and low-frequency spectral shapes (from NUREG/CR-6728) for hard rock. This resulted in two rock spectra: a high-frequency spectrum and a low-frequency spectrum that are both constrained to equal the spectral amplitudes for the 7 PSHA structural frequencies at which the PSHA was calculated. From the high-frequency and low-frequency rock spectra, a single spectrum was then derived

using the high-frequency rock spectrum for high frequencies and the low-frequency rock spectrum for low frequencies.

In order to determine the 10^{-4} soil spectrum (UHRS), the applicant multiplied the hard rock UHRS (now defined at 300 structural frequencies) by either the high- or low-frequency final amplification factors, which are shown in SER Figure 2.5.2-6. The applicant multiplied the hard rock UHRS by the high-frequency final amplification factors for frequencies above 8 Hz. For frequencies below 5 Hz, the applicant multiplied the hard rock UHRS by the low-frequency final amplification factors. In between 8 Hz and 5 Hz, the applicant interpolated the soil spectrum to achieve a smooth transition between the high-frequency and low-frequency controlled parts.

The applicant repeated the above process for the 10^{-5} hazard level to determine the final 10^{5} soil UHRS. SER Figure 2.5.2-7 provides the final soil UHRS for the 10^{-4} and 10^{-5} hazard levels.

Upon completing its review of the supplemental response to RAI 2.5.2-19 as well as the applicant's additional response to Open Item 2.5-6, summarized above, the staff concludes that the applicant provided sufficient information for the staff to perform its review of the methodology. The staff also concludes that the supplemental information is generally consistent with what the applicant provided in SSAR Section 2.5.2.5. Furthermore, the staff concludes that the applicant's site response methodology is adequate because it follows the guidance provided in RG 1.208.

SSAR Section 2.5.2.5.1.3 describes the development of low- and high-frequency target spectra based on the low- and high-frequency controlling earthquake magnitudes and distances. To determine the target low- and high-frequency spectra, the applicant used the average of the single and double corner source models provided in NUREG/CR-6728. In RAI 2.5.2-20, the staff asked the applicant why it did not use the EPRI ground motion models (EPRI 1009684 2004) to develop the high- and low-frequency target response spectra since the applicant used these ground motion models for its PSHA. In response to RAI 2.5.2-20, the applicant provided the following information:

> The 2004 EPRI ground motion report (EPRI 1009684) gives equations to estimate spectral acceleration at 7 structural frequencies (100, 25, 10, 5, 2.5, 1, and 0.5 Hz). To properly represent rock motion for input to a site response analysis, it is necessary to interpolate between these 7 structural frequencies to obtain a realistic spectral shape, rather than using linear interpolation. For this task, NUREG/CR-6728 was used, because one of its goals was specifically to develop realistic spectral shapes for the eastern U.S. to use in earthquake ground motion analyses.

The staff concurs with the applicant's use of NUREG/CR-6728 spectral models for the CEUS, since the EPRI 2004 ground motion models only provide 7 structural frequencies. Because the applicant used the NUREG/CR-6728 source models, it was able to avoid using linear interpolation and, subsequently, obtained a more accurate estimate of the site response.

A key step in the site response analysis is the selection of actual earthquake records that closely match the low- and high-frequency controlling earthquake magnitude and distance values. The response spectra from these earthquake records, which are generally from the WUS, are matched to the CEUS spectral shapes described in the preceding paragraph. SSAR Section 2.5.2.5.1.4 describes the spectral matching of the selected seed time histories to the target response spectra and states that "the spectral matching criteria given in NUREG/CR-6728 were used to check the average spectrum from the 30 time histories for a given frequency range (high- or low-frequency) and annual probability level. This is the recommended procedure in NUREG/CR-6728 when multiple time histories are being generated and used." In RAI 2.5.2-22, the staff asked the applicant to verify that it satisfied the NUREG/CR-6728 matching criteria for each individual earthquake time history. In response to RAI 2.5.2-22, the applicant pointed out that item (e) of the NUREG/CR-6728 matching criteria provides guidance for the use of a suite of ground motion records as well as for an individual record. In addition, the applicant stated that it matched the other relevant criteria for both the low-frequency and high-frequency spectra. Since the applicant followed the guidance specified in NUREG/CR-6728 for multiple time histories and also matched the other relevant criteria, the staff concludes that the applicant adequately matched the seed time histories to the CEUS spectral shapes.

In addition to the seed time histories, another important part of the site response analysis is the model of the site subsurface soil and rock properties. In particular, the applicant's site response analysis should incorporate the uncertainty in these properties. Key properties include the shear wave velocities, material damping, and the strain-dependent behavior of each of the soil layers underlying the site. To model the strain-dependent behavior of the soil, the applicant used shear modulus and damping curves developed by EPRI (EPRI TR-102293 1993), as well as curves developed for the SRS (Lee 1996). Besides these soil properties, in RAI 2.5.2-23, the staff asked the applicant to discuss results of its site response calculations in terms of the following:

1. the effects of the six alternative site response profiles in terms of the different depths to the top of the Paleozoic crystalline rocks
2. the possible effects of the Pen Branch fault zone (i.e., as a low-velocity zone or weak zone)
3. the effects of the low-velocity zones within the Blue Bluff Marl and Lower Sand Stratum

In response to RAI 2.5.2-23, the applicant performed additional sensitivity calculations to examine the effects of the different depths to the top of the Paleozoic crystalline rocks using the six base case profiles shown in SSAR Table 2.5.4-11, Part B. In order to represent the Pen Branch fault as a low-velocity zone, the applicant modified the rock S-wave velocities of the six base profiles to include a low-velocity zone and to represent the Pen Branch fault. The applicant concluded that the depth to the Pen Branch fault, and a lower velocity layer for the Pen Branch, does not affect the site response. The applicant observed very small differences between the results. Regarding the effects of the low-velocity zones within the Blue Bluff Marl and Lower Sand Stratum, the applicant stated the following:

> The low velocity zones in the Blue Bluff Marl and in the Lower Sand Stratum were incorporated in the site response calculations, i.e., the site response

calculation results inherently reflect the inclusion of these low velocity zones. The calculations were performed using the base case shear wave velocity profile that is based on field measurements, and randomized profiles.

The staff reviewed the applicant's response to RAI 2.5.2-23, as well as the results of its sensitivity calculations, and concludes that the applicant adequately captured the site variability in its site response calculations. The applicant generated randomized soil and rock S-wave velocity profiles and randomly paired them with 60 sets of shear modulus degradation and damping curves. According to RG 1.208, the use of 60 randomized profiles is generally adequate to determine a reliable estimate of the mean and standard deviation of the site response.

To determine the adequacy of the applicant's site response calculations, the staff performed its own confirmatory site response calculations. The staff used a site response methodology similar to that used by the applicant and, like the applicant, the staff used the program SHAKE. The main difference between the two sets of calculations is that the staff did not use as many input time histories as the applicant used for its analysis. In addition, the staff did not use randomized soil and rock S-wave velocity profiles, soil shear modulus reduction and damping relationships, and rock damping values. Instead, as inputs to its confirmatory analysis, the staff used the applicant's base case S-wave velocity profiles (given in SSAR Table 2.5.4-11) and shear modulus reduction and damping relationships (given in SSAR Tables 2.5.4-12 and 2.5.4-13).

SER Figures 2.5.2-19 to 2.5.2-22 show the mean AFs resulting from the staff's confirmatory site response calculations. Each figure plots the mean results of the six alternative subsurface profiles for both the EPRI and SRS shear modulus and damping curves. SER Figures 2.5.2-19 and 2.5.2-20 show the results corresponding to the 10^{-4} hazard levels for the respective high- and low-frequency input motions, while SER Figures 2.5.2-21 and 2.5.2-22 plot the results corresponding to the 10^{-5} hazard levels for the respective high- and low-frequency input motions. SER Figures 2.5.2-18 to 2.5.2-22 also show the applicant's mean AFs for comparison. The applicant's results are similar overall. For each case, the amplification peaks are very similar, and in all cases, the peaks occur at approximately 0.6 Hz. The differences between the results are likely due to the greater variability that the applicant incorporated into its model through the use of randomized profiles and material properties, as well as the use of multiple time histories. This variability is illustrated in SER Figure 2.5.2-23 (reproduced from SSAR Figure 2.5.2-37). As a result of its analysis, the staff was able to confirm the applicant's overall site response results.

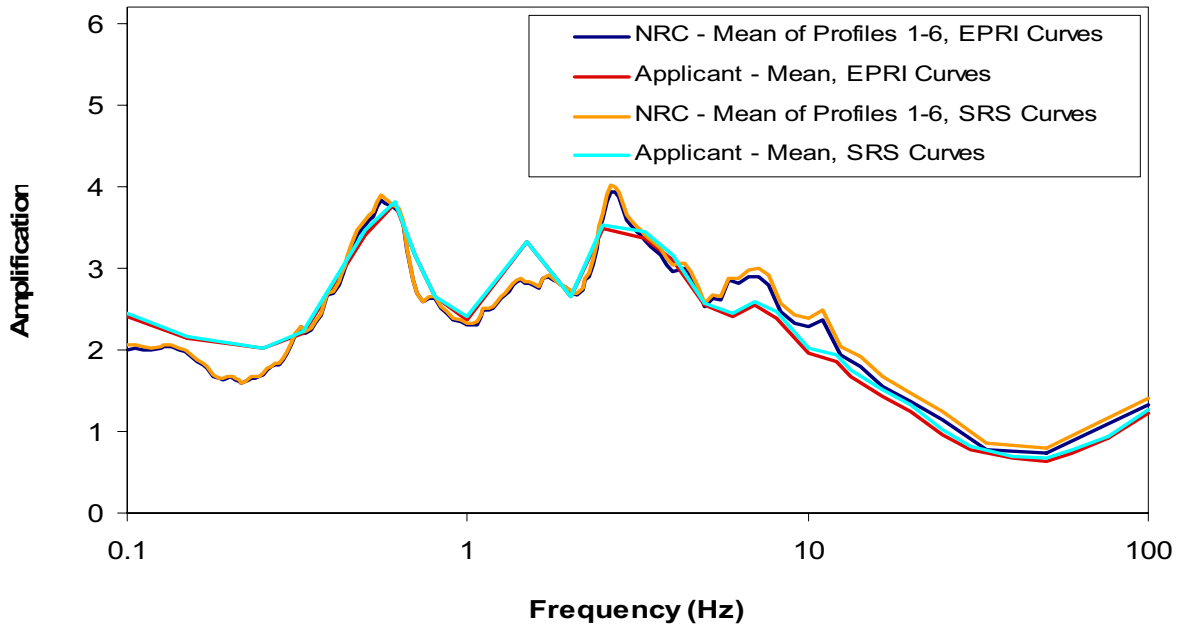


Figure 2.5.2-19 - Results of the staff's site response calculations for high-frequency rock motions for the 10^{-4} hazard level. The applicant's mean results are shown for comparison.

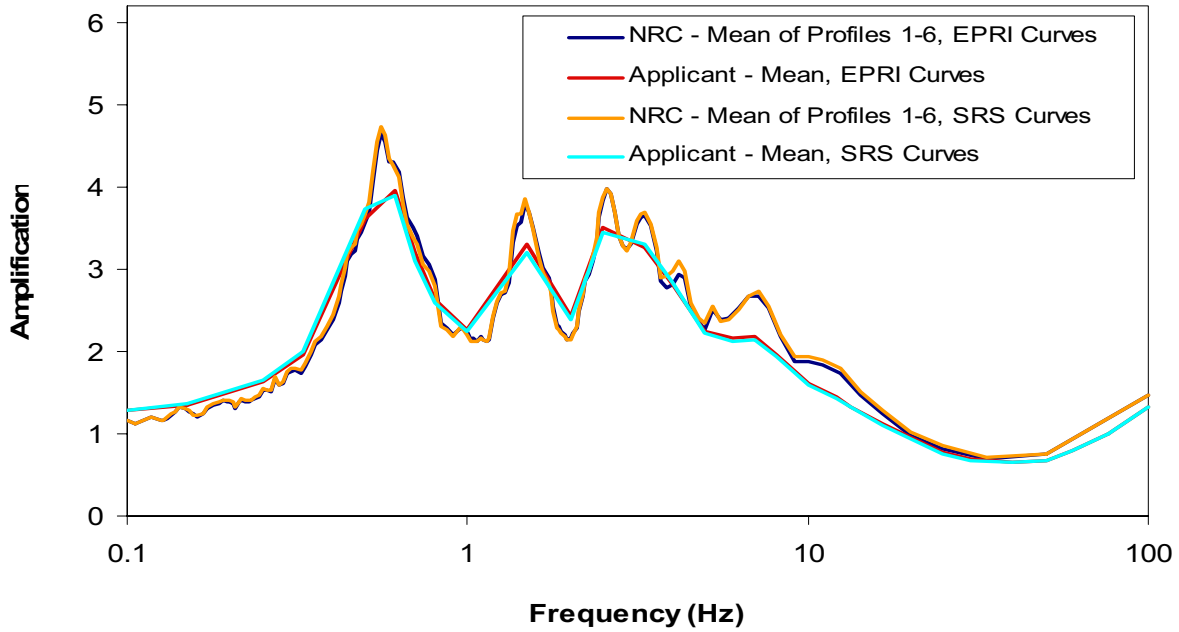


Figure 2.5.2-20 - Results of the staff's site response calculations for low-frequency rock motions for the 10^{-4} hazard level. The applicant's mean results are shown for comparison.

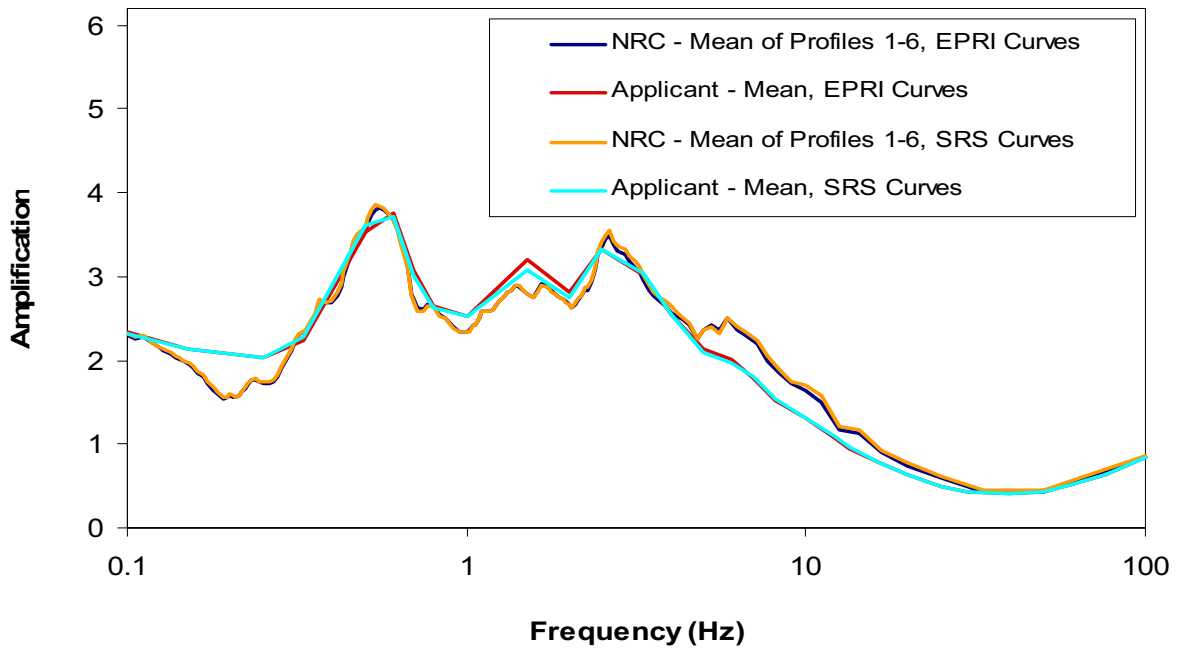


Figure 2.5.2-21 - Results of the staff's site response calculations for high-frequency rock motions for the 10^{-5} hazard level. The applicant's mean results are shown for comparison.

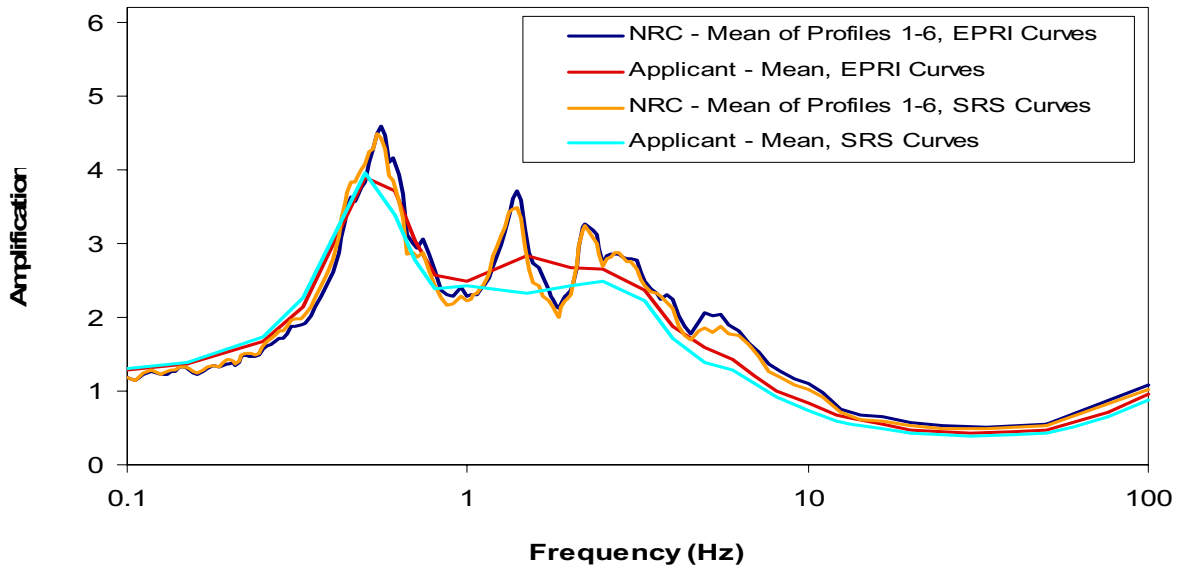


Figure 2.5.2-22 - Results of the staff's site response calculations for low-frequency rock motions for the 10^{-5} hazard level. The applicant's mean results are shown for comparison.

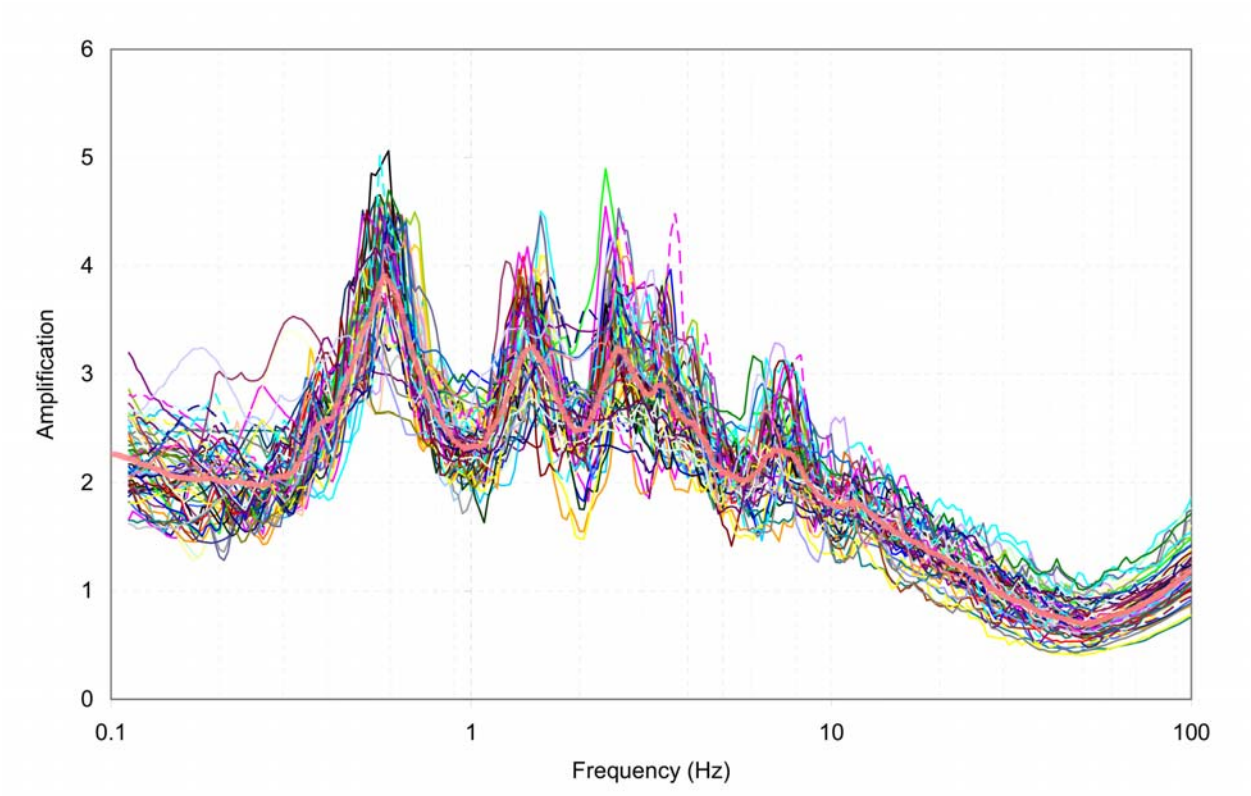


Figure 2.5.2-23 - Results of the applicant's site response calculations for high-frequency rock motions for the 10^{-4} hazard level using the EPRI degradation curves (reproduced from SSAR Figure 2.5.2-37).

In RAI 2.5.2-23, the staff asked the applicant to justify its use of an equivalent-linear approach rather than a nonlinear approach to model the soil nonlinearity at the ESP site. In response, the applicant provided a table containing the maximum shear strains obtained from its SHAKE analyses of the randomized profiles. The applicant's table is reproduced as SER Table 2.5.2-9. In reference to SER Table 2.5.2-9, the applicant stated, "The table shows that the maximum soil strain remained below 0.6 percent. The equivalent-linear approach is adequate for this low level of soil strain."

Table 2.5.2-9 - Applicant's Maximum Shear Strain Values Provided In Response to RAI 2.5.2-23

Earthquake Probability Level	EPRI Randomized Profiles		SRS Randomized Profiles	
	Low-Frequency Earthquake	High-Frequency Earthquake	Low-Frequency Earthquake	High-Frequency Earthquake
10^{-4}	0.078 percent	0.067 percent	0.082 percent	0.068 percent
10^{-5}	0.592 percent	0.300 percent	0.287 percent	0.353 percent

The staff believed that further justification was necessary in order for it to concur with the applicant's assertion that the equivalent-linear approach is suitable for strain levels as high as those for the 10^{-5} probability level. The equivalent-linear modeling approach produces a systematic shift in resonance peaks toward lower frequencies as the level of strain increases and also may predict a more dramatic reduction in AFs at higher frequencies. Accordingly, in the SER with open items, the staff identified Open Item 2.5-7, which requested that the applicant provide further justification for its claim that the equivalent-linear approach is suitable for higher strain levels.

In response to Open Item 2.5-7, the applicant referred to the 1993 EPRI study (EPRI TR102293), which presents a comprehensive study comparing the equivalent-linear method with nonlinear methods for seismic site response analysis. The applicant stated that the study involved a comparison using the equivalent-linear method using RASCAL/SHAKE and nonlinear methods with the programs SUMDES and TESS for three sites (Gilroy 2, Treasure Island, and Lotung, Taiwan). The study compared the actual recorded motion at each of the three sites with the solution from each method of analysis. The sites included soil layers ranging from sands and gravels to soft silts and stiff clays and had both high- and low- strain ground motion recordings. A comparison of the results showed reasonably good agreement between the different methods. In addition, the study analyzed higher ground motions (maximum input accelerations ranged from 0.5 g to 1.25 g) using a generic soil profile for Eastern North America using the same three programs. The applicant noted that the study also confirmed that the amplification factors obtained from the equivalent-linear method are in general agreement with those of the fully nonlinear methods. Furthermore, according to the EPRI study, the predicted peaks at the resonance frequency tend to be conservative using the equivalent-linear method.

With respect to the Vogtle site, the applicant stated that "the input motion is low compared to the range of motions used in the EPRI study and the site is generally stiffer. Therefore, the conclusion of the EPRI study applies, confirming the equivalent-linear method is adequate for the site response analysis at the Vogtle site."

The staff concludes that the applicant, in its response to Open Item 2.5-7, provided an adequate justification for using the equivalent-linear approach to perform site response calculations for the Vogtle ESP site. The applicant referred to the 1993 EPRI study (EPRI TR-102293), which showed that equivalent-linear method is in general agreement with fully nonlinear methods for the case studies considered. The EPRI study is also applicable to the Vogtle site because the

study considered a generic soil profile for Eastern North America. In addition, the maximum input peak accelerations ranged from 0.5 g to 1.25 g, which are larger than the expected ground motions at the Vogtle site. Futhermore, since the expected ground motions at the Vogtle site are less than, and the soil profile is generally stiffer than, the soil profiles considered in the EPRI study, the resulting soil nonlinearity is expected to be less at the Vogtle site.

In addition to Open Items 2.5-6 and 2.5-7, the staff noted in the SER with open items that the applicant did not perform any laboratory dynamic testing of the ESP soils, as specified in RG 1.138, "Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants," Revision 2, issued December 2003. Instead, as inputs to its site response calculations, the applicant relied on the EPRI and SRS shear modulus degradation and damping curves and assigned equal weights to the results for both sets of curves. This issue is discussed in greater detail in SER Section 2.5.4.4. Accordingly, in the SER with open items, the staff identified Open Item 2.5-19, in which the staff requested that the applicant justify its use of the EPRI and SRS shear modulus and damping curves in the absence of any dynamic testing of the ESP soils. In response to Open Item 2.5-19, the applicant submitted this information as Revision 4 of the SSAR. As part of its COL site investigation, the applicant developed site-specific strain-dependent shear modulus and damping relationships based on RCTS test results (performed on compacted backfill, Blue Bluff Marl, and Lower Sand samples), which are described in SSAR Section 2.5.4.7.5. Rather than recalculating site amplification factors using the site-specific strain-dependent shear modulus reduction and damping relationships, the applicant performed site response sensitivity calculations for a select number of cases in order to demonstrate that use of the SRS and generic EPRI strain-dependent shear modulus and damping curves are appropriate. The results of the applicant's sensitivity calculations are described in SSAR Section 2.5.2.9.3. The applicant evaluated the effects of the additional COL S-wave velocity and the strain dependent shear modulus and damping relationships based on RCTS test results, and compared these results to similar calculations performed using only ESP S-wave velocity data as well as the EPRI and SRS shear modulus degradation and damping curves. SER Figure 2.5.2-10 shows the applicant's results. The applicant concluded that the difference in amplification between the ESP and COL data is small.

In SSAR Section 2.5.2.9, the applicant conducted three sets of sensitivity calculations in order to evaluate: (1) the sensitivity of the AP1000 nuclear island responses to changes in the backfill S-wave velocity; (2) the effects of the backfill geometry on the site response and on the SSI response of the Nuclear Island; and (3) the effects of additional COL data on site response. In SER Section 2.5.2, the staff focused its review on the applicant's evaluation of the effects of the additional COL data on site response, which is described in SSAR Section 2.5.2.9.3. The staff reviewed the applicant's calculations to evaluate the sensitivity of the AP1000 nuclear island responses to changes in backfill S-wave velocity and the effects of the backfill geometry on the site response and on the SSI response of the Nuclear Island as part of SER Section 3.8.5.

The staff reviewed the results of the applicant's site response sensitivity calculations described in SSAR Section 2.5.2.9.3 and agrees with the applicant's conclusion that the differences between the applicant's original analysis using the ESP data and its analysis incorporating the additional COL data are insignificant. Thus, the staff concludes that the applicant's use of the

SRS and generic EPRI strain-dependent shear modulus and damping curves is appropriate. Therefore, the staff considers Open Item 2.5-19 to be resolved.

For the reasons stated above, the staff concludes that, overall, the applicant's site response methodology and results are acceptable. The applicant followed the general guidance provided in RG 1.208, and the results of the confirmatory site response calculations performed by the staff are similar to the applicant's results.

2.5.2.4.6 Ground Motion Response Spectra

SSAR Section 2.5.2.6 describes the method used by the applicant to develop the horizontal and vertical site-specific GMRS. To obtain the horizontal GMRS, the applicant used the performance-based approach described in RG 1.208 and ASCE/SEI Standard 43-05. The applicant developed the vertical GMRS by applying V/H ratios to the horizontal GMRS. The applicant based these V/H ratios on the information provided in NUREG/CR-6728 and Lee (2001).

Following the guidance in RG 1.208, the staff has recently adopted new terminology to differentiate between the different types of site and design ground motion response spectra. The staff now refers to the performance-based SSE as the site-specific GMRS. The GMRS represents the first part of the development of the SSE for a site as a characterization of the regional and local seismic hazard and must satisfy the requirements of 10 CFR 100.23. In accordance with Appendix S to 10 CFR Part 50, during the combined license phase, an additional check of the ground motion is required at the foundation level. Specifically, Appendix S to 10 CFR Part 50 states that the free-field foundation level ground motion must be represented by an appropriate response spectrum with a peak acceleration of at least 0.1 g. The GMRS becomes the site SSE if it exceeds the minimum requirements of Appendix S to 10 CFR Part 50. Otherwise, if any portion of the GMRS falls below the minimum response spectrum, then the site SSE becomes the ground motion spectrum that envelops the GMRS and the minimum response spectrum. As such, the final SSE must satisfy the requirements of both 10 CFR 100.23 and Appendix S to 10 CFR Part 50.

The staff reviewed the applicant's GMRS in terms of meeting the requirements of 10 CFR 100.23 with respect to the development of the SSE.

Horizontal GMRS

The ESP applicant for the Clinton, Illinois, site also used the performance-based approach to determine the horizontal GMRS. The staff's final SER for Clinton (ADAMS Accession No. ML0612204890) provides an extensive review and derivation of the performance-based approach. As described in RG 1.208, the performance-based approach combines a conservative characterization of the ground motion hazard with equipment/structure performance (fragility characteristics) to establish a risk-consistent GMRS. The performance-based GMRS is obtained by modifying the 10^{-4} UHRS at the free-field ground surface by a DF. The resulting GMRS meets the target performance goal of 10^{-5} per year for the mean annual probability of systems, structures, and components reaching the limit state of inelastic response. The performance-based approach achieves a relatively consistent annual

probability of plant component failure across the range of plant locations and structural frequencies. It does this by accounting for the slope of the seismic hazard curve, which changes with structural frequency and site location.

To verify the adequacy of the applicant's GMRS, the staff, in RAI 2.5.2-3, requested six PSHA hazard curves (1, 2.5, 5, 10, 25, and 100 Hz). The staff received the requested information from the applicant on June 18, 2007 (as supplemental information to RAI 2.5.2-3). Because the information was provided late in the review process, the staff identified this as Open Item 2.5-8 in the SER with open items. This was done to allow the staff additional time to complete its review of the applicant's response to RAI 2.5.2-3.

In response to RAI 2.5.2-3, the applicant provided the staff with soil hazard curves (corresponding to the top of the Blue Bluff Marl) at annual exceedance frequency levels of 10^{-4}, 10^{-5}, and 10^{-6}. The applicant obtained these hazard curves from its site response analysis described in SSAR Section 2.5.2.5. The applicant defined each hazard curve at a total of seven frequencies (0.5, 1, 2.5, 5, 10, 25, and 100 Hz). The applicant also obtained hazard curves at intermediate annual exceedance frequencies by performing interpolation. For each of the seven frequencies, the applicant fit a quadratic equation to the log (base 10) of the spectral ratios as a function of annual exceedance frequency.

Since the issuance of the SER with open items, the applicant changed the location of its GMRS from the top of the Blue Bluff Marl to the top of the structural backfill. At a public meeting on February 28, 2008, it was brought to the attention of the staff that the applicant's GMRS accounted for the effects of the material above the Blue Bluff Marl, which is contrary to the definition of the GMRS in RG 1.208. The applicant subsequently re-defined its GMRS and provided the updated soil hazard curves that corresponded to the top of the structural backfill.

The staff performed a confirmatory analysis in order determine the GMRS via the risk equation (Equation 1) as opposed to the direct convolution of the risk integral (Equation 3). The staff performed this confirmatory analysis in order to verify the acceptability of assuming a linear hazard curve in log-log space.

$$P_{FT} = \int_0^\infty H(a) f_a(a) da$$

Equation (4)

Since the seismic hazard curves have a slight downward curvature, assuming a linear fit results in slightly higher exceedance values and, as a result, slightly higher GMRS values, as illustrated in Table 2.5.2-10. Therefore, the staff concludes that the applicant's use of the approximate power law hazard curve is slightly conservative and therefore acceptable.

Table 2.5.2-10. Comparison of Site-Specific GMRS Values

Natural Frequency (Hz)	GMRS	
	Risk Integral (g)	Risk Equation (g)
1.0	0.276	0.285
2.5	0.714	0.775
5.0	0.693	0.709
10.0	0.702	0.789

The DF recommended in ASCE/SEI 43-05 (Equation 1) is slightly unconservative for β=0.3 and conservative for β of 0.4 to 0.6. To evaluate the significance of the range of β values on the DF, the staff determined the unacceptable performance frequency values (PFT) for the GMRS values for four natural frequency values 1, 2.5, 5, and 10 Hz. The applicant determined the four GMRS values shown in Table 2.5.2-10 using the performance-based approach as described in ASCE/SEI 43-05, which assumes a β value of 0.4 and a target performance goal of $1x10^{-5}$/yr. The staff used the four hazard curves provided by the applicant to determine PFT via direct integration of the risk integral (Equation 3) for β ranging from 0.3 to 0.6. As shown below in Table 2.5.2-11, the PFT values for β=0.3 are only slightly larger than the target value of $1x10^{-5}$/yr (with the exception of frequencies of 2.5 and 10 Hz, which are less than the target value of $1x10^{-5}$/yr). Since the PFT values for β=0.3 are only slightly larger (at frequencies of 1.0 and 5.0 Hz) than the target performance goal of 10^{-5}/yr and fragility β values of 0.3 are not common for SSCs, the staff concludes that the applicant's assumption that β=0.4 for determining the GMRS is acceptable.

Table 2.5.2-11. Unacceptable Performance Frequency Values for β Ranging from 0.3 to 0.6

Frequency (Hz)	GMRS (g)	PFT*10^{-5}/yr			
		β = 0.3	β = 0.4	β = 0.5	β = 0.6
1.0	0.285	1.073	0.925	0.661	0.506
2.5	0.775	0.689	0.706	0.539	0.500
5.0	0.709	0.950	0.920	0.668	0.539
10.0	0.789	0.518	0.579	0.500	0.500

As determined by the staff in its final SER for the Clinton Early Site Permit, essentially elastic behavior (or OSID (onset of significant inelastic deformation)) is just beyond the occurrence of insignificant (or localized) inelastic deformation, and in this way corresponds to essentially elastic behavior. As such, OSID of an SSC can be expected to occur well before seismically-induced core damage, resulting in much larger frequencies of OSID than seismic core damage frequency (SCDF (seismic core damage frequencies)) values. To further demonstrate that the frequency of OSID is larger than the SCDF, the staff used the four Vogtle ESP hazard curves (1, 2.5, 5, and 10 Hz) to calculate SCDF values each of the GMRS values. In performing this calculation of SCDF, the staff used the risk integral (Equation 3) with the complete range of β values (0.3 to 0.6) and assumed that the seismic margin (Ms) against core damage is 1.67 for new standard plant designs as specified in the staff requirements memorandum (SRM), dated July 21, 1993, on SECY 93-087. As shown in Table 2.5.2-12

below, SCDF values for the four natural frequencies and β values vary from 0.022×10^{-5}/yr to 0.318×10^{-5}/yr.

Table 2.5.2-12. SCDF Values for Vogtle GMRS

Frequency (Hz)	GMRS (g)	SCDF*10^{-5}/yr			
		β = 0.3	β = 0.4	β = 0.5	β = 0.6
1.0	0.285	0.318	0.210	0.152	0.120
2.5	0.775	0.072	0.055	0.052	0.055
5.0	0.709	0.156	0.105	0.086	0.079
10.0	0.789	0.022	0.027	0.033	0.040

For comparison, NUREG-1742 shows, based on the results of seismic PRAs of 25 nuclear power plants, that the median value for mean core damage frequency is 1.2×10^{-5}/yr. Therefore, by setting the target performance goal, PFT, to be a frequency of onset of inelastic deformation (FOSID) value of 1×10^{-5}/yr, the resulting GMRS computed using the ASCE/SEI 43-05 methodology provides SCDF values that are substantially lower than those for most of the 25 nuclear power plants provided in NUREG-1742. For the natural frequencies of 5 and 10 Hz and for β values of 0.4 and 0.5, SCDF is about 1×10^{-6}/yr to 3×10^{-7}/yr for the Vogtle ESP performance-based SSE, which is about 12 to 40 times lower than the median of the mean SCDF for the 25 nuclear power plants considered in NUREG-1742.

In summary, the staff concludes that the applicant provided sufficient information in response to RAI 2.5.2-3 in order for the staff to verify the adequacy of the applicant's GMRS. Based on the results of the confirmatory analyses described above, the staff concludes that the applicant's use of the approximate power law hazard curve to determine the GMRS is slightly conservative and therefore acceptable. In addition, the staff concludes that the applicant's assumption that β=0.4 for determining the GMRS is acceptable. Furthermore, the staff concludes that the applicant targeted a sufficiently low structural performance frequency value (1×10^{-5}/yr), which is set equivalent to FOSID (frequency of onset of significant inelastic deformation), such that the resulting performance-based GMRS achieves an SCDF which is approximately 12 to 40 times smaller than the median of the mean SCDF for the 25 nuclear power plants considered in NUREG-1742. Therefore, the staff considers Open Item 2.5.2-8 to be resolved.

<u>Vertical GMRS</u>

To compute the vertical GMRS, the applicant used a combination of V/H ratios obtained from NUREG/CR-6728 and Lee (2001). Since the V/H ratios presented in NUREG/CR-6728 and Lee (2001) are functions of magnitude, source distance, and local site conditions, the applicant developed V/H ratios corresponding to the final high-frequency (**M** 7.2, 130 km) and low-frequency (**M** 5.6, 12 km) controlling earthquakes described in SSAR Section 2.5.2.4. The applicant referred to these high- and low-frequency controlling earthquakes as "near" and "far" events, respectively.

In Part A of RAI 2.5.2-24, the staff asked the applicant to justify its rationale for assigning the approximate weights of 1:3 to the V/H ratios corresponding to the respective "near" and "far"

events. In response to Part A of RAI 2.5.2-24, the applicant concluded that it developed this weighting based on a review of the high- and low-frequency distance deaggregations as well as the relative contributions of the 10^{-4} and 10^{-5} hazard levels to the GMRS. Based on its review of the high-frequency distance deaggregation at the 10^{-4} hazard level (shown in SSAR Figure 2.5.2-30), the applicant concluded that approximately three-fourths of the area under the 10^{-4} hazard probability density curve corresponds to the "far" event, while about one-fourth of the area under the curve corresponds to the "near" event. In comparison, the applicant found that the relative contribution of the "near" and "far" events at the 10^{-5} hazard level is approximately the same. The applicant also reviewed the low-frequency distance deaggregation (shown in SSAR Figure 2.5.2-31) at both the 10^{-4} and 10^{-5} hazard levels and concluded that the hazard is dominated by the "far" event.

As stated in its response to Part A of RAI 2.5.2-24, the applicant focused on the 10^{-4} high-frequency distance deaggregation and the associated weights of 1:3 to determine the relative contributions of the respective "near" and "far" events because the GMRS is generally only slightly higher than the 10^{-4} ground motion. The applicant used the high-frequency distance deaggregation, rather than the low-frequency distance deaggregation, because it concluded "the low-frequency end of the spectrum is not as sensitive to magnitude and distance nor, therefore, to the distinction between 'near' and 'far' events."

The staff concludes that the applicant's use of NUREG/CR-6728 to develop V/H ratios is acceptable because the report considers the effects of magnitude and distance on spectral ratios and is applicable to CEUS soil sites. Previous regulatory guidance (RG 1.60 and NUREG/CR-0098, "Development of Criteria for Seismic Review of Selected Nuclear Power Plants") recommended that the V/H ratio be fixed at two-thirds, independent of ground motion frequency, earthquake magnitude, distance, and local site conditions. More recent regulatory guidance (RG 1.208) recommends the use of V/H ratios that incorporate magnitude, distance, and local site conditions, such as those found in NUREG/CR-6728. Because of the observed similarity between the GMRS to the 10^{-4} soil UHRS, and because V/H ratios are observed to be higher in the near-field region and in the high-frequency range of the response spectrum (e.g., NUREG/CR-6728), the staff concurs with the applicant's rationale for weighting the relative contributions of the "near" and "far" events based on the 10^{-4} high-frequency distance deaggregation.

In Part B of RAI 2.5.2-24, the staff asked the applicant to discuss the similarities and differences between the site-specific soil profile used by Lee (2001) and the VEGP soil profile. In response to Part B of RAI 2.5.2-24, the applicant stated that the SRS site-specific soil profile is not published in Lee (2001) so that a comparison with the ESP profile could not be made. The applicant also stated that given the proximity of the ESP site to the SRS, it assumed that the site conditions at the SRS are more comparable to those at the ESP site than to the generic CEUS profile used in NUREG/CR-6728.

In Part C of RAI 2.5.2-24, the staff asked the applicant to provide justification for the relative weights assigned to the NUREG/CR-6728 and Lee (2001) results and final smoothing to develop the final V/H ratios for the ESP site. In response, the applicant stated that it used an approximate envelope of the two results. For frequencies between 1 and 100 Hz, the applicant approximated the V/H ratios of Lee (2001) by two log-log line segments. For frequencies less

than 1 Hz, the applicant used a constant ratio of 0.5, which is greater than both Lee (2001) and NUREG/CR-6728, and more closely resembles the V/H values in RG 1.60.

For CEUS soil sites, RG 1.208 endorses the procedure provided in NUREG/CR-6728 to determine a WUS-to-CEUS transfer function to modify the WUS V/H ratios. The staff, therefore, concludes that the applicant's use of the formula provided in Appendix J to NUREG/CR-6728 to determine the ESP site V/H ratios is acceptable. However, the formula in Appendix J, shown in Equation (2) in SER Section 2.5.2.6, requires the input of site-specific V/H ratios, V/HCEUS,Soil,Model, based on ground motion modeling. For this site-specific V/H ratio, the applicant used the results of Lee (2001), which are applicable to the SRS soil profile, and NUREG/CR-6728, based on a generic CEUS soil profile. SER Figure 2.5.2-9 shows the applicant's final V/H ratios as a function of frequency. At frequencies above approximately 1 Hz, the applicant estimated the V/H ratios of Lee (2001) by two log-log line segments. At frequencies between 1–2 Hz and 10–20 Hz, this log-log line segment is less that the V/H ratios of Lee (2001). In the SER with open items, the staff concluded that the applicant did not provide adequate justification to support the applicability of either the Lee (2001) or the NUREG/CR-6728 soil V/H ratios at the ESP site. The staff further concluded that the applicant's approximate envelope was arbitrary. For example, the applicant did not provide its rationale for excluding the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10-20 Hz frequency ranges. Accordingly, in Open Item 2.5-9, in the SER with open items, the staff requested that the applicant provide more detail regarding the applicability of the Lee (2001) and the NUREG/CR-6728 V/H ratios to the ESP site. In addition, the staff requested that the applicant provide its justification for the use of an approximate envelope of the Lee (2001) and NUREG/CR-6728 V/H ratios.

In response to the staff's request to provide more detail regarding the applicability of the Lee (2001) and the NUREG/CR-6728 V/H ratios to the ESP site, the applicant stated that it considered the implementation of the NUREG/CR-6728 approach in two cases as a guide for recommending a V/H for the Vogtle ESP site. In the first case, the applicant relied on the transfer functions presented in Appendix J to NUREG/CR-6728, where the CEUS soil model corresponds to a generic "deep soil" (500 ft) site. In the second case, the applicant relied on an evaluation of V/H for the nearby SRS (Lee, 2001), which also followed the NUREG/CR-6728 approach. The applicant stated that the subsurface S-wave velocity profiles and depths to water table are similar at Vogtle and at the SRS. The applicant also stated that "while the results from the SRS (first case) may seem arguably the most applicable for the Vogtle site, the generic nature of the first case is consistent with the generic character of the rock V/H recommendation of the NUREG. Therefore both results are considered in the SSAR."

Regarding additional justification for the use of an approximate envelope of the Lee (2001) and NUREG/CR-6728 V/H ratios, the applicant stated that, similar to the RG 1.60 V/H ratios and the recommended V/H functions for rock sites in NUREG/CR-6728, it intended to derive a V/H (based on both Lee and NUREG/CR-6728 soil) that is relatively simple and smooth, yet also reflects the following general features:

- Similar to RG 1.60 and the NUREG/CR-6728 V/H for rock, V/H is higher at high frequencies than at low frequencies;

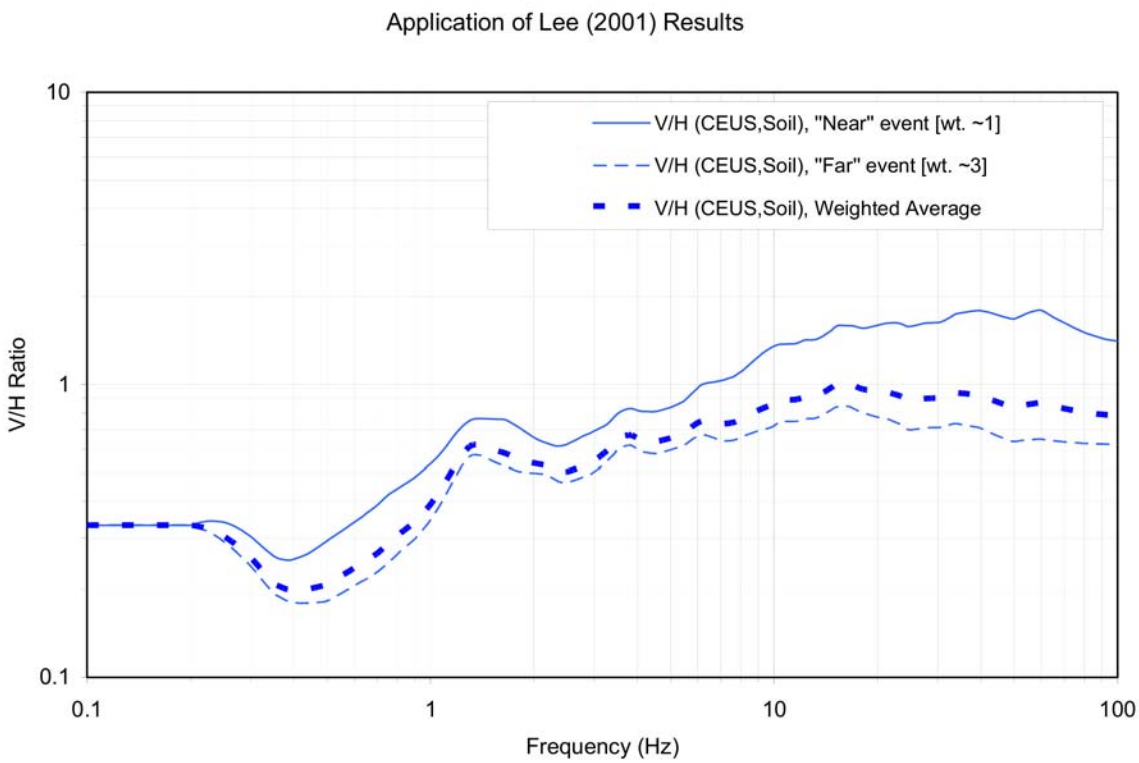
- The two cases evaluated suggest that a relatively flat V/H value at high frequencies, slightly lower (~0.9) than that given by RG 1.60 (1.0);
- both cases suggest a lower V/H value (0.5) than that given by RG 1.60 (2/3) in the lowest frequencies;
- the envelope of the two cases suggests that the transition between the higher V/H at high frequencies and the lower V/H at low frequencies is more gradual than the relatively abrupt transition in Reg. Guide 1.60; and
- the V/H at high frequencies is sustained at its high value longer toward lower frequencies (flatter) than suggested by CEUS rock V/H from the NUREG/CR-6728.

In Open Item 2.5.2-9, the staff also requested that the applicant provide its rationale for excluding the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10–20 Hz frequency ranges. In response, the applicant stated that given the complexities, assumptions, and uncertainties of developing CEUS, deep soil V/H for the Vogtle site, as well as the desire to develop a smooth function, it developed a conservative envelope of the V/H for the two cases. The applicant further stated that three discrete acceleration intervals (≤0.2g, 0.2 – 0.5 g, and >0.5 g) for which the rock V/H is defined in NUREG/CR-6728 also suggests approximate evaluations of V/H. For this reason, some peaks are cut (e.g., 1.3 Hz) and valleys are filled (e.g., 2.5 Hz) by the applicant. However, the applicant stated that it did not consider this to be significant relative to other uncertainties in ground motion evaluations.

Based on the applicant's response to Open Item 2.5-9, the staff concludes that the applicant's use of the generic CEUS soil profile V/H ratios provided in Appendix J is acceptable because the applicant also considered the V/H ratios developed for the adjacent SRS, which has a similar S-wave velocity profile to the Vogtle site and is therefore more applicable. Furthermore, the V/H ratios for the SRS soil profile are always larger than the generic CEUS soil profile, and the applicant used an approximate envelope of the two results, with the exception of the peaks excluded in the 1–2 Hz and 10–20 Hz frequency ranges. The staff, however, concludes that the applicant's exclusion of the peaks observed in the Lee (2001) V/H ratios in the 1–2 Hz and 10-20 Hz frequency ranges is not significant. As observed in SER Figure 2.5.2-9, the peak excluded in the 10–20 Hz frequency range is approximately 10 percent larger than the approximate envelope, while the peak in the narrow 1–2 Hz frequency range is less than 20 percent larger. Furthermore, the valleys on either side of the narrow peak at 1–2 Hz have also been filled. The staff notes that the applicant's final vertical GMRS is not changed significantly as a result of this smoothing. In addition, the staff notes that the applicant's use of 100 km instead of 130 km distance to obtain V/H corresponding to the **M** 7.2, 130-km earthquake from the Lee (2001) results is conservative because V/H decreases with distance for a given magnitude. This would effectively increase the final V/H based on the Lee (2001) results for the SRS shown in SER Figure 2.5.2-9. Therefore, the staff considers Open Item 2.5-9 to be resolved.

Based on its review of SSAR Section 2.5.2.6, the staff thus concludes that, overall, the applicant's horizontal and vertical GMRS, which are shown in SSAR Figure 2.5.2-21, are acceptable. The applicant followed the general guidance provided in RG 1.208 to develop both the horizontal and vertical GMRS for the Vogtle site. In SSAR Table 1-1, the applicant identified

the GMRS as an ESP site characteristic[9]. For the reasons set forth above, the staff agrees with the applicant's GMRS as a site characteristic, which appears as SER figure 2.5.2-25 and in Appendix A of the SER.



Note: Considering the relative contribution of the "near" and "far" events to the horizontal SSE design response spectrum, the approximately 1:3 weighted average is shown.

Figure 2.5.2-24. Plots of recommended V/H CEUS,Soil ratios using the results from Lee (2001) for the SRS (reproduced from SSAR Figure 2.5.2-41).

9 The staff notes that this site characteristic for the GMRS is not bounded by the AP1000 certified design response spectrum (CSDRS). However, the staff considers the GMRS value determined for the Vogtle site to be within the range of values that new reactor designs generally are engineered to withstand. Accordingly, the staff considers the approval of this site characteristic to be consistent with the staff's determination that, from a geologic and seismologic perspective, the ESP site is suitable and meets the applicable requirements of Part 52 and Part 100. Whether the reactor design ultimately chosen for the site bounds the GMRS site characteristic will be determined at the COL stage.

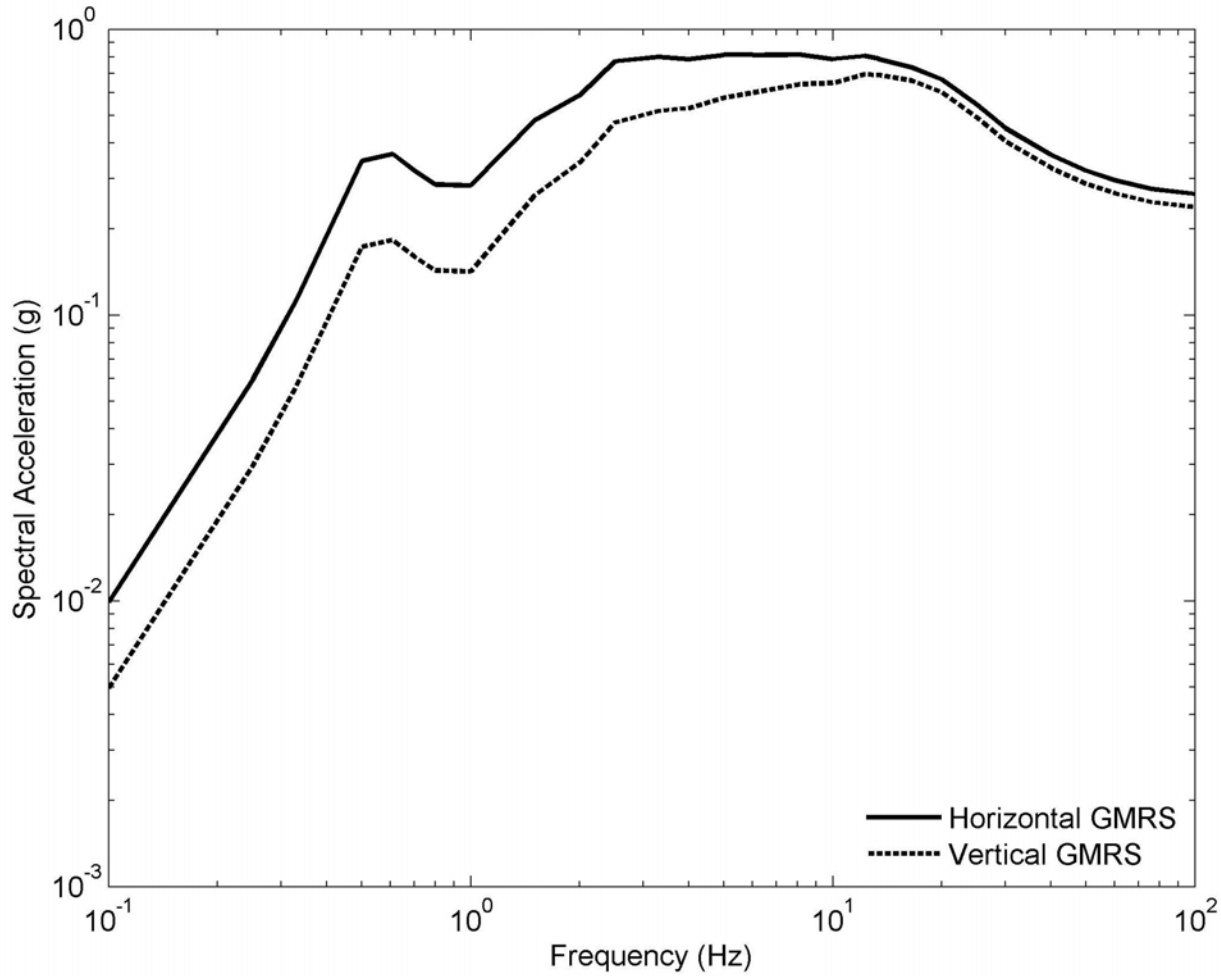


Figure 2.5.2-25. Plots of the horizontal and vertical GMRS (based on the information provided in SSAR Table 2.5.2-22b and SSAR Section 2.5.2.7.1.3).

2.5.2.4 Conclusions

As set forth above, the staff reviewed the seismological information submitted by the applicant in SSAR Section 2.5.2. On the basis of its review of SSAR Section 2.5.2, the staff finds that the applicant has provided a thorough characterization of the seismic sources surrounding the site, as required by 10 CFR 100.23. In addition, the staff finds that the applicant has adequately addressed the uncertainties inherent in the characterization of these seismic sources through a PSHA, and this PSHA follows the guidance provided in RGs 1.165 and 1.208. The staff concludes that the controlling earthquakes and associated ground motion derived from the applicant's PSHA are consistent with the seismogenic region surrounding the ESP site. In addition, the staff finds that the applicant's GMRS, which was developed using the performance-based approach, adequately represents the regional and local seismic hazards

and accurately includes the effects of the local ESP subsurface properties. The staff concludes that the proposed ESP site is suitable with respect to the vibratory ground motion criteria for new nuclear power plants and meets the applicable requirements of 10 CFR 100.23.

2.5.3 Surface Faulting

In SSAR Section 2.5.3, the applicant evaluated the potential for tectonic and nontectonic surface and near-surface deformation at the VEGP ESP site. The applicant included a review of geologic, seismic, and geophysical investigations in SSAR Section 2.5.3.1.1 to assess the potential for surface deformation that could impact the ESP site. In SSAR Sections 2.5.3.1.2 and 2.5.3.1.4, the applicant assessed geologic evidence, or the absence of evidence, for surface deformation by evaluating known geologic structures in the VEGP site vicinity. SSAR Section 2.5.3.3 provides a review of seismicity within the site vicinity (a 40 km (25 mi) radius of the VEGP site) and addresses any correlation between the seismicity and capable tectonic structures. SSAR Sections 2.5.3.1.4 and 2.5.3.1.5 evaluate the tectonic structures in the site area, how these structures relate to the regional tectonics, and any ages of deformation associated with these structures. The applicant discussed the potential for tectonic and/or nontectonic deformation at the VEGP site in SSAR Section 2.5.3.1.8. On the basis of this evaluation, the applicant concluded that: (1) no capable tectonic sources exist within the VEGP site area (within an 8 km (5 mi) radius); (2) the potential for tectonic fault displacement is negligible; (3) only limited potential exists for nontectonic surface deformation within the site area; and (4) the potential for nontectonic surface deformation can be mitigated by excavation of materials.

2.5.3.1 Technical Information in the Application

2.5.3.1.1 Geologic, Seismic, and Geophysical Investigations

In SSAR Section 2.5.3.1, the applicant described the geologic, seismic, and geophysical investigations performed to assess the potential for tectonic and nontectonic surface and near-surface deformation at and within an 8 km (5 mi) radius of the VEGP site. The applicant reviewed previous VEGP site investigations, published geologic mapping, previous SRS investigations, previous seismicity data, previous seismic reflection data, current seismic reflection studies, and current aerial and field reconnaissance. The applicant stated that geologic and geomorphic investigations within and beyond the site vicinity (a 40 km (25 mi) radius) and interpretation of aerial photographs taken within the site area (an 8 km (5 mi) radius) were used to supplement existing information for documenting the presence or absence of features indicative of potential Quaternary (1.8 million years ago (mya) to present) fault activity at or near the site. Based on the information presented in SSAR Sections 2.5.3.1.1 through 2.5.3.1.7, the applicant concluded that no capable tectonic sources occur within the site area and that there is negligible potential for surface or near-surface fault rupture.

<u>Data from Previous Investigations</u>

SSAR Section 2.5.3.1.1 describes previous site area investigations conducted for VEGP Units 1 and 2. SSAR Section 2.5.3.1.2 describes the applicant's review of published geologic maps for analyzing surface deformation within the site area. The applicant reviewed previous SRS

investigations (SSAR Section 2.5.3.1.3), including geologic, seismic, hydrologic, and geophysical investigations, and concluded that the Pen Branch fault does not exhibit surface deformation, is not a capable tectonic structure, and is not favorably oriented in the modern-day stress regime to experience displacement. In SSAR Section 2.5.3.1.4, the applicant reviewed historical seismicity and microseismicity data for the site vicinity (within a 40 km (25 mi) radius) and the site area (within an 8 km (5 mi) radius). The applicant stated that no recent earthquake activity has occurred within the site area and that the closest microearthquake to the ESP site is located on the SRS, about 11 km (7 mi) to the northeast of the VEGP. In SSAR Section 2.5.3.1.5, the applicant discussed previous seismic reflection studies and again concluded that the Pen Branch fault is not a capable tectonic structure.

Data from Current Investigations

The applicant described current seismic reflection studies in SSAR Section 2.5.3.1.6 and current aerial and field reconnaissance studies in SSAR Section 2.5.3.1.7. These investigations were performed for the ESP application in order to image the Pen Branch fault beneath the surface and to check for evidence of surface faulting within the ESP site vicinity. The applicant stated that the Pen Branch fault was clearly imaged beneath the ESP site area in the seismic reflection data. The applicant concluded that, based on aerial and field reconnaissance data, no geomorphic features within the site vicinity display evidence for surface rupture, surface warping, or fault offset.

2.5.3.1.2 Geologic Evidence, or Absence of Evidence, for Surface Deformation

In SSAR Section 2.5.3.2, the applicant stated that four bedrock faults are mapped within a 5-mile radius of the VEGP ESP site. These faults, interpreted from seismic reflection, borehole, gravity, and magnetic and/or ground water data, include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults. Of these four faults, only the Pen Branch fault is interpreted to extend beneath the VEGP ESP site area, motivating the applicant to perform a detailed investigation of the Pen Branch fault as it relates to the ESP site. A complete description of the applicant's investigation of the Pen Branch fault is included in SSAR Section 2.5.1.2.4.1. The remaining three faults, mapped in relation to the SRS, are located within a 5-mile radius of the VEGP site, but are not interpreted to extend beneath the site. The applicant concluded that none of the four faults mapped within the site area displays evidence of surface rupture and that none of these faults is a capable tectonic structure.

Pen Branch Fault

The applicant presented its conclusions regarding the Pen Branch fault in SSAR Sections 2.5.3.2.1 and 2.5.3.5.1. The Pen Branch fault is more than 30 km (greater than 20 mi) in length along its northeastern strike direction and forms the northwest boundary of the Dunbarton Triassic basin. The fault initially accommodated regional crustal extension during the Mesozoic (248 to 65 mya) by normal slip during the Triassic (248 to 206 mya) period to form the Dunbarton Basin, and was reactivated in the Cretaceous (144 to 65 mya) and Tertiary (65 to 2 mya) as a reverse fault. The applicant stated that the Pen Branch fault is not exposed or geomorphically expressed at the surface, and borehole and seismic reflection data collected at the SRS show no evidence for post-Eocene slip on the fault. According to the applicant, the

Ellenton Quaternary terrace (Qte) at the SRS, dated between 350,000 and 1 mya in age, was evaluated for the ESP application and demonstrates no Quaternary tectonic deformation of the terrace surface within a resolution of about 1 m (3 ft). The applicant stated that both previous and more recent investigations to define the presence or absence of surface deformation related to displacement on the Pen Branch fault indicate no evidence of Quaternary (1.8 mya to present) deformation. Based on these findings, the applicant concluded that the Pen Branch fault is not interpreted as a capable tectonic source.

Ellenton Fault

In SSAR Sections 2.5.3.2.2 and 2.5.3.5.2, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Ellenton fault, located about 7.4 km (4.6 mi) from the VEGP site. As initially mapped by Stieve and Stephenson (1995), the Ellenton fault was a north-northwest striking fault located in the Dunbarton Basin between the Upper Three Runs and Pen Branch faults. The applicant stated that the Ellenton fault likely does not exist because the data used to suggest the existence of this potential structure were acknowledged to be of poor quality, there is no geomorphic expression of this fault at the surface, and the fault does not appear on the most recent SRS fault maps by Cumbest et al. (2000). Therefore, the applicant concluded that this fault could not represent a capable tectonic structure within the site area.

Steel Creek Fault

In SSAR Sections 2.5.3.2.3 and 2.5.3.5.3, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Steel Creek fault, located about 4.8 km (3 mi) from the VEGP site. This fault is interpreted to be more than 17.7 km (greater than 11 mi) in length, with a northeast strike and a northwest dip, and exhibits reverse slip movement. The Steel Creek fault cuts upward into Cretaceous units, but its uppermost extension remains unresolved. According to the applicant, longitudinal profiles along Quaternary fluvial terraces overlying the surface projection of the fault, with a resolution of 2-3 m (7-10 ft), show no evidence of warping or faulting of the terrace surfaces and therefore provides no evidence for Quaternary (1.8 mya) deformation. Based on a lack of geomorphic surface expression, the applicant concluded that the Steel Creek fault is not a capable tectonic structure within the site area.

Upper Three Runs Fault

In SSAR Sections 2.5.3.2.4 and 2.5.3.5.4, the applicant summarized geologic evidence for the absence of surface deformation due to slip on the Upper Three Runs fault, located about 8 km (5 mi) from the VEGP site. The fault is not included on the more recent fault map of the SRS by Cumbest et al. (2000), but its northernmost trace is roughly parallel to the Tinker Creek fault that is shown on the Cumbest et al. (2000) fault map. According to the applicant, seismic profiles show that Coastal Plain sediments are not offset or deformed by this structure, and the fault is interpreted to be confined to basement rocks. Based on these findings and the fact that there is no geomorphic surface expression of this fault, the applicant concluded that it is not a capable tectonic structure within the site area.

2.5.3.1.3 Correlation of Earthquakes with Capable Tectonic Sources

The applicant summarized seismicity data for the VEGP ESP site vicinity in SSAR Sections 2.5.3.3 and 2.5.3.1.4 in order to determine whether any correlation exists between seismicity and capable tectonic structures. Figure 2.5.3-1 of this SER, taken from SSAR Figure 2.5.1-16, shows diffuse microseismic activity recorded by the SRS seismic recording network since 1976 within a 40 km (25 mi) radius of the VEGP site.

Based on the data shown in this figure, the applicant concluded that there is no spatial correlation of earthquake epicenters with known or postulated faults. The applicant reviewed published literature to further conclude that there are no known historical earthquake epicenters associated with bedrock faults or known tectonic structures in the site vicinity. The EPRI catalog of historical seismicity demonstrates that no known earthquake greater than body wave magnitude (mb) 3 has occurred within the site vicinity, while the SRS seismic recording network documents no recent microseismic activity (mb less than 3) within an 8 km (5 mi) radius of the VEGP site since 1976. The applicant stated that the nearest microearthquake event to the VEGP ESP site was located about 11 km (7 mi) northeast of the VEGP site on the SRS.

The applicant described three small earthquakes that occurred between 1985 and 1997 with magnitudes ranging between 2.0 and 2.6 and depths ranging from 2.5 to 6 km (1.5 to 3.5 mi). In addition to these events, the applicant described a magnitude 3.2 event located north of the SRS in Aiken, South Carolina, and a series of several small events (magnitudes less than or equal to 2.6) that occurred in 2001–2002 within the SRS boundaries. The applicant reviewed the locations of these events with respect to mapped faults in the ESP site vicinity, as well as previous studies of these events by Stevenson and Talwani (2004), Talwani et al. (1985), and Crone and Wheeler (2000), and concluded that there is no spatial correlation of seismicity with known or postulated faults or geomorphic features.

2.5.3.1.4 Ages of Most Recent Deformations

In SSAR Section 2.5.3.4, the applicant stated that, based on information presented in SSAR Section 2.5.3.2, none of the four faults (Pen Branch, Ellenton, Steel Creek, or Upper Three Runs) exhibits Quaternary (1.8 mya to present) displacement. Thus, the applicant concluded that none is considered a capable tectonic structure. In particular, the applicant stated that the Pen Branch fault exhibits no post-Eocene (33.7 mya to present) displacement.

2.5.3.1.5 Relationship of Site Area Tectonic Structures to Regional Tectonic Structures

SSAR Section 2.5.3.5 discusses the four faults identified within the site area and previously discussed in SER Section 2.5.3.1.2. Of these four faults, the applicant stated that only the Pen Branch fault occurs west of the SRS and within the ESP site area. The applicant concluded that, based on a review of all available data, none of these four faults is considered a capable tectonic structure and none is associated with any capable regional tectonic structure.

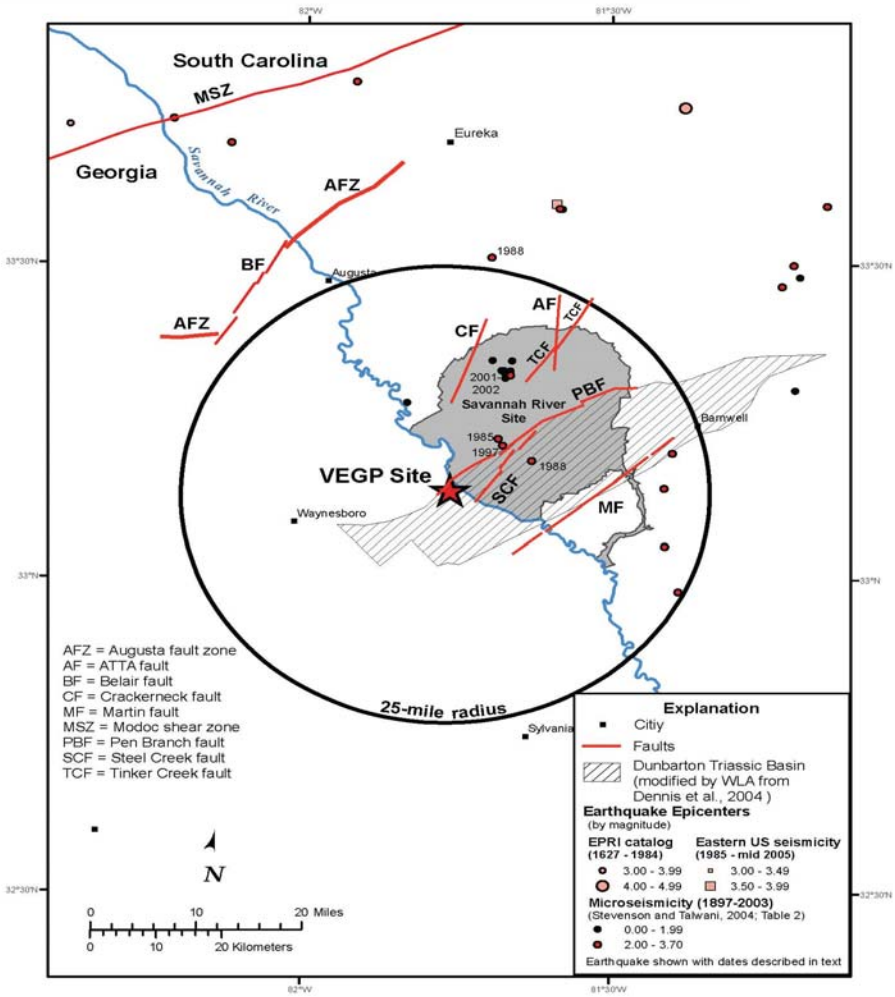


Figure 2.5.3-1 - Site Vicinity Tectonic Features and Seismicity (Reproduced from SSAR Figure 2.5.1-16)

2.5.3.1.6 Characterization of Capable Tectonic Sources

The applicant described characterization of capable tectonic sources in SSAR Section 2.5.3.6 and reiterated that no capable tectonic structures occur within 8 km (5 mi) of the VEGP site based on the following geologic evidence:

1. The Pen Branch fault is not exposed or expressed at the surface. Field reconnaissance and aerial photograph interpretations performed for the ESP study confirmed that there is no surface exposure of the fault or geomorphic expression indicative of Quaternary deformation.

2. Snipes et al. (1993) indicated that there was no displacement of a Quaternary soil horizon overlying the projected trace of the Pen Branch at the SRS, and the youngest horizon offset by fault displacement on the Pen Branch was the Dry Branch Formation of late Eocene age.

3. Geomatrix (1993) evaluated longitudinal profiles of Quaternary fluvial river terraces on the SRS and concluded that no evidence for warping or faulting of the terraces existed within a resolution limit of 2 to 3 m (7 to 10 ft).

4. Longitudinal terrace profiles across the now well-located Pen Branch fault also indicated no deformation of the Ellenton terrace (estimated to be 350,000 to 1 million years old) within a resolution limit of 1 m (3 ft).

5. Also as part of the ESP study, geomorphic analysis of the Ellenton terrace, which overlies the surface projection of the Pen Branch, demonstrates a lack of tectonic deformation of this Quaternary surface within a resolution limit of 1 m (3 ft). Details of this ESP study are presented in SSAR Section 2.5.1.2.4.3.

2.5.3.1.7 Designation of Quaternary Deformation Zones Requiring Detailed Investigation

In SSAR Section 2.5.3.7, the applicant concluded that no zones of Quaternary deformation requiring detailed fault investigation exist within the VEGP site area based on the absence of any Quaternary deformation features in the ESP site area.

2.5.3.1.8 Potential for Tectonic or Nontectonic Deformation at the Site

In SSAR Section 2.5.3.8.1, the applicant concluded that the potential for tectonic deformation at the ESP site is negligible and stated that no new information has been reported since the original site studies for VEGP Units 1 and 2 in the early 1970s to suggest the existence of Quaternary surface deformation. Also in SSAR Section 2.5.3.8, the applicant addressed the potential for nontectonic deformation features at the VEGP ESP site, including dissolution collapse features and clastic dikes.

In SSAR Section 2.5.3.8.2, the applicant specifically discussed the potential for nontectonic surface deformation at the ESP site, including interpretation of dissolution collapse features and

clastic dikes. Regarding dissolution collapse features, which are discussed in SSAR Section 2.5.3.8.2.1, the applicant indicated that small-scale structures, including warped bedding, fractures, joints, minor fault offsets, and injected sand dikes, identified in the walls of a trench at the VEGP site were local features related to dissolution of the Utley Limestone and subsequent collapse of overlying Tertiary sediments. Age of these features was interpreted to be younger than Eocene-Miocene host sediments and older than the overlying late-Pleistocene Pinehurst Formation. The applicant stated that no late Pleistocene or Holocene dissolution features were identified at the site. The applicant indicated that mitigation of collapse due to dissolution of the Utley Limestone, which overlies the Blue Bluff Marl (BBM) at the site, could be accomplished by planned excavation and removal of the Utley Limestone to establish the foundation grade of the plant atop the BBM.

In SSAR Section 2.5.3.8.2.2, the applicant addressed clastic dikes, described as relatively planar, narrow (centimeter-to-decimeter wide) clay-filled features that flare upwards and are decimeters to meters in length. The applicant stated that Bechtel (1984) distinguished two types of clastic dikes in the walls of the trench on the VEGP site where dissolution collapse features were found. The first type of clastic dikes was interpreted to be sand dikes that resulted from injection of poorly consolidated fine sand into overlying sediments; the second type was clastic dikes produced by weathering and soil formation processes that were enhanced along fractures that formed during dissolution collapse. Bechtel (1984) concluded that the dikes were primarily a weathering phenomenon controlled by depth of weathering and paleosol development in Coastal Plain sediments and subsequent erosion of the land surface. According to the applicant, clastic dike features identified by Bartholomew et al. (2002) within the site area were observed during the ESP field reconnaissance. The applicant interpreted these features to be nontectonic in origin, although Bartholomew et al. (2002) suggested that they might be evidence for paleoearthquakes associated with late-Eocene to late-Miocene faulting, possibly along the Pen Branch fault.

2.5.3.2 Regulatory Evaluation

The acceptance criteria for evaluating the potential for surface or near-surface tectonic and nontectonic deformation are based on meeting the relevant requirements of 10 CFR 52.17 and 10 CFR Part 100.23. The staff considered the following regulatory requirements in reviewing the applicant's discussion of information on surface faulting:

1. 10 CFR 53.17(a)(1)(vi), which requires that an ESP application contain a description of the geologic and seismic characteristics of the proposed site.

2. 10 CFR 100.23(c), which requires an ESP applicant to investigate geologic, seismic, and engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site, to provide sufficient information to support evaluations performed to determine the SSE Ground Motion, and to permit adequate engineering solutions to actual or potential geologic and seismic effects at the proposed site.

3. 10 CFR 100.23(d), which requires that geologic and seismic siting factors considered for design include a determination of the SSE Ground Motion for the site, the potential for

surface tectonic and non-tectonic deformation, the design bases for seismically-induced floods and water waves, and other design conditions including soil and rock stability, liquefaction potential, and natural and artificial slope stability. Siting factors and potential causes of failure to be evaluated include physical properties of materials underlying the site, ground disruption, and effects of vibratory ground motion that may affect design and operation of the proposed power plant.

The basic geologic and seismic information assembled by the applicant in compliance with the above regulatory requirements should also be sufficient to allow a determination at the COL stage of whether the proposed facility complies with the following requirements in Appendix A to 10 CFR Part 50:

1. 10 CFR Part 50, Appendix A, GDC 2, which requires that SSCs important to safety be designed to withstand the effects of natural phenomena such as earthquakes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

2. 10 CFR Part 50, Appendix S - IV, “Application to Engineered Design”, which requires that vibratory ground motion (including the Safe Shutdown Earthquake Ground Motion and the Operating Basis Earthquake Ground Motion) and surface deformation be considered in the design of a nuclear power plant.

To the extent applicable in the regulatory requirements cited above, and in accordance with RS-002, the staff applied NRC-endorsed methodologies and approaches (specified in Section 2.5.3 of NUREG-0800) for evaluation of information characterizing the potential for surface or near-surface tectonic and nontectonic deformation at the proposed site as recommended in RG 1.165.

Section 2.5.3 of NUREG-0800 and RG 1.165 provide specific guidance concerning the evaluation of information characterizing the potential for surface and near-surface deformation, including the geologic, seismic, and geophysical data that the applicant needs to provide to establish the potential for surface deformation.

2.5.3.3 Technical Evaluation

This SER section presents the staff’s evaluation of the geologic, seismic, and geophysical information submitted by the applicant in SSAR Section 2.5.3 to address the potential for surface or near-surface tectonic and nontectonic deformation within an 8 km (5 mi) radius of the ESP site (i.e., the “site area” as defined in RG 1.165). The technical information presented in SSAR Section 2.5.3 resulted from the applicant’s surface and subsurface geologic, seismic, and geophysical investigations performed within the site area, supplemented by aerial and field reconnaissance studies undertaken within a 40 km (25 mi) radius of the site (i.e., the “site vicinity” as defined in RG 1.165). Through its review, the staff determined whether the applicant complied with the applicable regulations and conducted its investigations with an appropriate level of detail in accordance with RG 1.165.

To thoroughly evaluate the geologic, seismic, and geophysical information presented by the applicant, the staff obtained the assistance of the USGS. The staff and its USGS advisors visited the ESP site to confirm interpretations, assumptions, and conclusions presented by the applicant and related to the potential for surface or near-surface faulting and nontectonic deformation.

2.5.3.3.1 Geologic, Seismic, and Geophysical Investigations

In SSAR Sections 2.5.3.1.1 through 2.5.3.1.7, the applicant reviewed and summarized information related to previous VEGP site investigations (Section 2.5.3.1.1), published geologic mapping (Section 2.5.3.1.2), previous SRS investigations (Section 2.5.3.1.3), previous seismicity data (Section 2.5.3.1.4), previous seismic reflection data (Section 2.5.3.1.5), current seismic reflection studies (Section 2.5.3.1.6), and current aerial and field reconnaissance (Section 2.5.3.1.7).

Based on the information presented in SSAR Sections 2.5.3.1.1 through 2.5.3.1.7, the applicant concluded that no capable tectonic sources occur within the site area and that there is negligible potential for surface or near-surface fault rupture. Consequently, the applicant considered the site to be suitable in regard to the potential for surface or near-surface faulting. The staff's review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.7 is presented below.

Data from Previous Investigations

The staff focused its review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.5 on the applicant's descriptions of previous studies and data collected within the site area in order to assess the potential for surface tectonic deformation at the ESP site. In SSAR Section 2.5.3.1.1, the applicant described the results of previous investigations conducted for VEGP Units 1 and 2, which support the concepts that the Pen Branch fault (known to underlie the ESP site) exhibits no surface displacement and is a noncapable tectonic structure and that nontectonic deformation features occur in the site area. In SSAR Section 2.5.3.1.2, the applicant discussed information from published geologic maps documenting the existence of nontectonic deformation features in the site area. SER Section 2.5.3.3.9 provides a more detailed discussion of nontectonic features in the site area. The applicant also stated in SSAR Section 2.5.3.1.2 that Crone and Wheeler (2000) and Wheeler (2005) classified the Pen Branch fault as a Class C feature based on insufficient geologic evidence to document Quaternary displacement along the fault. In SSAR Section 2.5.3.1.3, the applicant cited evidence collected from the SRS that the Pen Branch fault does not exhibit surface displacement, is not a capable tectonic structure, and is not favorably oriented in the modern-day stress field to experience displacement. In SSAR Section 2.5.3.1.4, the applicant stated that no recent earthquake activity has occurred within the site area based on microseismicity data. In SSAR Section 2.5.3.1.5, the applicant discussed previous seismic reflection studies supporting the interpretation that the Pen Branch fault is not a capable tectonic structure.

Based on a review of SSAR Sections 2.5.3.1.1 through 2.5.3.1.5, the staff concludes that the applicant presented thorough and accurate descriptions of previous studies and data collected within the site area. The applicant used this information to assess the potential for tectonic deformation at the ESP site, which is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c),

and 10 CFR 100.23(d). These five SSAR sections present well-documented geologic information that the applicant derived from published sources. The applicant provided an extensive list of references for these sources, which the staff examined in order to ensure the accuracy of the information presented by the applicant in the SSAR.

Data from Current Investigations

The staff focused its review of SSAR Sections 2.5.3.1.6 and 2.5.3.1.7 on the applicant's descriptions of the investigations performed to image the Pen Branch fault at the ESP site using seismic reflection and to look for evidence of surface faulting in the site vicinity using field and aerial reconnaissance. In SSAR Section 2.5.3.1.6, the applicant stated that the Pen Branch fault is clearly imaged beneath the ESP site in the seismic reflection data. In SSAR Section 2.5.3.1.7, the applicant indicated that no geomorphic evidence exists for surface rupture, surface warping, or fault offset. The applicant also reported its reinterpretation of features observed within the site vicinity and initially considered as possible evidence for tectonic activity. The applicant reinterpreted these features as nontectonic in origin.

Based on its review of SSAR Sections 2.5.3.1.6 and 2.5.3.1.7, the staff concludes that the applicant presented thorough and accurate descriptions of data from current investigations within the site area in order to assess the potential for tectonic deformation at the ESP site. This information supports the requirements set forth in 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff further concludes that the applicant presented adequate evidence to support the conclusions that the Pen Branch fault underlies the ESP site. The staff believes that the applicant also provided adequate evidence that no surface rupture due to displacement along the Pen Branch fault exists in the site area or site vicinity. SER Section 2.5.1.3.4 presents the staff's evaluations and conclusions regarding all new information that was collected by the applicant to assess the Pen Branch fault. This information was used to support the applicant's conclusions that the Pen Branch fault does not exhibit surface rupture or Quaternary (1.8 mya to present) displacement and is not a capable tectonic feature at the ESP site.

2.5.3.3.2 Geologic Evidence for Surface Deformation

In SSAR Section 2.5.3.2, the applicant described four bedrock faults identified within the site area. These structures include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults, which the applicant discussed in SSAR Sections 2.5.3.2.1, 2.5.3.2.2, 2.5.3.2.3, and 2.5.3.2.4, respectively. Based on information presented in SSAR Sections 2.5.3.2 and 2.5.1.2.4, the applicant concluded that none of the four faults mapped within the site area shows any evidence of surface rupture and that none of the faults is a capable tectonic source. The staff's evaluation of SSAR Section 2.5.3.2, including Sections 2.5.3.2.1, 2.5.3.2.2, 2.5.3.2.3, and 2.5.3.2.4, is presented below.

The staff focused its review of SSAR Section 2.5.3.2 on the applicant's descriptions of the four bedrock faults mapped within the site area. The staff concludes that the applicant presented accurate descriptions of these four faults to enable assessment of the potential for tectonic surface deformation within the site area. This assessment is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the

information presented by the applicant in SSAR Section 2.5.3.2, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant that none of these four faults exhibits surface displacement and none is a capable tectonic feature.

The rationale for the staff's conclusions in regard to the existence of surface faulting in the site vicinity and at the site, particularly in relation to the Pen Branch fault, is presented in detail in SER Section 2.5.1.3.4, which discusses geology of the site area. Also in SER section 2.5.1.3.4, the staff presents a summary of the lines of evidence cited by the applicant in the SSAR to indicate that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.3 Correlation of Earthquakes with Capable Tectonic Sources

In SSAR Section 2.5.3.3, the applicant described the distribution of epicenters for instrumentally recorded earthquakes that have occurred in the site vicinity (within an 8-km (5-mi) radius). The applicant stated that neither historical nor instrumentally recorded earthquake epicenters show a correlation with known or postulated faults in the site vicinity. Based on information presented in SSAR Section 2.5.3.3, as well as in SSAR Section 2.5.1.1.4.3 and SSAR Figure 2.5.1-16, the applicant concluded that no spatial correlation exists between earthquake epicenters and known or postulated faults in the site vicinity or site area. The staff's evaluation of SSAR Section 2.5.3.3 is presented below.

The staff focused its review of SSAR Section 2.5.3.3 on the applicant's description of historical and instrumentally recorded earthquake epicenters and faults that have occurred within the site vicinity. The staff concludes that the applicant presented convincing data and logical interpretations related to a lack of correlation between earthquakes and tectonic sources in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.3, as well as information presented by the applicant in SSAR Section 2.5.1.1.4.3 and SSAR Figure 2.5.1-16, the staff concurs with the applicant's conclusion that no spatial correlation exists between earthquake epicenters and faults in the site vicinity or site area.

2.5.3.3.4 Ages of Most Recent Deformations

In SSAR Section 2.5.3.4, the applicant discussed information related to ages of the most recent deformations indicated for the four bedrock faults identified within the site area (i.e., the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults). Based on information presented in SSAR Sections 2.5.3.4 and 2.5.1.2.4, the applicant concluded that none of these four faults exhibits Quaternary displacement and none is considered a capable tectonic structures. For the Pen Branch fault, the applicant stated that there is no evidence indicating this fault has experienced displacement younger than Eocene (i.e., less than 33.7 mya). The Pen Branch fault is of particular interest to the staff because it underlies the ESP site. The staff's evaluation of SSAR Section 2.5.3.4 is presented below.

The staff focused its review of SSAR Section 2.5.3.4 on the applicant's discussion of the ages of most recent deformations indicated for the four bedrock faults mapped within the site area. The

staff concludes that the applicant presented accurate descriptions of the ages of deformation for these four faults in order to enable an accurate assessment of Quaternary displacement along faults within the ESP site area and at the ESP site. This assessment is required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.4, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant's conclusion that none of these four faults exhibits Quaternary displacement.

The rationale for the staff's conclusions in regard to the ages of most recent deformation, specifically for the Pen Branch fault, is presented in detail in SER Section 2.5.1.3.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence used by the applicant in the SSAR indicating that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.5 Relationship of Site Area Tectonic Features to Regional Tectonic Structures

In SSAR Section 2.5.3.5, the applicant discussed the four faults identified within the site area. These structures include the Pen Branch, Ellenton, Steel Creek, and Upper Three Runs faults, which the applicant discussed in SSAR Sections 2.5.3.5.1, 2.5.3.5.2, 2.5.3.5.3, and 2.5.3.5.4, respectively. Of these four faults, the applicant indicated that only the Pen Branch fault occurs west of the SRS on the ESP site. Based on information presented in SSAR Section 2.5.3.5, the applicant concluded that none of the four faults is considered a capable tectonic feature within the site area, effectively concluding that none is linked with any capable regional tectonic structure. The staff's evaluation of SSAR Section 2.5.3.5 is presented below.

The staff focused its review of SSAR Section 2.5.3.5 on the applicant's descriptions of these four faults identified within the site area. The staff concludes that the applicant presented accurate descriptions of these four faults to enable assessment of possible linkage with regional tectonic structures in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.5, as well as information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the conclusions of the applicant that none of the four faults is a capable tectonic feature and none is linked with a capable regional tectonic structure.

2.5.3.3.6 Characterization of Capable Tectonic Sources

In SSAR Section 2.5.3.6, the applicant stated that no capable tectonic sources occur within the site area. The applicant summarized the data supporting a noncapable status for the Pen Branch fault. Based on information presented in SSAR Section 2.5.3.6, the applicant concluded that no capable tectonic sources exist in the site area that would require characterization. The staff's evaluation of SSAR Section 2.5.3.6 is presented below.

The staff focused its review of SSAR Section 2.5.3.6 on the applicant's description of the Pen Branch fault. The staff concludes that the applicant presented an accurate summary to enable assessment of the capability of the Pen Branch fault in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), and 10 CFR 100.23(c), 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Section 2.5.3.6, as well as

information discussed in SSAR Section 2.5.1.2.4, the staff concurs with the applicant's conclusion that no capable tectonic sources exist in the site area requiring characterization, including the Pen Branch fault.

The rationale for the staff's conclusions in regard to the noncapability of the Pen Branch fault is presented in detail in SER Section 2.5.1.2.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence used by the applicant in the SSAR indicating that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.7 Designation of Zones of Quaternary Deformation for Detailed Investigation

In SSAR Section 2.5.3.7, the applicant concluded that there are no zones of Quaternary deformation within the site area which require detailed investigation. The applicant based its conclusion on data presented in SSAR Sections 2.5.1.2.4, 2.5.3.2, 2.5.3.4, and 2.5.3.5. The staff's evaluation of SSAR Section 2.5.3.7 is presented below.

The staff focused its review of SSAR Section 2.5.3.7 on the applicant's descriptions of faults identified in the site area and discussed in SSAR Sections 2.5.1.2.4, 2.5.3.2, 2.5.3.4, and 2.5.3.5. The staff concludes that the applicant presented accurate descriptions of faults identified in the site area to enable an assessment of Quaternary deformation within the site area and at the ESP site in support of the ESP application and as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of this information, the staff concurs with the applicant's conclusion that there are no zones of Quaternary deformation within the site area that require a detailed investigation.

The rationale for the staff's conclusions in regard to a lack of Quaternary deformation in the site area is presented in detail in SER Section 2.5.1.3.4. Also in SER Section 2.5.1.3.4, the staff presents a summary of the lines of evidence cited by the applicant in the SSAR to indicate that the Pen Branch fault does not exhibit Quaternary displacement and is not a capable tectonic feature.

2.5.3.3.8 Potential for Surface Tectonic Deformation

In SSAR Section 2.5.3.8.1, the applicant stated that the Pen Branch fault is noncapable and will not cause surface rupture in the future. The applicant also stated that the nonbrittle folding of the Blue Bluff Marl, interpreted to result from displacement along the Pen Branch fault, indicates near-surface tectonic deformation that is not younger than Eocene (i.e., less than 33.7 mya). Based on information summarized in SSAR Section 2.5.3.8.1, which is discussed in more detail by the applicant in SSAR Section 2.5.1.2.4.2, the applicant concluded that the potential for tectonic deformation at the site is negligible. The staff's evaluation of SSAR Section 2.5.3.8.1 is presented below.

The staff focused its review of SSAR Section 2.5.3.8.1 on the applicant's discussion of near-surface tectonic deformation interpreted by the applicant to result from displacement along the Pen Branch fault more than 33.7 mya. The staff concludes that the applicant presented an accurate discussion of the field data indicating no displacement younger than Eocene along the Pen Branch fault in the site area. This assessment is required by 10 CFR 52.17(a)(1)(vi),

10 CFR 100.23(c), and 10 CFR 100.23(d). Based on a review of the information presented by the applicant in SSAR Sections 2.5.3.8.1 and 2.5.1.2.4.2, the staff concurs with the conclusion of the applicant that the potential for tectonic deformation at the site is negligible.

2.5.3.3.9 Potential for Nontectonic Deformation

In SSAR Section 2.5.3.8.2, the applicant discussed dissolution collapse features (SSAR Section 2.5.3.8.2.1) and "clastic" dikes (SSAR Section 2.5.3.8.2.2). Based on information presented in SSAR Section 2.5.3.8.2.1, the applicant stated that dissolution collapse features are not considered to be tectonic structures or paleoseismic features, and concluded that they do not represent a safety issue for the ESP site in regard to nontectonic surface deformation. Based on information presented in SSAR Section 2.5.3.8.2.2, the applicant indicated that two types of so-called "clastic" dikes occur in the site area: (1) sand dikes that resulted from injection of poorly-consolidated, liquefied fine sand into overlying sediments; and (2) pedogenic clastic dikes related to weathering and soil formation (i.e., pedogenic) processes that were enhanced along fractures. The applicant stated that these two types of dikes are also not tectonic structures or paleoseismic features and likewise concluded that they do not represent a safety issue for the ESP site in regard to nontectonic surface deformation. The staff's evaluation of SSAR Section 2.5.3.8.2 is presented below.

The staff focused its review of SSAR Section 2.5.3.8.2 on the applicant's descriptions of the modes of formation of the dissolution collapse features and "clastic" dikes (i.e., both the injection type and the pedogenic clastic type) because the applicant used this descriptive information to conclude that these features resulted from nontectonic deformation. The applicant also referred to "small-scale deformation features" in SSAR Sections 2.5.3.1.2 and 2.5.3.1.7, considered by McDowell and Houser (1983) and Bartholomew et al. (2002) to be possible evidence of tectonic activity. The applicant stated in SSAR Sections 2.5.3.1.2, 2.5.3.1.7, and 2.5.3.8.2.2 that these small-scale features are considered to be nontectonic in origin based on observations made by the applicant during field reconnaissance studies performed for the ESP application. However, the applicant did not fully discuss the field observations and reasoning used to conclude that these small-scale deformation features are nontectonic in origin, and did not provide adequate information about the origin of the injection sand dikes or the pedogenic clastic dikes.

In RAI 2.5.3-1, the staff asked the applicant to more clearly describe its logic for concluding that the deformation features mapped and described by McDowell and Houser (1983) and Bartholomew et al. (2002) are nontectonic in origin. In RAI 2.5.3-2, the staff asked the applicant for additional information on field data used by the applicant to conclude that both the injection sand dikes and the pedogenic clastic dikes are nontectonic in origin. This clarification is important because paleoliquefaction features related to the 1886 Charleston earthquake or other previous seismic events are known to occur in the region, and the staff must ensure that none of the features described by the applicant in SSAR Sections 2.5.3.1.2, 2.5.3.1.7, and 2.5.3.8.2.2 are related to Quaternary tectonic deformation.

In response to RAI 2.5.3-1, the applicant stated that, based on reconnaissance of exposures in the site area, certain primary characteristics of the pedogenic type of clastic dikes suggested an origin consistent with weathering and soil forming processes for these features. Specifically, (1) the dikes are widely distributed in deeply weathered clayey and silty sands of the Hawthorne

Formation and the Barnwell Group formations; (2) the dikes occur in nearly all exposures of the weathered profile, but are generally absent in exposures of stratigraphically lower, less weathered sedimentary units; (3) the dikes contain a central zone of bleached host rock bounded by a cemented zone of iron oxide and may contain a clay core; (4) grain-size analyses indicate that the dikes contain the same grain-size distribution as the host sediment, but with more silt and clay; and (5) the dikes decrease downward in width and density, usually tapering and pinching out over a distance of 5 to 15 feet. The applicant indicated that the "clastic" dikes identified by Bartholomew et al. (2002) are syndepositional, as indicated by the presence of marine animal burrows crossing the dikes, and that they developed in a subaqueous marine environment during the Late Eocene (i.e., more than 33.7 mya). Based on these lines of evidence, the applicant concluded that the clastic dikes observed in the site area are pedogenic, and not tectonic, in origin. The applicant also concluded that the clastic dikes described by Bartholomew et al. (2002), whether their origin is tectonic or nontectonic, developed more than 33.7 mya.

Based on its review of the applicant's response to RAI 2.5.3-1, the staff concurs with the applicant's conclusion that the clastic dikes described by Bartholomew et al. (2002) are older than 33.7 mya. The staff further concludes, in agreement with the applicant, that the clastic dikes observed in the site area are the result of pedogenic processes and are nontectonic in origin.

In response to RAI 2.5.3-2, the applicant indicated that the deformation features (i.e., warped bedding, fractures, small-scale faults, injection sand dikes, and clastic dikes), interpreted by the applicant to be nontectonic in origin, occurred in a garbage trench on the VEGP site mapped by the Bechtel staff in 1984. The trench (now filled but illustrated in SSAR Figures 2.5.3-1 and 2.5.3-2, as well as in Figure 2.5.3-2A accompanying the applicant's RAI response) contained a monocline in the Blue Bluff Marl that is interpreted by the applicant as related to Eocene displacement along the Pen Branch fault. The monocline is positioned above the subsurface line of intersection of the Pen Branch fault with the contact of basement rock and Coastal Plain sediments.

In response to RAI 2.5.3-2, the applicant also stated that the local spatial relationships of warped bedding, fractures, and small-scale faults with the margins of dissolution depressions clearly demonstrate a nontectonic, dissolution collapse origin for these features. The applicant cited the trench map produced by Bechtel (1984), illustrated in Figure 2.5.3-2A, which accompanied its response to RAI 2.5.3-2, as conclusive evidence for this statement. The applicant reiterated the five primary characteristics of clastic dikes presented in its response to RAI 2.5.3-1, which suggested an origin consistent with a pedogenic origin for these features. In response to RAI 2.5.3-2, the applicant further indicated that the injection sand dikes likely were formed by fluid or plastic injection of an underlying source sand and that the close spatial association of the injection dikes with the sides of dissolution collapse depressions suggests that this type of dike is also related to a nontectonic, dissolution collapse origin. The applicant also stated that the injection sand dikes likely formed prior to an erosional event that occurred at the end of the Miocene (i.e., more than 5.3 mya), but did not discuss the basis for this statement in detail in the RAI response. The applicant stated that clastic dikes developed during a weathering event that is older than Late Pleistocene (i.e., more than 10,000 years ago).

Based on its review of the applicant's response to RAI 2.5.3-1, the staff concurs with the applicant that the clastic dikes described by Bartholomew et al. (2002) are older than 33.7 mya. The staff further concludes, in agreement with the applicant, that the clastic dikes observed in the site area are the result of pedogenic processes and are nontectonic in origin. Based on its review of the applicant's response to RAI 2.5.3-2, the staff concludes that the response qualifies timing of the development of warped bedding, fractures, small-scale faults, clastic dikes, and injection sand dikes. The timing of that development as suggested by information presented by the applicant is as follows:

1. Deposition of Tertiary (i.e., a range of 65 to 1.8 mya in age) sedimentary units, including at least Eocene (54.8 to 33.7 mya) and Miocene (23.8 to 5.3 mya) sediments, with some periods of subaerial (i.e., above water in open air) erosion.

2. Initiation of dissolution of the Utley Limestone (Late Eocene in age) at the base of the Eocene Barnwell Group, with development of incipient depressions and formation of injected sand dikes in Barnwell Unit "D" above the Utley Limestone as illustrated in Figure 2.5.3-2A of the applicant's response to RAI 2.5.3-2. The initiation of dissolution and development of the injected sand dikes occurred after deposition of the sedimentary units in which they are found, and the applicant reported Late Pleistocene (more than 10,000 years in age) to Holocene (less than 10,000 years in age) sands that do not appear to be deformed overlying the warped bedding, fractures, small-scale faults, clastic dikes, and injection sand dikes in the trench mapped by Bechtel (1984).

3. Continued and increasing dissolution of the Utley Limestone, with numerous nontectonic dissolution collapse features developed in overlying units, including collapse-generated faults that cut, and consequently postdate, the injected sand dikes. Consequently, the injected sand dikes are the oldest of the deformation features mapped that the applicant equated with a response to nontectonic near-surface deformation.

4. Development of nontectonic clastic dikes above the sedimentary units that experienced dissolution collapse, many in the Miocene-age Hawthorne Formation based on Figure 2.5.3-2A of the applicant's response to RAI 2.5.3-2. The clastic dikes do not extend into Late Pleistocene to Holocene-age sands, indicating that the clastic dikes are at least 10,000 years old.

The staff concludes that the evidence presented by the applicant in the response to RAI 2.5.3-2 clearly documents a nontectonic origin for the warped bedding, fractures, small-scale faults, and clastic dikes.

In regard to the origin of the injection sand dikes, the applicant made the case that these features are the oldest structures generated by nontectonic deformation in the site area. That is, the applicant considered that the injection sand dikes are not related to paleoliquifaction resulting from Quaternary tectonic deformation and seismic shaking in the site area. From information presented by the applicant in the SSAR and its response to RAI 2.5.3-2, the staff concludes that the injection sand dikes are the oldest of the observed features, and the age constraints discussed by the applicant appear to limit the youngest timing for development of these features to earlier than Late Pleistocene (i.e., more than 10,000 years in age) and

possibly Pliocene (5.3 to 1.8 mya). This upper age limit for the injection sand dikes is supported by information provided by the applicant in the response to RAI 2.5.3-2, suggesting that the dikes pre-date an erosional event at or near the end of the Miocene (23.8 to 5.3 mya). Consequently, even if the injection sand dikes were the result of seismically-induced paleoliquefaction, the features are not Holocene (10,000 years to present) in age. However, a Pleistocene age (1.8 mya to 10,000 years) is not precluded for the injection sand dikes based on information provided by the applicant in the response to RAI 2.5.3-2.

The staff concurs with the applicant that no evidence exists to indicate that any of these features represent a safety issue for the ESP site in regard to nontectonic surface or near-surface deformation. However, in developing the SER with open items, the staff considered that the applicant's response to RAI 2.5.3-2 in regard to the injection sand dikes did not provide adequate information to bracket the pre-Miocene upper age limit for development of this feature as suggested by the applicant. Furthermore, the staff considered that the applicant did not clearly show that the injection sand dikes are spatially related to what must have been incipient dissolution depressions (i.e., much of the dissolution must have occurred after development of the injection sand dikes since, as the applicant pointed out, nontectonic small-scale faults associated with dissolution collapse cross-cut the injection dikes). Since the mechanism described by the applicant as responsible for the sand injection (i.e., fluid or plastic injection of the liquefied source sand) could be associated with seismic shaking and liquefaction of the sand materials, the staff formulated Open Item 2.5-10 to request that the applicant provide a more detailed description of geometry and physical characteristics of the injection sand dikes and their spatial association with dissolution depressions. The applicant's response and the staff's evaluation in regard to this open item are presented below.

In response to Open Item 2.5-10, the applicant cited all available field evidence used to interpret the injection sand dikes as nontectonic in origin (i.e., unrelated to seismic shaking and resultant liquefaction of materials) and pre-Quaternary in age. The applicant presented the following field evidence and logic to support its conclusions in regard to the injection sand dikes:

1. All injection sand dikes were found at a single location at the site and occurred within stratigraphic horizon "Unit D" of the Upper Eocene (more than 33.7 mya) Barnwell Group in the Coastal Plain sedimentary sequence.

2. The dikes registered upward movement of liquefied sands from a sand source in stratigraphic Unit C of the Barnwell Group, which directly underlies Unit D. The dikes, which penetrated and were confined to Unit D, clearly flattened along the base of Barnwell stratigraphic Unit E, which directly overlies Unit D. Since Units C, D, and E are Upper Eocene Barnwell Group stratigraphic horizons that sequentially overlie each other from C to E, all units involved are older than 33.7 mya.

3. The injection sand dikes appear to be spatially related to areas of localized dissolution at depth in the Utley Limestone, as shown by location of the sand dikes in relation to surface morphology of Unit F (Upper Eocene Barnwell Group) in Figure 2.5-10B which accompanied the applicant's response to Open Item 2.5-10. The surface of Unit F clearly reflects a dissolution-related morphology of generally circular to elongated

depressions due to the collapse of overlying sediments as dissolution of the underlying Utley Limestone occurred.

4. Based on the three field observations stated above, the applicant proposed a sand injection mechanism related to the response of sands in Unit C to increased overburden pressure associated with an early phase of collapse of sedimentary units overlying dissolution depressions in the Utley Limestone.

5. The Hawthorne Formation (Miocene, 23.8 to 5.3 mya) is the youngest unit showing effects of dissolution at depth (i.e., the "dissolution-related morphology" described above in Item 3). An erosion surface/relict paleosol (i.e., an earlier soil horizon that has persisted without major alteration of its morphology) overlying the Hawthorne does not show these effects. The applicant interpreted the erosion surface/paleosol to be Late Miocene to Pliocene in age (i.e., Late Tertiary, more than 1.8 mya, and therefore pre-Quaternary).

6. The erosion surface/paleosol is in turn overlain by Pleistocene-Holocene (less than 1.8 mya) eolian sands, which the applicant also reported showed no morphological effects of dissolution at depth.

7. Based on stratigraphic ages of units reflecting the dissolution-related morphology, the applicant interpreted the dissolution to be no younger than Late Miocene-Pliocene (i.e., more than 1.8 mya). By association, the injected sand dikes are also interpreted by the applicant to be no younger than Late Miocene-Pliocene.

The staff considers that the applicant used all available field evidence as cited above to conclude that the injected sand dikes formed in response to movement of liquefied sands resulting from collapse of overlying sediments related to dissolution at depth, rather than in response to liquefaction of saturated sands resulting from seismic shaking, and are most likely no younger than Late Miocene-Pliocene (i.e., more than 1.8 mya, so pre-Quaternary). Although the staff was not able to examine the injected sand dikes in the field because the trench in which they occurred is now filled, the applicant did show that the dikes are spatially related to areas of localized dissolution at depth in the Utley Limestone. Furthermore, the dikes are wholly confined to Upper Eocene sediments that are older than 33.7 mya, and it is not likely that such features would have been produced in units this old by historical seismicity and associated liquefaction. The applicant used stratigraphic constraints to suggest relative timing of dike formation (i.e., the applicant presented relative ages, rather than absolute age dates derived from radiometric dating methods). Use of stratigraphic data to determine relative age of a geologic feature is a standard method that is often applied when radiometric age dates are not available, and staff agrees that use of this method is appropriate in this case. In light of the information presented in the applicant's detailed response to Open Item 2.5-10, the staff agrees with the conclusions drawn by the applicant that the injection sand dikes are nontectonic in nature and pre-Quaternary in age. Therefore, Open Item 2.5-10 is resolved.

Based on a review of information presented by the applicant in SSAR Section 2.5.3.8.2 and the responses to RAI 2.5.3-1, RAI 2.5.3-2, and Open Item 2.5-10, the staff concurs with the applicant's conclusion that warped bedding, fractures, small-scale faults, clastic dikes, and

injection sand dikes represent nontectonic deformation. The staff concludes that the applicant presented thorough descriptions of these features to enable assessment of nontectonic surface or near-surface deformation within the site area and at the ESP site in support of the ESP application as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). Based on review of SSAR Section 2.5.3 and the applicant's responses to RAIs and Open Item 2.5-10 as set forth above, the staff concludes that the applicant properly characterized the potential for surface and near-surface tectonic and nontectonic deformation at the ESP site, including the possibility of Quaternary tectonic deformation along the Pen Branch fault. The staff also concludes that SSAR Section 2.5.3 provides accurate and thorough descriptions of the potential for surface and near-surface tectonic and nontectonic deformation at the ESP site, with emphasis on the Quaternary Period, as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d).

2.5.3.4 Conclusions

As set forth in SER Sections 2.5.3.1, 2.5.3.2, and 2.5.3.3, the staff carefully reviewed the information on surface faulting submitted by the applicant in SSAR Section 2.5.3. On the basis of its detailed review, as fully described in the above SER sections, the staff concludes that the applicant provided a thorough and accurate characterization of surface and near-surface faulting and nontectonic deformation at the site as required by 10 CFR 52.17(a)(1)(vi), 10 CFR 100.23(c), and 10 CFR 100.23(d). The staff concurs that data and analyses presented by the applicant in the SSAR provide an adequate basis to conclude that there is no evidence to indicate that surface or near-surface faulting or nontectonic deformation presents a hazard for the site area.

Based on information from the applicant's thorough review of the literature on site area geology in regard to surface expression of faulting, and the applicant's literature review and geologic, seismic, and geophysical investigations of the site vicinity and site area, the staff further concludes that the applicant has properly characterized the potential for surface or near-surface faulting and nontectonic deformation at the ESP site.available, and staff agrees that use of this method is appropriate in this case. In light of the information presented in the applicant's detailed response to Open Item 2.5-10, the staff agrees with the conclusions drawn by the applicant that the injection sand dikes are nontectonic in nature and pre-Quaternary in age. Therefore, Open Item 2.5-10 is resolved.

2.5.4 Stability of Subsurface Materials and Foundations

Section 2.5.4 of this SER evaluates the stability of subsurface materials and foundations at the site of Vogtle Electric Generating Plant (VEGP) Units 3 and 4. Section 2.5.4.1 of this SER provides a summary of the relevant geologic and seismic information contained in Section 2.5.4 of the Site Safety Analysis Report (SSAR) of the VEGP Units 3 and 4 Early Site Permit (ESP) application and LWA request. SER Section 2.5.4.3 provides the staff's evaluation of SSAR Section 2.5.4, including with respect to the applicant's responses to any requests for additional information, the resolution of open items, and the results of confirmatory analyses performed by the staff. SER Section 2.5.4.4 summarizes the applicant's conclusions as well as the staff's conclusions, and confirms that the applicable regulations have been met by the applicant.

2.5.4.1 Summary of Application

With respect to the stability of subsurface materials and foundations, the SSAR addresses information items contained in the AP1000 Standard Plant Design, Design Control Document (DCD), Revision 15. The applicant developed geological, geophysical, geotechnical, and seismological information to be used as the basis for the evaluation of the stability of the subsurface materials and foundations at the proposed site. The applicant initially reviewed analyses and reports prepared for the existing VEGP Units 1 and 2 as well as the readily available geotechnical literature. The applicant then conducted field investigations and performed field and laboratory testing during the initial phase of the ESP site subsurface investigation. These subsequent investigations were conducted with the intent of obtaining additional site information to further the understanding of the VEGP site and to complement the existing geotechnical data from the previous investigations completed for VEGP Units 1 and 2.

The applicant augmented the ESP field and laboratory test data with field and laboratory data from an investigation it performed in support of a Limited Work Authorization (LWA) request, which the applicant submitted on August 16, 2007. In addition to performing this investigation to support the LWA request, the applicant conducted comprehensive site geotechnical field and laboratory investigations to enhance the existing ESP geotechnical data as well as to support the COL application that the applicant submitted to the NRC on March 31, 2008. This additional data allowed the applicant to further develop and understand the geotechnical data at the specific locations proposed for the VEGP Units 3 and 4 site structures and at the locations of the proposed borrow sources for the structural backfill materials. Because the staff determined that this additional information was necessary only to the staff's finding associated with the LWA request, the staff has summarized and evaluated these additional data and analyses separately from the ESP information in this section and Section 2.5.4.3, respectively. Finally, the applicant conducted a test pad program in support of the LWA request to establish site-specific design properties for the structural backfill and to verify that the proposed backfill materials would meet the AP1000 standard design siting criteria.

2.5.4.1.1 Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Sections 2.5.1.1 and 2.5.1.2 for detailed descriptions of the regional and site geology, including structural geology, physiography, geomorphology, geologic history, stratigraphy, structures, and hazards.

2.5.4.1.2 Properties of Subsurface Materials

SSAR Section 2.5.4.2 describes the static and dynamic engineering properties of the subsurface materials at the ESP site. In this section, the applicant described the subsurface materials, field investigations, laboratory tests, and the engineering properties it determined for the subsurface materials. The applicant also described the ESP and COL investigations and results for each stratum.

In support of the ESP application, the applicant submitted the following information:

Description of Subsurface Materials

SSAR Section 2.5.4.2.2 provides an overview of the subsurface profile and materials, including detailed descriptions of the underlying strata. The applicant categorized the soils underlying the ESP site into three groups based on their stability for geotechnical purposes. Group 1 soils include sands with silt and clay, Group 2 is the Blue Bluff clay marl layer, and Group 3 is made up of coarse-to-fine sand with interbedded thin seams of silt or clay. The applicant stated that the Group 1 soils would be completely removed and replaced with compacted backfill prior to construction of VEGP Units 3 and 4. In addition to grouping the soils, the applicant divided the VEGP site soils and bedrock into five strata:

1. Upper Sand Stratum (Group 1: Barnwell Group)
2. Marl Bearing Stratum (Group 2: Blue Bluff Marl or Lisbon Formation)
3. Lower Sand Stratum (Group 3)
4. Dunbarton Triassic Basin Bedrock
5. Paleozoic Crystalline Bedrock

The applicant developed the static design and engineering properties of the five strata from field and laboratory tests that it performed during the ESP and COL subsurface investigations, the results of which are summarized in Table 2.5.4-1 of this SSAR. A brief description of each stratum is provided below, including the soil and rock constituents and their ranges of thickness at the site. The applicant determined this information from 14 borings and 10 cone penetrometer tests (CPT) that it performed during the ESP subsurface investigation, and from 70 borings and 8 CPTs performed during the COL investigation. SSAR Figure 2.5.4-1a (Figure 2.5.4-1 of this SER) shows the locations of most of the ESP and COL borings. The applicant also provided cross-sectional profiles of subsurface conditions across the site and the nuclear island (SSAR Figures 2.5.4-3 through 2.5.4-5b; Figure 2.5.4-2 of this SER).

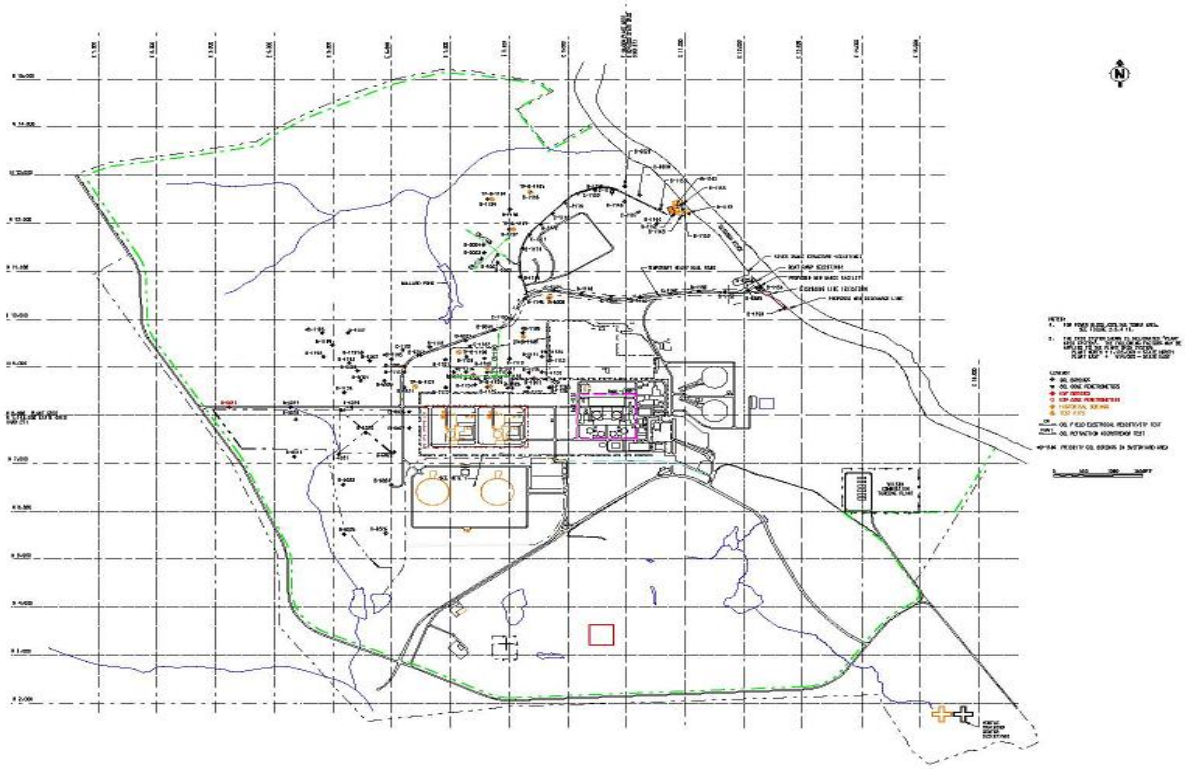


Figure 2.5.4-1 COL Site Boring Plan, Including Locations of ESP and Units 1 and 2 Borings (SSAR Figure 2.5.4-1a)

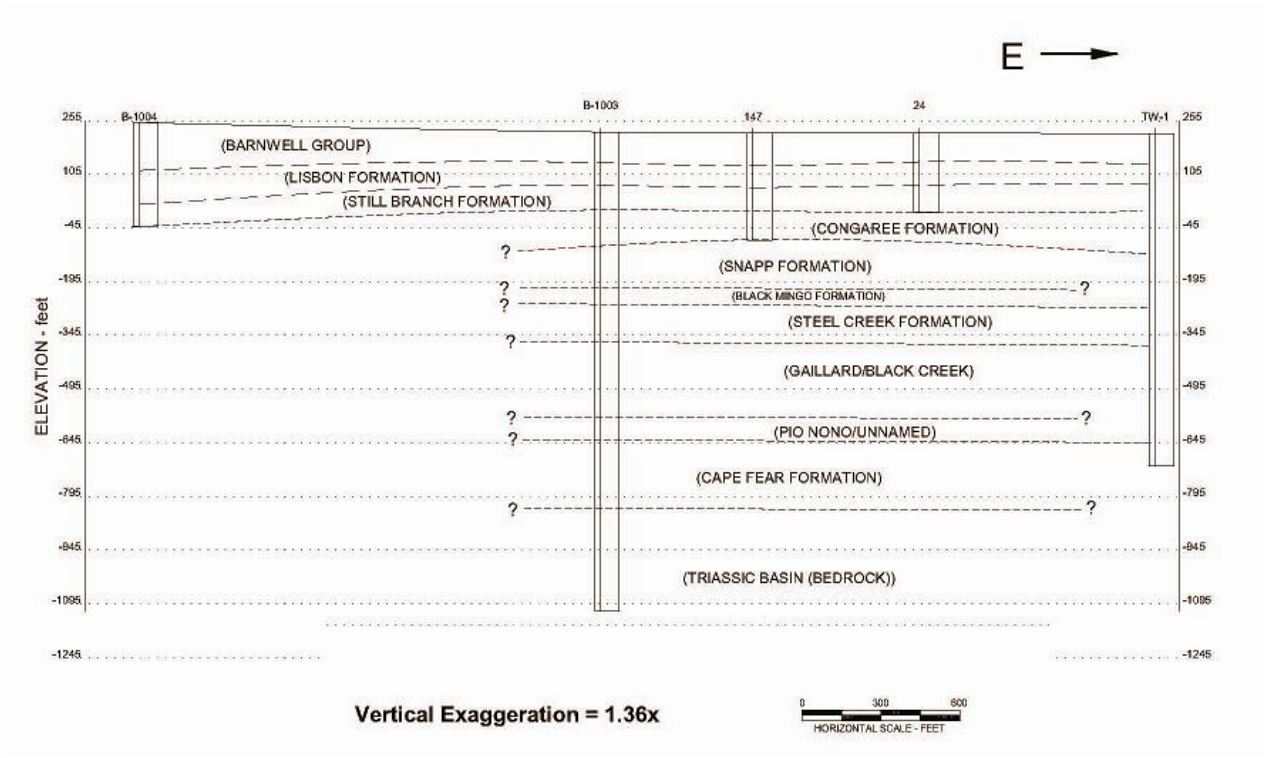


Figure 2.5.4-2 Subsurface Profile B-B' from SSAR Figure 2.5.4-1 (SSAR Figure 2.5.4-4)

1. Upper Sand Stratum (Barnwell Group). SSAR Subsection 2.5.4.2.2.1 describes the Upper Sand Stratum, or Barnwell Group, as consisting of predominantly sands, silty sands, and clayey sands with occasional clay seams, soft zones, and shell zones. The applicant encountered a shelly limestone layer, the Utley limestone, which contains significant solution channels, cracks, and other discontinuities, and observed severe fluid loss in the stratum while drilling. The applicant also determined that the stratum ranged in thickness from 24 to 48 meters (m) (78 to 157 feet (ft)), and attributed the large range to the westerly to northwesterly dip of the underlying Blue Bluff Marl. Based on its review of previous investigations for Units 1 and 2, the applicant determined that the Upper Sand Stratum is susceptible to liquefaction during seismic ground motion equivalent to the safe shutdown earthquake (SSE). The applicant found that the relative density of the stratum is highly variable, ranging from very loose to dense with clay lenses within the stratum ranging from soft to medium stiff. Therefore, the applicant concluded that the entirety of the Upper Sand Stratum, including the limestone layer, would need to be completely removed before it begins construction for VEGP Units 3 and 4.

   The applicant performed field Standard Penetration Tests (SPT) within the Upper Sand Stratum and obtained very high blow count values indicative of the previously observed shelly limestone and shell hash (mixture or pieces of shell) zones. Samples were recovered by the applicant at varying depths within the stratum and submitted for laboratory testing, including percent fines, moisture content, and Atterberg Limits (a measure of the relationship between percentage of fines and water content that affects the ability of a soil to remain plastic). The applicant indicated that the test results for percent fines ranged from 3 to 60 percent and 5 to 96 percent for the ESP and COL investigations, respectively, suggesting the stratum was made up of very fine grained sands, silts, and clay particles. From the results of the Atterberg Limits tests, the applicant determined a liquid limit of 43 to 97 for ESP investigations and an average of 72 for COL investigations. The applicant also determined a range of plasticity index from 21 to 67 for ESP investigations and an average index of 39 for COL investigations, indicating that the stratum's materials were inorganic and organic silts and clays of high plasticity. The natural moisture content of samples the applicant tested for Atterberg Limits ranged from 20 to 93 percent for the ESP investigations and again indicated the highly variable and fine grained nature of the sand, silt, and clay materials. The applicant calculated moist unit weights from 1,505 to 1,986 kilograms per cubic meter (kg/m^3; 94 to 124 pounds per cubic feet (pcf)) for fifteen samples, and specific gravities of 2.7 and 2.8 for two samples.

2. Blue Bluff Marl (Lisbon Formation). SSAR Subsection 2.5.4.2.2.2 describes the Blue Bluff Marl, which underlies the Upper Sand Stratum, in much greater detail because it is the load-bearing stratum at the proposed site of VEGP Units 3 and 4. The applicant stated that the Blue Bluff Marl consists of hard, slightly sandy, cemented, overconsolidated, calcareous clay with some shells and partially cemented, well-hardened layers varying between 19 to 29 m (63 to 95 ft) in thickness, with an average thickness of 23 m (76 ft) and a design ground water level at a depth of 16.7 m (55 ft). The top of the Blue Bluff Marl was mapped by the applicant between Elevation 37 and 42 m (122 and 140 ft) dipping downward towards the west side of the VEGP site. The applicant relied on 70 soil borings as part of its COL subsurface investigations to confirm its earlier ESP investigations of the Blue Bluff Marl. This reliance is especially important in the immediate area of the nuclear island, where 42 of

the applicant's 70 borings penetrated into the Blue Bluff Marl layer. The applicant also considered the previous investigations completed for Units 1 and 2 to further determine the subsurface properties of the Blue Bluff Marl.

The applicant conducted a series of standard penetration tests (SPTs) within the marl layer at the VEGP site. The results of SPTs are reported as the total blows summed over the distance to give blows per meter (or per foot), a measure commonly referred to as the N-value. The average N-values from the SPTs conducted as part of the ESP and COL investigations were high, 272 and 233 blows per meter (bpm) (83 and 71 blows per foot (bpf)), respectively, which the applicant attributed to the hard to very hard consistency of the fossiliferous limestone, and cemented layers and nodules of the marl. As expected, the applicant noted that the SPT N-values increased with depth. Finally, although the applicant noted the presence of soft zones (N-values below 16 bpm (5 bpf)) in the marl at the adjacent Savannah River Site (SRS), none of the SPTs conducted on the marl underlying the VEGP site yielded N-values less than 30.48 bpm (10 bpf): therefore, the applicant concluded that soft zones were not present in the marl beneath the site of VEGP Units 3 and 4.

The applicant recovered samples from within the Blue Bluff Marl during the ESP and COL subsurface investigations and submitted these samples for laboratory testing of percent fines, moisture content, and Atterberg Limits. SSAR Tables 2.5.4-1 thru 2.5.4-4 provide a summary of these laboratory tests. The applicant also provided the average values from both the ESP and COL laboratory tests, which included: 48 and 74 percent fines; plastic limits of 29 and 34 percent; liquid limits of 51 and 67 percent; and a Plasticity Index of 22 and 33 percent, respectively. The natural moisture content of the samples the applicant tested ranged from 14 to 67 percent and 14 to 62 percent for the ESP and COL investigations, respectively, with an average of 35 percent for the ESP investigations and 33 percent for the COL investigations. The applicant also calculated moist unit weights from 1,521 to 2,130 kg/m^3 (95 to 133 pcf) for 69 COL samples, and specific gravities of 2.61 and 2.66 for eight COL samples.

As part of its ESP investigations, the applicant also performed 15 one-point unconsolidated undrained triaxial shear tests on marl stratum samples. From these tests the applicant found that the undrained shear strength of the marl ranged from 7 to 205 kilopascals (kPa) (150 to 4,300 pounds per square foot (psf)), far lower than the undrained shear strength measured by Southern for Units 1 and 2, which was between 12.5 and 23,900 kPa (260 to 500,000 psf). The applicant stated that the disagreement between the two results stems from "severe sample disturbance due to sampling technique (pitcher sampler) and preparation of testing specimen." During the COL investigation, the applicant performed several additional laboratory strength tests on relatively undisturbed marl stratum samples. Specifically, these tests included 27 unconfined compression, 11 UU triaxial, and 27 consolidated undrained (CU) triaxial tests. The applicant reported that the average undrained shear strength from the UU and CU tests was 564 kPa (11,800 psf), which supported the design value of 478 kPa (10,000 psf) obtained for Units 1 and 2.

The applicant monitored the average heave during excavation for Units 1 and 2 and observed an average heave of 3.75 cm (1.25 in.), which corresponded to an undrained Young's modulus value of 478,000 kPa (10,000,000 psf). Using the average value of shear

strength results that failed at 2,394 kPa (50,000 psf), which was 766 kPa (16,000 psf), the applicant used the ratio of undrained shear strength to effective overburden pressure to calculate the preconsolidation pressure of 3,830 kPa (80,000 psf) and the overconsolidation ratio of 8. Due to this high preconsolidation pressure and the small foundation settlements measured by Southern during its VEGP Units 1 and 2 settlement monitoring program (less than 9.14 cm (3.6 in.), the applicant concluded that settlements due to new structures would be small. The applicant also measured the in-situ shear wave velocity which was used to calculate the dynamic shear modulus.

3. Lower Sand Stratum. SSAR Subsection 2.5.4.2.2.3 describes the Lower Sand Stratum, the top of which was mapped at a depth of about 50 m (165 ft) below the ground surface beneath the Blue Bluff Marl and underlain by the Dunbarton Triassic Basin rock. The applicant described the units of the stratum collectively as fine to coarse sands with interbedded silty clay and clayey silt, which, from top to bottom were identified as the Still Branch, Congaree, Snapp, Black Mingo, Steel Creek, Gaillard/Black Creek, Pio Nono/Unnamed, and Cape Fear formations. From the ESP subsurface investigations, the applicant determined that the Lower Sand Stratum was 275 m (900 ft) thick at the location of the one borehole (B-1003) that fully penetrated the stratum. Figure 2.5.4-4 of the SSAR illustrates the typical depths of the stratum as observed in B-1003.

   The applicant performed field SPTs during the ESP investigations and obtained an average N-value of 194 bpm (59 bpf). During the COL investigations, the applicant obtained SPT N-values for the Lower Sand in 42 penetrations as deep as 80 m (263 ft) into the unit, which averaged 230 bpm (70 bpf). The applicant observed that for the COL N-values, nearly all were above 98 bpm (30 bpf), indicative of very dense material. Furthermore, as was expected, both the ESP and COL investigation SPT N-values increased with depth. The applicant noted that the only evidence suggesting the presence of soft zones or loose material, a low N-value and lack of sample recovery, was an anomalous condition attributable to disturbed soil conditions at the bottom of the borehole caused by an imbalance between borehole and in-situ hydrostatic pressures.

   During the course of both the ESP and COL investigations, the applicant selected and submitted samples recovered from within the stratum for laboratory testing. The test results for percent fines and Atterberg Limits can be found in SSAR Table 2.5.4-1. The applicant reported that percent fines averaged 23.6 and 23 percent for the ESP and COL investigations, respectively. Atterberg Limit tests were performed as part of the ESP investigation and resulted in an average liquid limit percent of 47 percent, a plastic limit of 30 percent, a moisture content of 30 percent, and an average Plasticity Index of 17 percent. The applicant determined that samples with the higher percent fines and plasticity were from the silty clay and clayey silt layers. As part of the COL investigation, the applicant determined the moist unit weight of sixteen samples ranged from 1,810 to 2,178 kg/m³ (113 to 136 pcf), with an average specific gravity of 2.67 for four samples.

4. Dunbarton Triassic Basin Rock. SSAR Subsection 2.5.4.2.2.4 describes the Dunbarton Triassic Basin rock as red sandstone, breccia, and mudstone, weathered through the upper 37 m (120 ft). The applicant drilled only one borehole deep enough to encounter the Dunbarton during the ESP investigation, B-1003. The applicant measured shear wave

velocity in the upper 84 m (274 ft) of the rock profile and used the results to develop the shear wave velocity profile for site amplification. Finally, the applicant concluded that the rock was too deep to be of any interest to foundation design, and therefore performed no laboratory tests.

5. Paleozoic Crystalline Rock. SSAR Subsection 2.5.4.2.2.5 states that at a depth of 320 m (1,049 ft) below the surface, the applicant encountered the top portion of the weathered Dunbarton Triassic Basin rock. Beneath the adjacent SRS, the southeast dipping non-capable Pen Branch fault separates the Dunbarton Triassic Basin rock from Paleozoic crystalline rock to the northwest, a relationship the applicant suggested may occur at some depth below the VEGP site as well. According to the applicant, the results of a seismic reflection survey at the VEGP site supported the continuation of the Pen Branch fault beneath the VEGP site, and therefore the presence of Paleozoic crystalline rock as well.

6. Subsurface Profiles. SSAR Figures 2.5.4-3, -4, and -5 present the typical subsurface profiles across the powerblock areas as determined from the ESP borings. The applicant presented the subsurface profiles across the powerblock area based on the COL borings in SSAR Figures 2.5.4-3a, -4a, and -5a.

Field Investigations

The applicant presented its field and subsurface investigation programs in SSAR Section 2.5.4.2.3. While the locations of borings completed for Units 1 and 2 were shown on site investigation maps and were referenced by the applicant, the applicant did not include boring logs from these previous investigations. The applicant utilized borings, geophysical surveys, CPTs, seismic CPTs, and test pits as part of the ESP and COL field investigations.

Laboratory Testing

SSAR Section 2.5.4.2.4 describes the laboratory testing of soil samples completed as part of the ESP and COL investigations. The applicant stated that laboratory testing was completed in accordance with the guidance presented in Regulatory Guide 1.138, was performed under an approved quality assurance program with work procedures developed specifically for the ESP and COL applications, and the soil samples were shipped from the onsite storage area to the testing laboratory under Chain-of-Custody procedures. The applicant focused the ESP laboratory test on verifying basic properties of the Upper Sand Stratum, the Blue Bluff Marl and the upper formations of the Lower Sand Stratum. The types and number of tests performed for the ESP investigations are listed in SSAR Table 2.5.4-3, while SSAR Table 2.5.4-4 presents the results. For the COL investigations, the applicant presented the types and number of tests in SSAR Table 2.5.4-3a and the results in Appendix 2.5C. The applicant also performed Resonant Column Torsional Shear (RCTS) testing on samples from the COL investigation and as a part of Phase 1 of the backfill test pad program at the Fugro facility in Houston, TX. The applicant presented the RCTS results for the COL investigation in Appendix 2.5C, Attachment G, while it presented the results for the test Pad program in Appendix 2.5D.

Engineering Properties

SSAR Section 2.5.4.2.5 describes the engineering properties for the soil and rock strata obtained during the ESP and COL subsurface investigations, and the chemical properties deduced as part of the COL investigation. The applicant used data from the COL borings in the immediate vicinity of the VEGP Units 3 and 4 nuclear island power block excavation areas as the basis for the determination of engineering properties. The engineering properties determined during the ESP investigations were derived from both the ESP subsurface and laboratory investigations and the data available from Units 1 and 2. The applicant determined the engineering properties of backfill from the COL and Test Pad program investigations. The applicant compared the properties from the ESP, COL and Test Pad Program to those developed during the previous field and laboratory testing programs conducted for Units 1 and 2 and concluded that the results were similar.

1. Rock Properties. SSAR Subsection 2.5.4.2.5.1 describes the engineering properties of rock at the VEGP Units 3 and 4 site. The applicant based Recovery and Rock Quality Designations (RQD) on results obtained from borehole B-1003, the deepest borehole drilled during the ESP subsurface investigation, which extended 88 m (290 ft) into the bedrock. Although the applicant did not perform any laboratory strength testing of rock cores due to the extreme depth, suspension P-S velocity seismic testing in the borehole was performed to determine shear and compressional wave velocities.

2. Soil Properties. In SSAR Subsection 2.5.4.2.5.2, the applicant described the properties of the soil as determined from ESP and COL investigations, reviews of previous investigations for VEGP Units 1 and 2, and the Phase I test pad program results. To that end, the applicant performed sieve analyses, natural moisture content, and Atterberg Limits tests on Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples as part of the ESP and COL investigations, and made specific gravity measurements on Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples as part of the COL program. The applicant selected design values using the average of the test results for the respective soil strata.

Laboratory test data, SPT N-values, and shear wave velocity measurements from the ESP and COL investigations were used by the applicant to determine the undrained shear strength of the Blue Bluff Marl stratum. This data included UU and CU test results, in addition to laboratory strength testing data from the previous subsurface investigations and construction of VEGP Units 1 and 2. During the ESP investigation, the applicant correlated the average SPT N-value to an internal angle of friction of 34 and 41 degrees for the Upper and Lower Sand Stratum, respectively. Moist unit weights were determined by the applicant for select Blue Bluff Marl and Lower Sand Stratum samples from the ESP laboratory testing program, and Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum samples from the COL laboratory testing program. The applicant stated that the average unit weight for 15 ESP marl stratum and 3 Lower Sand Stratum samples was 1,922 and 1,970 kg/m³ (120 and 123 pcf), respectively. During the COL laboratory testing program, the applicant measured the unit weight of 15 Upper Sand Stratum, 69 Blue Bluff Marl, and 16 Lower Sand Stratum samples, with average unit weights of 1,810, 1,842, and 1,970 kg/m^3 (113, 115, and 123 pcf). The applicant also included the in-situ moist unit weights from previous investigations for the Upper Sand Stratum (1,890

kg/m³ (118 pcf)), the Blue Bluff Marl (1,906 kg/m³ (119 pcf)), and the Lower Sand Stratum (1,874 kg/m³ (117 pcf)).

The applicant compared the design SPT N-values for the ESP investigations with the range and average of the COL and Units 1 and 2 investigations. Based on the ESP results, the applicant concluded that the design SPT N-value for the Upper Sand Stratum (82 bpm (25 bpf)) was within the anticipated range and close to the average. Similarly, the applicant concluded that the design SPT N-value for the Blue Bluff Marl, taken as 328 bpm (100 bpf), also fell within the expected range and near the average N-value. However, when the design SPT N-value for the Lower Sand Stratum (203 bpm (62 bpf)) was compared to the results from the previous investigations, the applicant stated that the design value was less than the assumed range and average.

The applicant measured shear wave velocities by suspension P-S velocity tests and seismic CPTs during the ESP and COL subsurface investigations. Although suspension P-S velocity tests were performed in five ESP boreholes, the applicant acknowledged that only three of the tests extended into the Blue Bluff Marl and Lower Sand Strata, and it therefore the applicant performed tests in six additional COL boreholes. The applicant performed three seismic CPTs for the ESP investigation and eight for the COL; however, due to penetration resistance, the seismic CPTs did not extend into the Blue Bluff Marl. The applicant also determined the shear wave velocities for all strata based on all available data, including measurements from depths of up to 88 m (290 ft) made during the previous VEGP units 1 and 2 investigations and data from seven deep borings performed at the SRS. The velocity ranges determined by the applicant were: 173 to 1,008 meters per second (m/s) (570 to 3310 feet per second (fps)) within the Upper Sand Stratum, 323 to 1298 m/s (1060 to 4260 fps) within the Blue Bluff Marl, 283 to 1423 m/s (930 to 4670 fps) within the Lower Sand Stratum, and 707 to 2849 m/s (2320 to 9350 fps) within the Dunbarton Triassic Basin. The applicant also calculated average shear wave velocities for the formations in the strata: 286 m/s (940 fps) in the Barnwell Formation and 348 m/s (1142 fps) in the Utley Limestone of the Upper Sand Stratum, 624 m/s (2050 fps) in the Blue Bluff Marl, and 533, 567, and 570 m/s (1750, 1863, and 1871 fps) in the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum, respectively.
SSAR Table 2.5.4-6 lists the shear wave velocities for all formations. Using both suspension P-S velocities and seismic CPT results, the applicant developed a complete shear wave velocity profile from the surface to a depth of 408 m (1340 ft).

The applicant derived high strain elastic modulus values for the Upper and Lower Sands, compiled in SSAR Table 2.5.4-1, using the relationship with the SPT N-value given in Davie and Lewis (1988). The applicant derived the high strain elastic modulus for the Blue Bluff Marl stratum using the relationship with undrained shear strength given in Davie and Lewis (1988). The applicant calculated shear modulus values using the relationship between elastic modulus, shear modulus, and Poisson's ratio. The applicant derived the low strain shear modulus values for the strata using the average shear wave velocity. The elastic modulus values were obtained by the applicant from the shear modulus values using the relationship described by Bowles (1982) between elastic modulus, shear modulus, and Poisson's ratio.

3. Chemical Properties. The applicant did not include chemical tests as part of the ESP laboratory testing program, because there were no aggressive chemical subsurface conditions identified during the license renewal aging management analysis of the buried concrete at VEGP Units 1 and 2.

In support of the LWA request, the applicant submitted the following information:

Field Investigations

The applicant's field investigations included the construction of a 6 m (20 ft) thick test pad to test the proposed borrow materials, which aided in the evaluation of the compacted backfill.

Engineering Properties

The applicant also determined the engineering properties of the proposed borrow materials and derived the engineering properties of the structural backfill from the data obtained from the COL investigation and Phase 1 of the test pad program.

Chemical Properties. SSAR Subsection 2.5.4.2.5.3 describes the chemical property testing of the proposed backfill material conducted as part of the COL investigation. The applicant performed laboratory testing for pH, chloride, and sulfate on samples from the Upper Sand Stratum in the power block area, test pits excavated in the switchyard borrow area, and soil samples from Borrow Area 4. Based on the average pH test results of 6.8, 5.2, and 5.4 for samples from the Upper Sand, switchyard, and Borrow Area 4, respectively, and corresponding average chloride test results of 188, 76, and 138 parts per million (ppm), the applicant concluded the soil was mildly corrosive. Citing average sulfate test results of 21, 9.8, and 16.3 ppm, the applicant indicated that the soil/concrete interaction would provide a mild exposure for sulfate attack.

2.5.4.1.3 Exploration

SSAR Section 2.5.4.3 summarizes the results of the subsurface investigation programs conducted by the applicant at the VEGP site, including the previous VEGP Units 1 and 2 program, and the Units 3 and 4 ESP and COL subsurface investigation programs.

Previous Subsurface Investigation Program

SSAR Subsection 2.5.4.3.1 summarizes field investigations completed in the early 1970s for VEGP Units 1 and 2. The applicant stated that although borings, geophysical surveys and groundwater studies were included in these field investigations, additional investigations were needed during the excavation of the power block areas to further understand and verify the subsurface conditions. The applicant stated that of the 474 borings completed for Units 1 and 2, twenty fell within, or are in the immediate vicinity of, the proposed VEGP Units 3 and 4 power block site, and the locations of these borings were provided on SSAR Figure 2.5.4-1b. Some of the investigations the applicant considered during the review of the previous programs included: electric logging, natural gamma, density, neutron, caliper, and 3-D velocity logs (Birdwell) in selected boreholes, water pressure and Menard pressuremeter testing of the Blue Bluff Marl,

and fossil, mineral or soluble carbonate testing on recovered samples. The applicant supplemented test borings with geophysical methods, completing a total of 8,650 m (28,400 ft) of shallow refraction lines, 1,525 m (5,000 ft) of deep refraction lines, and subsurface cross-hole velocities from the ground surface to a depth of 88 m (290 ft). The applicant referenced the results of these investigations to support the data obtained during the later ESP and COL subsurface investigations.

ESP Subsurface Investigation Program

SSAR Subsection 2.5.4.3.2 describes the ESP subsurface investigation program performed in late 2005 over a substantial portion of the site which would contain the VEGP Units 3 and 4 reactors, switchyard, and cooling towers. The applicant utilized exploration points, as shown on Figure 2.5.4-1 of this SER, to confirm the results of the previous investigation. In addition, the applicant stated that it developed an exploration program, in accordance with Regulatory Guide 1.132, including an audited and approved quality assurance program, and site-specific work procedures. Once the program was established, the applicant performed a variety of field investigations, including 13 exploratory borings, ten CPTs, three seismic CPTs, in-situ hydraulic conductivity tests, five geophysical down-hole suspension P-S velocity logging, a topographic survey of exploration points, and laboratory testing of borehole samples. The applicant also completed a seismic reflection and refraction survey at the VEGP site to collect additional data, which helped delineate the rock profile associated with the non-capable Pen Branch fault.

a) Borings and Samples/Cores. SSAR Subsection 2.5.4.3.2.1 describes the thirteen borings drilled for the ESP investigation with depths from 27 m (90 ft) to 93 m (304 ft). The applicant advanced the borings using mud-rotary drilling techniques, polymer and/or bentonite drilling fluids, and an SPT sampler with automatic hammers to collect samples at continuous intervals to 5 m (15 ft) and at 1.5 to 3 m (5 to 10 ft) intervals thereafter. SSAR Table 2.5.4-7 provides a summary of the ESP boring and CPT locations and depths, and identifies the geophysical testing performed in the boreholes. In addition, the applicant obtained undisturbed samples of the Blue Bluff Marl using rotary pitcher samplers. In accordance with ASTM D 2488, the applicant processed the recovered soil samples by first visually describing the samples and placing them in a labeled moisture-proof glass jar before transporting the samples, in boxes, to an onsite storage facility. Finally, the applicant provided a summary of all undisturbed samples collected from the Blue Bluff Marl during the ESP investigation and described the materials encountered during the ESP borings as similar to those found in the borings from the previous investigation at the VEGP site.

   The applicant performed one continuous core boring, B-1003, that was cased through the soil column to prevent cave-ins and allowed for coring of the rock at depths below 320 m (1,049 ft). The applicant placed the recovered soil and rock core samples in wooden boxes lined with plastic sheeting, and the onsite geologist visually described the core. The applicant's geologist computed and recorded the percentage recovery (average core recovery was 77 percent) and the rock quality designation (RQD), before the filled core boxes were transported to the onsite sample storage facility where the core was photographed.

b) Cone Penetrometer Tests. SSAR Subsection 2.5.4.3.2.2 describes the CPTs conducted in accordance with ASTM D 5778 during the ESP site investigations. Using a Type 2 piezocone, the applicant advanced each CPT to refusal at depths ranging from 2 to 35 m (6 to 116 ft); offset CPTs were performed for borings with shallow refusal depths. The applicant noted that, with few exceptions, all of the CPT locations met refusal at or near the top of the Blue Bluff Marl. The applicant performed down-hole seismic testing at 1.5 m (5 ft) intervals in three CPTs to measure shear wave velocity in the Upper Sand Stratum and pore pressure dissipation tests at depths between 17 and 30 m (56 and 99 ft) in four CPTs. SSAR Appendix 2.5A contains the CPT logs, shear wave velocity results, and the pore pressure versus time plots developed from the dissipation tests.

c) In-situ Hydraulic Conductivity Testing. The applicant installed fifteen observation wells in the ESP project limits and developed each by pumping until the pH and conductivity stabilized and the pumped water was reasonably free of suspended sediment. SSAR Subsection 2.5.4.3.2.3 describes the slug tests performed in each well in accordance with ASTM D 4044. The applicant described the slug test method as the lowering of a solid cylinder into a well to increase the water level, recording the time it took the well water to return to the pre-static level, then rapidly removing the cylinder and again recording the time it took the water to recover to the pre-static level. To record the water levels and time intervals during testing, the applicant used electronic transducers and data loggers. SSAR Section 2.4.12 and Appendix 2.4A contain additional details.

d) Sample Re-evaluation. SSAR Subsection 2.5.4.3.2.4 describes the revisions the applicant made to the ESP data report based on additional laboratory data and upon re-evaluation of samples. Upon re-examination of the coarse grained fractions, previously described in the Blue Bluff Marl and Utley Limestone as gravel, the applicant found the samples consisted of angular, gravel-sized, carbonate particles that were attributed to mechanical breakage of cemented nodules, shells, cemented limestone, and fossiliferous limestone by the split barrel sampler. The applicant also redefined the top of the Utley Limestone in some of the ESP boreholes based on the identification criteria developed for the COL subsurface investigation program.

COL Subsurface Investigation Program

SSAR Subsection 2.5.4.3.3 details the COL subsurface investigation conducted over a large portion of the site, including the VEGP Units 3 and 4 power block areas, cooling towers, switchyard/borrow areas, haul road, intake structure, pump house, pipeline, and other construction-related areas, locating the exploration points in accordance with guidelines in RG 1.132. As part of its investigation, the applicant completed 174 exploratory borings across the site, 21 CPTs, eight seismic CPTs, geophysical down-hole suspension logging in six boreholes, electrical resistivity testing along ten arrays across the site, geophysical refraction microtremor (ReMi) testing across four arrays, horizontal and vertical surveys of all exploration points, and laboratory testing, including RCTS tests for selected borehole samples. The applicant stated that it performed the field investigations under an audited and approved quality assurance (QA) program using approved work procedures developed specifically for the COL site investigation. Prior to the start of the field investigations, the applicant established an onsite

storage facility for soil samples which included an inventory control system. SSAR Table 2.5.4-7a provides a summary of the locations of COL borings, CPTs and test pits.

1. Borings and Samples/Cores. SSAR Subsection 2.5.4.3.3.1 describes the 174 borings drilled to depths of 6.5 to 128 m (21.5 to 420 ft). Using mud-rotary methods, polymer and/or bentonite drilling fluids, and an SPT sampler with automatic hammers, the applicant sampled the soil at 0.75 m (2.5 ft) intervals within the upper 4.5 m (15 ft) and at 1.5 to 3 m (5 or 10 ft) intervals thereafter. The applicant stated that the soils encountered in the COL borings were similar to those encountered during the ESP and Units 1 and 2 investigations at the VEGP site. The applicant used the same sample processing and storage procedures that were used for the ESP investigation. The applicant also obtained relatively undisturbed samples from the Upper Sand Stratum using the direct push method, and, due to the very hard/dense nature of the materials, used a Pitcher sampler (a double-tube core barrel sampler) for sampling the Blue Bluff Marl and Lower Sand Stratum.

2. Cone Penetrometer Tests. The applicant advanced 21 CPTs to refusal for the COL investigation. SSAR Subsection 2.5.4.3.3.2 states that refusal was generally encountered at or near the top of the Blue Bluff Marl stratum and ranged in depth from 20 to 30.5 m (65.4 to 100.4 ft). The applicant performed seismic testing in eight of the CPTs located in the power block and cooling tower areas of Units 3 and 4.

3. Test Pits. The applicant excavated eight test pits in the proposed borrow areas. SSAR Subsection 2.5.4.3.3.3 describes how a geologist visually examined the excavation walls, prepared a Geotechnical Test Pit log based on the visual examination in accordance with ASTM D 2488, and collected representative bulk samples of the material types in moisture retaining glass jars. The applicant also used a backhoe to backfill the test excavation with the excavated materials.

4. Resistivity. Using the Wenner four electrode test method, the applicant performed field resistivity testing along ten arrays in the proposed switchyard, cooling tower and circulating water line areas of the site. SSAR Figures 2.5.4-1a and -1b illustrate the locations of arrays and SSAR Subsection 2.5.4.3.3.4 states that the locations and array lengths were adjusted to accommodate obstructions. The applicant used electrode spacings from 1 to 91 m (3 to 300 ft) to determine the soil resistivity at increasing depths.

2.5.4.1.4 Geophysical Surveys

SSAR Section 2.5.4.4 includes four subsections summarizing the applicant's previous geophysical investigations for VEGP Units 1 and 2, the geophysical program used for the ESP investigation, the geophysical surveys performed as part of the COL investigation, and geophysical surveys from the Phase I test pad program conducted in support of the LWA request.

In support of the ESP application, the applicant submitted the following information:

Previous Geophysical Survey Programs

SSAR Subsection 2.5.4.4.1 describes the geophysical seismic refraction and cross-hole surveys used to evaluate the subsurface materials during the investigations for VEGP Units 1 and 2. The applicant used the seismic refraction survey to determine the depths to seismic discontinuities based on compressional wave velocity measurements, and obtained shallow and deep refraction profiles throughout the site for a combined total depth of 8,650 and 1,525 m (28,400 and 5,000 ft), respectively. The applicant conducted a cross-hole survey in the power block area to determine the in-situ velocity data for both compressional and shear waves to a depth of 88 m (290 ft), or approximately 25 m (82 ft) below sea level, in six boreholes. The applicant also determined cross-hole velocities by lowering three-component geophones into four of the boreholes to equal levels and generating energy at the same level in a fifth hole.

The applicant also examined compressional and shear wave velocity data from the previous investigations, and used the velocities to determine the Young's Modulus and Shear Modulus for the 88 m (290 ft) closest to the surface. The applicant stated that the seismic (compressional) wave velocities ranged from 426 to 2,026 m/s (1,400 to 6,650 fps) with a shear wave velocity of 182 to 502 m/s (600 to 1,650 fps) for the Upper Sand Stratum (depth from 0 to 27 m (90 ft), while the Blue Bluff Marl stratum, and the underlying Lower Sand Stratum, had a compressional wave velocity of 2,072 m/s (6,800 fps), with shear wave velocities from 487 to 548 m/s (1,600 to 1,800 fps) from 27 to 88 m (90 to 290 ft). The applicant calculated a range of Young's and Shear Moduli for the Upper Sand and the Blue Bluff Marl, including the Lower Sand Stratum.

ESP Geophysical Surveys

SSAR Subsection 2.5.4.4.2 describes the geophysical surveys performed by the applicant as part of the ESP investigations, including suspension P-S velocity tests and down-hole seismic CPTs, as well as a discussion and interpretation of results.

1. Suspension P-S Velocity Tests in Boreholes. The applicant conducted suspension P-S velocity tests in five ESP borings, two of which did not extend below the Upper Sand Stratum. The applicant referred to Ohya (1986) for the details of equipment used to create the seismic compressional and shear waves and to measure the seismic wave velocities. SSAR Subsection 2.5.4.4.2.1 describes the suspension P-S velocity logging system used by the applicant, which incorporated a 7 m (23 ft) probe containing a source near the bottom, and two geophone receivers spaced 1 m (3.3 ft) apart. The applicant lowered the probe into the borehole, where the source generated a pressure wave at depth that was converted to seismic waves (P-wave and S-wave) at the borehole wall. These waves were converted back to pressure waves in the fluid and received by the geophones, which sent the data to a recorder at the surface. The applicant repeated the procedure every 0.5 to 1.0 m (1.65 to 3.3 ft) and used the results to determine the average velocity of a 1 m (3.3 ft) high column of soil around the borehole.

   The applicant defined the shear wave and compressional wave velocities for each stratum to the maximum depth of 407 m (1,338 ft). The average shear wave velocities determined by the applicant were 331 m/s (1,089 fps) for the Upper Sand stratum, 717 m/s (2,354 fps)

for the Blue Bluff Marl, and 695 m/s (2,282 fps) for the Lower Sand Stratum, while average compressional wave velocities were 784 m/s (2,572 fps), 2,070 m/s (6,793 fps), and 2,014 m/s (6,610 fps), respectively. The applicant also presented typical values for shear wave velocities for each geologic formation contained within the Lower Sand Stratum; 518 m/s (1,700 fps) in the Still Branch, 594 m/s (1,950 fps) in the Congaree, 624 m/s (2,050 fps) in the Snapp, 716 m/s (2,350 fps) in the Black Mingo, 807 m/s (2,650 fps) in the Steel Creek, 868 m/s (2,850 fps) in the Gaillard/Black Creek, 874 m/s (2,870 fps) in the Pio Nono, and 826 m/s (2,710 fps) in the Cape Fear. The shear wave and compressional wave velocity range was also measured for a portion of the Dunbarton Triassic Basin rock, which the applicant determined was between 707 to 2,849 m/s (2,320 to 9,350 fps) and 2,225 to 5,596 m/s (7,300 to 18,360 fps), respectively. The applicant concluded that shear wave velocities increased linearly with depth at a very high rate, a rate that lessened once shear wave velocities achieved values of about 1,615 m/s (5,300 fps). The applicant noted that sound rock with an average shear wave velocity of 2804 m/s (9,200 fps) was not encountered at the site, but was extrapolated from the measured results. The applicant used both shear and compressional wave velocities to calculate Poisson's ratios for the Upper Sand, Blue Bluff Marl, Lower Sand and Dunbarton Triassic Basin rock strata.

2. Down-Hole Seismic Tests with Cone Penetrometer. SSAR Subsection 2.5.4.4.2.2 describes the three CPTs performed at 1.5 m (5 ft) intervals as part of the ESP investigation. The applicant obtained measurements at depths within the Upper Sand Stratum since all CPTs reached refusal at the top of the Blue Bluff Marl. To complete this test, the applicant located a seismic source on the surface that generated shear waves, and it mounted two geophones horizontally near the bottom of the cone string to record incoming seismic data. The applicant measured shear wave velocities that were lower than those determined by the suspension P-S velocity technique: these lower velocities may reflect site variability.

3. Discussion and Interpretation of Results. The applicant recommended design values for each stratum based on shear and compressional wave velocity measurements. SSAR Subsection 2.5.4.4.2.3 states that seismic CPTs and suspension velocity logging were used to develop the values for the Upper Sand Stratum, but, due to the CPT refusal at the top of the Blue Bluff Marl, only suspension velocity logging results were used to determine the values for the Blue Bluff Marl and Lower Sand Stratum. The applicant did not make any shear or compressional wave velocity measurements for compacted fill during the ESP subsurface investigation, but it recommended values for the compacted fill based on data from VEGP Units 1 and 2, values which would be confirmed during the COL investigations and Phase 1 of the test pad program.

COL Geophysical Surveys

SSAR Subsection 2.5.4.4.3 describes the suspension P-S velocity tests, down-hole seismic CPT tests, and ReMi tests performed during the COL site investigation.

1. Suspension P-S Velocity Tests in Boreholes. The applicant conducted six suspension P-S velocity tests using the equipment described by Ohya (1986) to measure the seismic wave velocities. The method used by the applicant was the same as was used during the ESP investigations summarized in the previous section of this SER. The applicant defined the

shear wave velocity to a maximum depth of 128 m (420 ft). Shear wave velocities were determined by the applicant for the Blue Bluff Marl (386 to 909 m/s (1,267 to 2,984 fps)) and the Lower Sand Stratum (227 to 781 m/s (745 to 2,563 fps). The applicant also provided the average velocities for the geologic formations contained within the Lower Sand Stratum; 494 m/s (1,621 fps) for the Still Branch, 567 m/s (1,863 fps) for the Congaree, and 570 m/s (1,871 fps) for the Snapp. As with the ESP investigation, the applicant also determined a range of Poisson's ratios and Figure 2.5.4-3 of this SER illustrates the shear wave velocity profile through borehole B-1003.

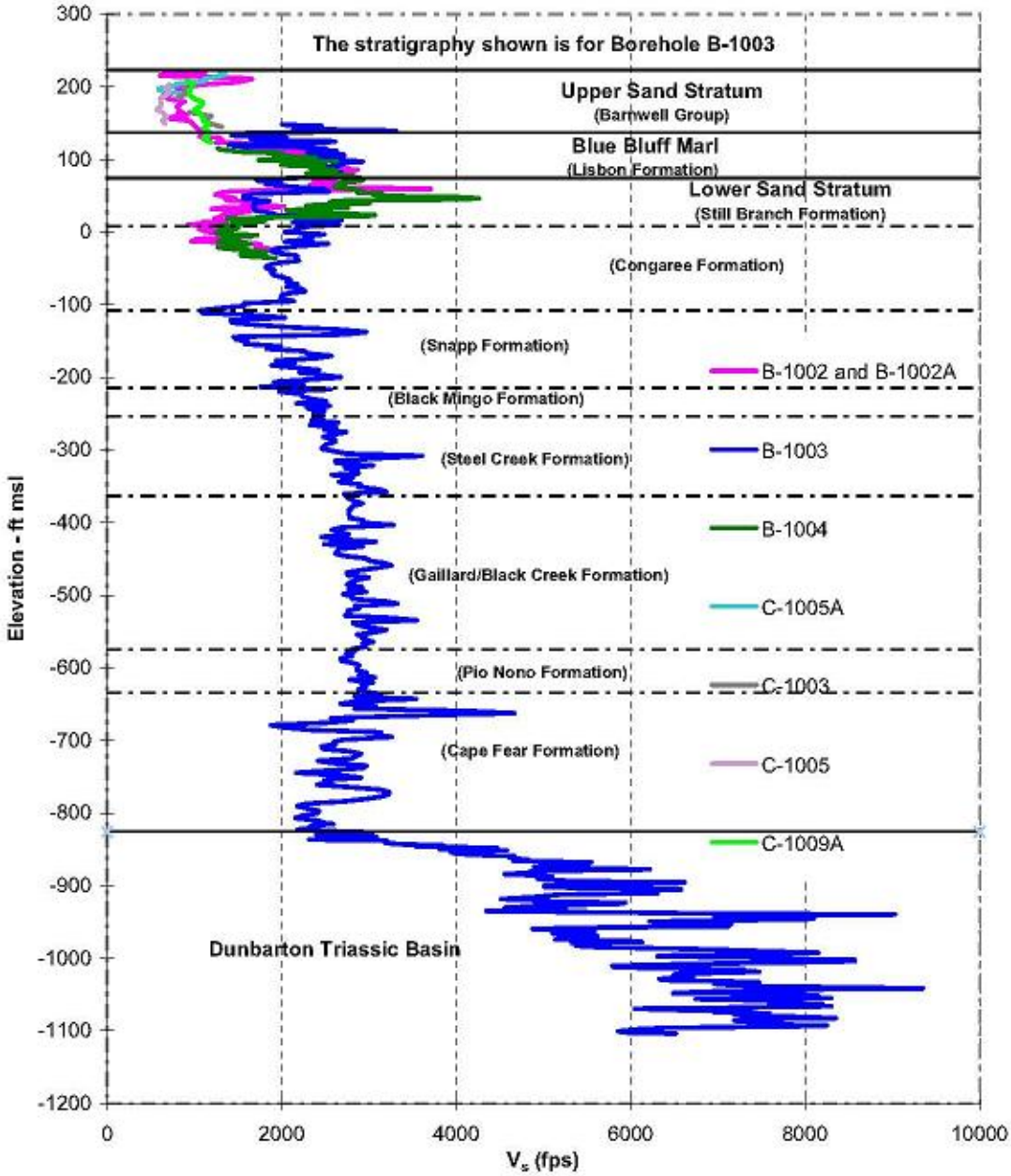


Figure 2.5.4-3 Shear Wave Velocity Measurements (SSAR Figure 2.5.4-6)

2. Down-Hole Seismic Tests with Cone Penetrometer. SSAR Subsection 2.5.4.4.3.2 describes the eight CPTs performed at 0.2 m (0.6 ft) intervals as part of the COL investigation. The method used by the applicant was the same as was used during the ESP investigations which the applicant presented in SSAR Subsection 2.5.4.4.2. Although penetrations depths ranged from 20 to 30.5 m (65.4 to 100.4 ft), CPT soundings could not penetrate the dense/hard materials encountered in the Utley Limestone and Blue Bluff Marl, and therefore the applicant was only able to obtain measurements in the Upper Sand Stratum. The applicant reported shear wave velocity measurements of 132 to 1,158 m/s (435 to 3,802 fps), and it plotted the summary of the average COL shear wave velocity profiles in the Upper Sand Stratum in SSAR Figure 2.5.4-6a.

3. Refraction Microtremor Testing. The applicant conducted ReMi testing across two arrays in the power block areas of the existing VEGP Units 1 and 2 and two arrays in the footprint of the proposed Units 3 and 4. SSAR Subsection 2.5.4.4.3.3 states that although ReMi testing was originally intended to establish the shear wave velocity characteristics of the existing backfill at Units 1 and 2, the applicant noticed a frequency interference from the equipment of the operating plant on the ReMi. Although the applicant attempted to overcome the interference and consulted with Dr. K. Stokoe, the applicant concluded that the results did not truly represent the shear wave velocity profile, and therefore these results were not considered in the COL geophysical survey conclusions.

In support of the LWA request, the applicant submitted the following information:

Geophysical Surveys in Compacted Fill

The applicant conducted a test pad program that included the construction of a 6 m (20 ft) deep compacted test fill pad using the proposed backfill materials. SSAR Subsection 2.5.4.4.4 describes the geophysical surveys conducted at three different levels within the test pad to evaluate the shear wave profile in the compacted backfill. The applicant stated that it determined the shear wave velocity using the Spectral Analysis of Surface Waves (SASW) method at various stages of construction and upon completion of the test pad; the cross-hole method was used to measure shear wave velocity through the compacted test fill. Upon completion of the test pad, the applicant installed and measured compressional and shear wave velocities between three cased boreholes extending through the test pad into native materials. The applicant incorporated the results, along with RCTS test results, into the analysis to develop the shear wave profile through the entire depth (about 27 m (90 ft)) of proposed backfill.

2.5.4.1.5 Excavation and Backfill

SSAR Section 2.5.4.5 summarizes the excavation and backfill for VEGP Units 3 and 4, including the extent of safety-related excavations, fills, and slopes; excavation methods and stability; an overview of backfill design; a discussion of backfill sources; quality control and ITAAC; control of groundwater during excavation; and retaining wall construction.

In support of the LWA application, the applicant submitted the following information:

Extent of Excavations, Fills, and Slopes

SSAR Subsection 2.5.4.5.1 describes the substantial excavations necessary for construction of VEGP Units 3 and 4. The applicant presented subsurface profiles providing the grade elevation range across the site, one of which is presented as Figure 2.5.4-2 in this SER. Since the existing ground elevation was at Elevation (El.) 67 m (220 ft) above mean sea level (msl), while the base of the nuclear island foundations for the proposed new units would be at about El. 55 m (180 ft) msl, the applicant determined that the entirety of the Upper Sand Stratum would be excavated for the Units 3 and 4 power blocks. Based on the borings completed during the ESP and COL subsurface investigations, the applicant concluded that the total depth of excavation to the top of the Blue Bluff Marl will range from 24 to 27 m (80 to 90 ft) below the existing grade, with deeper localized excavations using conventional excavating equipment to remove potentially weathered zones in the upper portion of the Blue Bluff Marl.

The applicant stated that once the excavation was complete, Seismic Category 1 backfill would be placed from the top of the Blue Bluff Marl to the bottom of the nuclear island foundation. Although Seismic Category 2 backfill would be used above the nuclear island foundation level, the applicant stated that all of the backfill placed above the foundation would be engineered to the same criteria as Seismic Category 1 backfill. The applicant also described plans to construct a retaining wall along the perimeter of the nuclear island to facilitate construction and backfilling operations with Seismic Category 2 backfill behind it to final grade or foundation elevation of non-nuclear island structures. The applicant described this backfill as granular material selected from portions of the excavated Upper Sand Stratum and other acceptable onsite borrow sources.

Excavation Methods and Stability

SSAR Subsection 2.5.4.5.2 describes the applicant's plans to excavate and stabilize the large volume of Upper Sand Stratum that needs to be removed. The applicant described plans to use conventional equipment to remove any weathered material encountered at the top of the Blue Bluff Marl, and would slope any necessary excavations to facilitate placement of compacted structural fill. The applicant described the overall excavation as an open-cut excavation, with slopes no steeper than 2-horizontal to 1-vertical (2h:1v), and adhering to OSHA regulations (OSHA 2000). The applicant stated that all slopes would be sealed and protected from the highly erosive sandy soils. The applicant determined that where vertical cuts were required due to space constraints, sheet pile or soldier and lagging walls would be adequate support. The applicant determined there were no permanent slopes that need to be considered for stability in the nuclear island area. Finally, the applicant concluded that dewatering operations would be needed once the excavation progressed to depths beneath the groundwater table, approximately El. 45 to 47 m (150 to 155 ft), based on groundwater monitoring results from SSAR Section 2.4.12.

Control of Groundwater During Excavation

SSAR Subsection 2.5.4.5.6 refers to SSAR Subsection 2.5.4.6.2 for a discussion of construction dewatering. However, the applicant stated that because the Upper Sand Stratum soils were highly erosive, the tops of all excavations would be sloped back to prevent runoff, and sumps

and ditches constructed for dewatering purposes would be lined, although the applicant did not describe the liner material.

Backfill Design

The applicant established the design of the Seismic Category 1 and Seismic Category 2 backfill for VEGP Units 3 and 4 through analysis and testing of the proposed borrow materials during the COL investigation, Phase I of the test pad program, and the previous site investigations for VEGP Units 1 and 2. SSAR Subsection 2.5.4.5.3 describes the selection and compaction requirements for the backfill. The applicant stated that it selected materials for Seismic Category 1 and Seismic Category 2 backfill that were sands and silty sands that met the gradation requirements specified in SSAR Table 2.5.4-14. According to the applicant, material not within the requirements was evaluated on a case-by-case basis to assess the overall impact of the material on backfill design, although the applicant considered borrow material that did not meet the limits on percentage of particle sizes smaller than the No. 200 (0.075mm) sieve to be unacceptable for use. The applicant stated that all Seismic Category 1 and 2 backfill materials would be compacted to a minimum of 95 percent of the maximum dry density as determined by the ASTM D 1557 standard test method.

The applicant utilized a two-phase test pad program to establish site-specific design properties for the structural backfill materials, verify the materials would satisfy the AP1000 standard plant design siting criteria for a shear wave velocity of at least 304.8 m/s (1,000 fps), and finalize the placement procedures and equipment. For Phase I, the applicant constructed a 6 m by 18 m by 6 m (20 ft by 60 ft by 20 ft) test pad below grade in the switchyard borrow area using methods similar to those used to construct the VEGP units 1 and 2 structural backfill. The applicant stated that it utilized field and laboratory tests, including density, SASW, SPTs, moisture density relationships, grain size distribution, percentage of fine material and plasticity, shear, and shear modulus and damping relationships, to determine the backfill properties. SER Table 2.5.4-1 presents the calculated shear wave velocity profile based on field measurements of velocity in the test pad and in laboratory samples. After interpreting this data, the applicant concluded that the siting criterion for a shear wave velocity of at least 304.8 m/s (1,000 fps) at the nuclear island foundation had been achieved using the proposed backfill materials within the thickness of the test pad.

Table 2.5.4-1 Estimated (ESP) Shear Wave Velocity and Dynamic Shear Modulus Values and Calculated (COL) Shear Wave Velocity Values for Compacted Backfill

Estimated (ESP)			Calculated (COL)	
Depth m (ft)	**Vs (fps)**	**Gmax (ksf)**	**Depth m (ft)**	**Vs (fps)**
0 to 1.8 (0 to 6)	573	1,255	0 (0)	550
1.8 to 3 (6 to 10)	732	2,049	1.5 (5)	724
3 to 4.2 (10 to 14)	811	2,510	3 (10)	832
4.2 to 5.5 (14 to 18)	871	2,898	6 (20)	975
5.5 to 7 (18 to 23)	927	3,280	9.1 (30)	1,064
7 to 8.8 (23 to 29)	983	3,694	12.2 (40)	1,130
8.8 to 11 (29 to 36)	1,040	4,130	15.2 (50)	1,183
11 to 13.1 (36 to 43)	1,092	4,553	18.2 (60)	1,228
13.1 to 15.2 (43 to 50)	1,137	4,940	21.3 (70)	1,267
15.2 to 17 (50 to 56)	1,175	5,274	24.4 (80)	1,302
17 to 19.2 (56 to 63)	1,209	5,588	25.9 (85)	1,318
19.2 to 21.6 (63 to 71)	1,232	5,796	26.3 (86.5)	1,327
21.6 to 24 (71 to 79)	1,253	6,001	26.8 (88)	1,327
24 to 26.2 (79 to 86)	1,273	6,186	-	-

The applicant stated that Phase II of the test pad program would be used to finalize the placement procedures and equipment, including the material placement procedures and equipment types, construction methods, compaction requirements and methods, and the testing protocol, that would be used during the emplacement of backfill. The applicant described plans to use onsite borrow material excavated from the switchyard and nuclear island areas and its eventual intent to incorporate the backfill placement and compaction methodologies into its earthwork specifications and implementing procedures prior to beginning approved excavation and backfill operations. The applicant completed the Phase II test pad program in July 2008 and incorporated the results into the revised SSAR. The applicant evaluated the results of the

various types and combinations of equipment and methodologies used during the program and stated that it determined the optimum placement and compaction strategy for the material types proposed for structural backfill. The applicant stated that it planned to develop its soils specification and structural backfill implementing procedures prior to the start of approved construction activities. However, the applicant did provide the staff with the draft procedures used for the test pad program, which the applicant stated it would use as the basis for its actual specification and procedures. The applicant also stated that the final specifications and corresponding implementing procedures would be developed in accordance with the applicant's approved quality assurance/quality control program prior to its commencement of any actual construction activities approved under the LWA.

Backfill Sources

SSAR Subsection 2.5.4.5.4 describes the backfill material sources that the applicant identified at the Vogtle site through borings and laboratory testing programs and analyses. The applicant identified onsite borrow material sources, including the acceptable portion of Upper Sand Stratum material excavated from the power block and switchyard area north of the power block, and from an alternative location (Borrow Area 4) that was identified and investigated during construction of VEGP Units 1 and 2. The applicant stated that flowable backfill may be used in small restricted areas where adequate compaction may not be achieved; this flowable backfill would be designed to have similar strength characteristics as the proposed compacted backfill materials. The applicant stated that approximately 2,750,000 cubic meters (m^3; 3,600,000 cubic yards(yds^3)) of material were necessary to complete backfilling of the planned 3,000,000 m^3 (3,900,000 yds^3) excavation. Based on the COL investigation and laboratory testing, the applicant estimated that 50 percent of the material excavated from the powerblock area would be suitable backfill material; however, as the suitable and unsuitable materials were generally inter-layered, the applicant conservatively estimated the recovery of about 900,000 m^3 (1,200,000 yds^3) of usable material.

The applicant determined that the remaining 1,800,000 m^3 (2,400,000 yds^3) of backfill needed for the power block areas was available from an old borrow stockpile area, developed during the construction of Units 1 and 2 and located to the north of the power blocks in the area of the switchyard for Units 3 and 4. SER Figure 2.5.4-4 (SSAR Figure 2.5.4-15) show the plan and section views, respectively, of this borrow area. The applicant explored the switchyard area with fifteen SPT borings and five test pits during the COL investigation and determined that the needed volume of suitable backfill material was available at the switchyard borrow source. The applicant classified the material as silty sands and poorly graded sands, with lesser amounts of clayey sands in some samples.

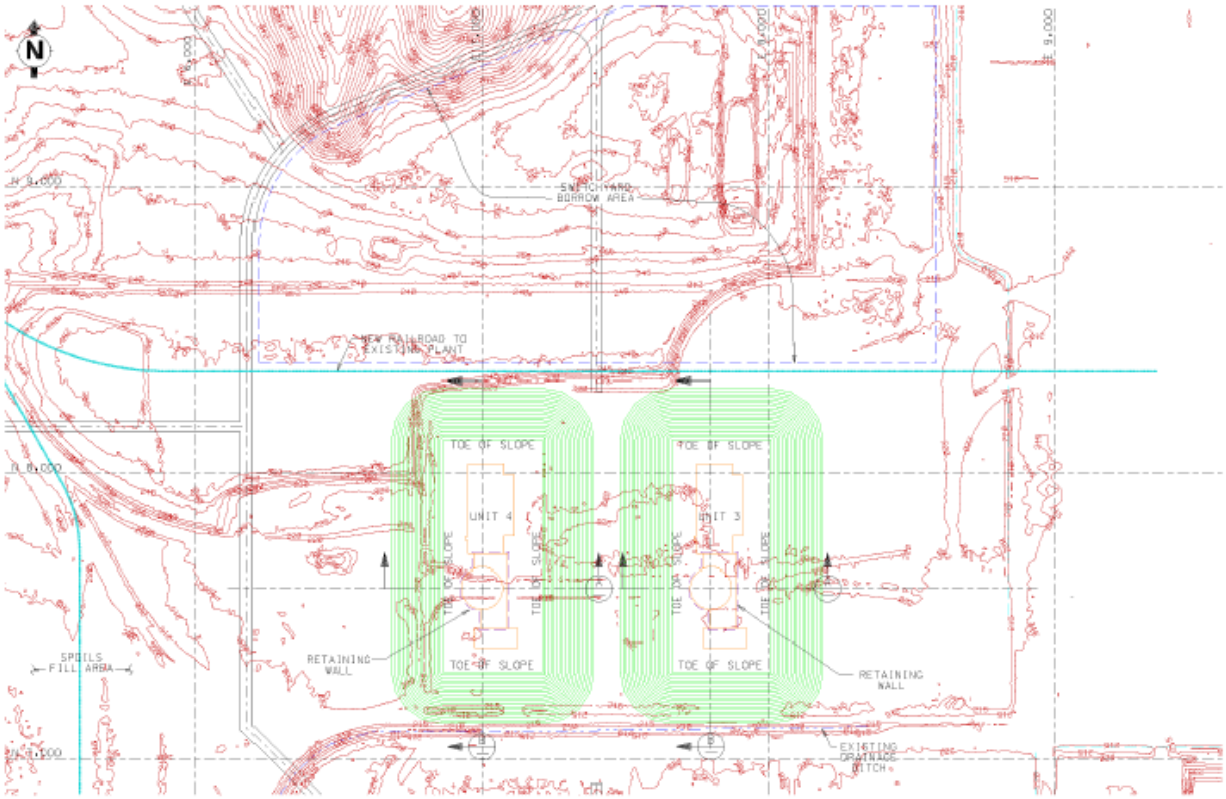


Figure 2.5.4-4 Power Block Excavation and Switchyard Borrow Areas (SSAR Figure 2.5.4-15)

In addition to the switchyard borrow source, the applicant also explored an alternative borrow source, Borrow Area 4, located about 1,220 m (4,000 ft) north of the power block area. Utilizing the results of four SPT borings and three test pits to add to the exploration data for Units 1 and 2, the applicant concluded that approximately 900,000 cubic meters (1,200,000 cubic yards) of suitable backfill material were available from the surface to a depth of 11 m (36 ft; El. 75 m) at Borrow Area 4.

Quality Control and ITAAC

SSAR Subsection 2.5.4.5.5 describes the quality control and quality assurance program that would be established by the applicant to verify that the backfill was constructed to design requirements as well as the applicable Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC). The applicant detailed plans to use a soil testing contractor with an onsite laboratory and a separate earthwork contractor, each of which would be monitored independent of the other. From the soil testing contractor, the applicant expected that sufficient laboratory modified compaction and grain size distribution tests would be performed to ensure that variations of fill material were addressed.

The applicant stated that an additional quality control program would be applied to all aspects of the backfill testing program, from qualification of borrow material to confirmatory shear wave velocity testing of the as-placed backfill. Qualification of the borrow materials would include soil classification, grain size distribution, and laboratory moisture-density relationship (modified Proctor compaction) tests. These results were used by the applicant to determine the acceptability of borrow materials and the optimum moisture content for field soil compaction. The applicant stated that field density testing would be performed to verify the compaction requirements were met. For earthwork in limited areas, where fill was compacted with hand equipment, there would be one test for every 608 square meters per meter (2,000 square ft per ft) of material placed; for mass earthwork for both Seismic Category 1 and Seismic Category 2, a minimum of one test for every 382 cubic meters (500 cubic yards) of compacted fill, but no less than one test per every lift was performed, and at least two field density tests per lift were located within the footprint directly beneath the nuclear island.

The applicant also planned to review backfill test results, backfill-related non-conformance reports, and QA audits of backfill operations to determine if the as-built backfill met the requirement of 95 percent for minimum compaction for backfill under Seismic Category 1 structures. Only the field density tests performed on backfill directly beneath the nuclear island would be used in the evaluation that would be submitted by the applicant in a report to support ITAAC closure.

Shear wave velocity tests, as measured by the SASW method, would be performed by the applicant on the completed backfill to confirm that the shear wave velocity at the bottom of the nuclear island foundation was greater than or equal to 304.8 m/s (1,000 fps). The applicant also described plans to develop a report to document that the ITAAC requirement for shear wave velocity was met. Preliminary measurements of the shear wave velocity characteristics of the backfill made when placement of backfill reached the approximate elevation of the bottom of the nuclear island foundation, SASW measurements taken within the foundation footprint, representative measurements from locations outside the nuclear island footprint, and SASW

measurements made at finish grade would all be used by the applicant to document that the backfill shear wave velocity profile at the elevation of the bottom of the foundation and below was greater than or equal to 304.8 m/s (1,000 fps). Finally, the applicant described plans to use a second method, such as cross-hole testing or seismic CPT, to measure shear wave velocity at one of the finish grade reference locations to validate the SASW results at the same reference. In the event that the velocity measurements do not provide adequate evidence to support closure of the ITAAC, the applicant stated that additional testing and evaluations would be completed before the final report to close the ITAAC is completed. A table of the backfill ITAAC was also provided in the SSAR (now SER Table 2.5.4-2):

Table 2.5.4-2 Backfill ITAAC

Design Requirement	Inspections and Tests	Acceptance Criteria
Backfill material under Seismic Category 1 structures is installed to meet a minimum of 95 percent modified Proctor compaction.	Required testing will be performed during placement of the backfill materials.	A report exists that documents that the backfill material under Seismic Category 1 structures meets the minimum 95 percent modified Proctor compaction.
Backfill shear wave velocity is greater than or equal to 1,000 fps at the depth of the NI foundation and below.	Field shear wave velocity measurements will be performed when backfill placement is at the elevation of the bottom of the Nuclear Island foundation and at finish grade.	A report exists and documents that the as-built backfill shear wave velocity at the NI foundation depth and below is greater than or equal to 1,000 fps.

Retaining Wall

SSAR Subsection 2.5.4.5.7 describes the applicant's plans to construct a mechanically stabilized earth (MSE) retaining wall within each power block excavation to facilitate construction of the nuclear island. The applicant stated that the MSE wall would permit backfilling of the excavations before construction of the nuclear island foundations and substructure walls as well as act as the exterior formwork for the foundation and substructure walls. The applicant also described plans to waterproof the surface of the pre-cast concrete MSE wall facing panels before placing the concrete for the nuclear island foundation and substructure walls.

2.5.4.1.6 Groundwater Conditions

SSAR Section 2.5.4.6 describes the groundwater conditions at the site, including groundwater measurements and elevations, and construction dewatering.

In support of the ESP application, the applicant submitted the following information:

Groundwater Measurements and Elevations

In SSAR Section 2.5.4.6.1, the applicant presented a summary of groundwater conditions at the site of VEGP Units 3 and 4; additional detailed discussions can be found in SSAR Section 2.4.12. The applicant stated that groundwater was present in unconfined conditions in the Upper Sand Stratum and in confined conditions in the Lower Sand Stratum at the VEGP site. The applicant concluded that the Blue Bluff Marl was an aquiclude, a unit which absorbs and holds but does not transmit water, separating the unconfined water table aquifer in the Upper Sand from the confined Tertiary aquifer in the Lower Sand, with groundwater generally occurring at depths between 19 and 21 m (65 and 70 ft) below the existing ground surface.

In mid-2005, prior to the start of the ESP subsurface investigation program, the applicant installed ten observation wells in the unconfined aquifer and five wells in the confined aquifer. The applicant also used the existing wells, thirteen in the unconfined aquifer and nine in the confined aquifer, to monitor groundwater levels at the site. The groundwater levels in the unconfined water table wells ranged from elevation (El.) 40 to 50 m (132 to 165 ft), and the levels in the confined aquifer ranged from El. 25 to 39 m (82 to 128 ft). The applicant performed hydraulic conductivity (slug) tests in the wells, using the same method that was described in SSAR 2.5.4.3.2.3. Based on the slug test results, the applicant concluded that the hydraulic conductivity (k) values for the unconfined water table aquifer in the Upper Sand Stratum ranged from 4.4×10^{-5} to 9.3×10^{-4} cm/second, while the values for the confined Tertiary aquifer in the Lower Sand Stratum ranged from 1.3×10^{-4} to 7.5×10^{-4} cm/sec.

Due to groundwater levels that would be higher than the depth of planned excavations at the site, the applicant described its plans to temporarily dewater the excavations that extended below the water table during construction of the new units, and further stated that the dewatering would be performed in a manner that minimized the effects of drawdown on the environment and the operating units. The applicant expected the drawdown effects would be limited to the VEGP site and would have only a negligible effect on the existing Units 1 and 2.

The design groundwater level for VEGP Units 3 and 4 was at El. 50 m (165 ft) msl based on the results of ten years of groundwater monitoring prior to and during the ESP subsurface investigation. The El. 50 m (165 ft) msl level also corresponded to the design groundwater level for the existing VEGP Units 1 and 2, and the applicant based the static stability of the proposed structures on this design groundwater level.

In support of the LWA request, the applicant provided the following information:

Construction Dewatering

Due to the relatively impermeable nature of the Upper Sand Stratum and underlying Blue Bluff Marl, the applicant concluded that sumps and pumps would be sufficient for construction dewatering, and dewatering would be accomplished using gravity-type systems for sump-pumping of ditches that would advance below the progressing excavation grade. SSAR Subsection 2.5.4.6.2 also describes the dewatering methods used during construction of Units 1 and 2, which included a series of ditches oriented in an east-west direction and connected by a

north-south ditch that drained to a sump equipped with four high-volume pumps. The applicant stated that the dewatering plans for Units 3 and 4 would use similar methods.

2.5.4.1.7 Response of Soil and Rock to Dynamic Loading

SSAR Section 2.5.4.7 describes the applicant's estimates of the amplification and attenuation of the seismic acceleration at sound bedrock through the soil and rock column. The applicant stated that it compiled data from shear wave velocity profiles of soils and rock, variations of the shear modulus and damping values of soils with strain, and site-specific seismic acceleration-time history, all analyzed using an appropriate computer program.

In support of the ESP application, the applicant provided the following information:

Shear Wave Velocity Profile

SSAR Subsection 2.5.4.7.1 describes the shear wave velocity profiles developed for both soil and rock in the site area.

1. Soil Shear Wave Velocity Profile. During the ESP investigation, the applicant collected a variety of measurements to obtain estimates of shear wave velocity in the soil, estimates that were later confirmed during the COL investigation. The applicant used P-S velocity and CPT down-hole seismic testing to measure the shear wave velocity as part of the ESP subsurface investigations. The applicant developed the shear wave velocity profile used in the seismic amplification/attenuation analysis from the ESP investigation, shown on SSAR Figure 2.5.4-7, and the soil profile used consists of compacted backfill from 0 to 26 m (86 ft), Blue Bluff Marl from 26 to 45.5 m (86 to 149 ft), Lower Sand Stratum from 45.5 to 320 m (149 to 1,049 ft), and Dunbarton Triassic Basin and Paleozoic Crystalline Rock below 320 m (1,049 ft).

   The applicant stated that when compared, the profile of the combined data set (COL) in the middle and upper portions of the Blue Bluff Marl was in good agreement with the ESP profile, although, in the lower portions of the Blue Bluff Marl and the Lower Sand Stratum, the COL profile exhibited slightly lower shear wave velocity values than in the ESP profile. The applicant concluded that the COL shear wave velocity generally increased with depth and supported the findings of the ESP.

2. Rock Shear Wave Velocity Profile. SSAR Subsection 2.5.4.7.1.2 states that due to the thickness of sediments at the VEGP site, the applicant needs to know the shear wave velocity profile and material properties for the site down to the depth where the material shear wave velocity is approximately 2804 m/s (9,200 fps). Since the site is underlain by both the Triassic Basin and Paleozoic crystalline rocks, the applicant considered the effect of shear wave velocities and the material properties of both rocks and their geometries. The applicant concluded that shear wave velocities measured at the top of the Triassic Basin, including the weathered portion, did not reach 2,804 m/s (9,200 fps). The applicant then compared deep borehole shear wave velocity data available from the Savannah River Site (SRS) with data from borehole B-1003 to determine the character of the rock shear wave in the Triassic Basin. The applicant concluded that a weathered zone 61 m (200 ft) thick was

present at the top of the Triassic Basin, characterized by the shear wave velocity rapidly increasing with depth to a point where there was a relatively high shear wave velocity, but still less than 2,804 m/s (9,200 fps). The applicant observed a gentler shear wave velocity gradient increasing with depth below the weathered zone. Finally, the applicant noted an arrangement of gentle gradients and shear wave velocities at the top of the unweathered Triassic basin that was interpreted as a continuation of the site-specific profile from borehole B-1003.

After considering data suggesting that the non-capable Pen Branch fault separated the Triassic Basin from the Paleozoic crystalline rocks, as well as the structural geometry of the rock units and the fault, and the velocity profiles from SRS investigations, the applicant stated the shear wave velocity profile through the Triassic Basin probably would not reach 2,804 m/s (9,200 fps) before encountering the Paleozoic crystalline rock, where the shear wave velocity was interpreted as at least 2,804 m/s (9,200 fps). Accounting for the variability of the depth where the Paleozoic crystalline rock was encountered and the uncertainty of the shear wave velocity gradient, the applicant considered six rock shear wave velocity profiles to comprise the base case used in the seismic amplification and attenuation analysis. The applicant also considered the deep boring rock shear wave velocities from three SRS locations, velocities that suggested additional geometries for the shear wave velocity profiles of the Triassic Basin and the Paleozoic crystalline rock that could impact site response. A closer inspection of the shear wave velocity profile from three SRS locations suggested there was a low velocity zone at the bottom of the Triassic basin where the Pen Branch fault was encountered. The applicant determined through sensitivity analyses that the alternate shear wave velocity models suggested by these observations resulted in insignificant variations in the site response relative to the six profiles previously considered.

Variation of Shear Modulus and Damping with Shear Strain

SSAR Subsection 2.5.4.7.2 describes the variations of the shear modulus and damping with shear strain for both the ESP and COL analyses. Site-specific shear modulus and damping curves are presented as Figures 2.5.4-6 and 2.5.4-7 of this SER.

1. Shear Modulus (ESP Analysis). SSAR Subsection 2.5.4.7.2.1.1 describes the variation of shear modulus with shear strain as determined during the ESP analysis at the VEGP site. The applicant derived the shear modulus from the unit weight data and shear wave velocity of the soil, the determination of which was described in SSAR 2.5.4.7.1. Using the SHAKE2000 (Bechtel 2000) analysis, the applicant tabulated values for shear modulus, as well as the low strain values for the existing soils and rock and for compacted backfill as shown in Tables 2.5.4-1 and 2.5.4-3 of this SER, respectively. The applicant also used the EPRI curves for sands and clays (EPRI TR-102293 1993) to derive the dynamic shear modulus reduction in terms of depth for granular soils (Upper and Lower Sand Stratum) and in terms of the Plasticity Index (PI) for cohesive soils (Blue Bluff Marl) using a PI of 25 percent for the clay of the Lisbon Formation. Table 2.5.4-4 of this SER provides the results of the shear modulus reduction factors. The applicant also used the shear modulus reduction factors developed for the neighboring SRS, selected based on their stratigraphic relationship to the site of VEGP Units 3 and 4, for the ESP analysis. The applicant equally

weighted the site amplification factors using the EPRI and SRS shear modulus degradation relationships as described in SSAR Subsection 2.5.2.5.1.2.1.

2. Shear Modulus (COL Analysis). SSAR Subsection 2.5.4.7.2.1.2 describes the development of site-specific dynamic shear modulus reduction curves using RCTS test results from the Blue Bluff Marl, Lower Sand Stratum, and the proposed borrow materials for the compacted backfill. The applicant tested undisturbed samples from both the Blue Bluff Marl and Lower Sand Stratum, plotted the shear modulus reduction data against shearing strain, and overlaid the data on the EPRI curves for clay or depth of granular soils. The applicant stated that for the Blue Bluff Marl, the site-specific data followed the EPRI trend of the relationship with plasticity index, while the Lower Sand Stratum followed the EPRI trend for depth for granular soils.

3. Damping (ESP Analysis). SSAR Subsection 2.5.4.7.2.2.1 describes the derivation of the damping ratio from EPRI in terms of depth for granular soils, such as the Upper and Lower Sand Strata, and in terms of Plasticity Index for cohesive soils, such as the Blue Bluff Marl, as conducted as part of the ESP site analysis. The applicant used the EPRI curves for sands to derive the damping ratios for the granular soil strata (compacted backfill and Lower Sand Stratum), and the EPRI curves for clays to derive the damping ratios for the Lisbon Formation using a PI of 25 percent. SER Table 2.5.4-4 provides the calculated damping ratios. The applicant also used certain damping ratio values developed for the SRS, selected based on their stratigraphic relationship to the VEGP site. The applicant stated that it weighted the mean site reduction and site amplification factors using EPRI and SRS shear modulus degradation relationships.

4. Damping (COL Analysis). SSAR Subsection 2.5.4.7.2.2.2 describes the development of the site-specific damping curves from the RCTS test results performed on samples from the Blue Bluff Marl, the Lower Sand Stratum, and the proposed borrow materials for compacted backfill. The applicant stated that it plotted the RCTS damping relationships for the Blue Bluff Marl samples, which were then overlain on the EPRI curves for clay, and it concluded that the site-specific data followed trends that were consistent with the EPRI damping relationships for PI. The applicant also derived site-specific curves for low and high PI materials based on the similarity of the EPRI PI curves. Utilizing similar plots and overlays for the Lower Sand Stratum and clayey samples, the applicant concluded that the site-specific data for both the sand and clay samples followed trends consistent with the EPRI relationships for depth for granular soils and were based on the EPRI curves for depth of granular soils.

Table 2.5.4-3 Design Dynamic Shear Modulus and Typical Shear Wave Velocity from ESP Investigations (Taken from SSAR Tables 2.5.4-2 and 2.5.4-6)

Geologic Formation	Depth m (ft)	Elevation m (ft)	Gmax (ksf)	Vs (fps)
Upper Sand Stratum (Barnwell Group)	0 to 4.8 (0 to 16)	68 to 63 (223 to 207)	7,000	1,400
	4.8 to 12.5 (16 to 41)	63 to 55.4 (207 to 182)	2,286	800
	12.5 to 17.7 (41 to 58)	55.4 to 50.2 (182 to 165)	2,580	850
	17.7 to 26.2 (58 to 86)	50.2 to 41.7 (165 to 137)	2,893	900
Blue Bluff Marl (Lisbon Formation)	26.2 to 28 (86 to 92)	41.7 to 40 (137 to 131)	6,978	1,400
	28 to 29.5 (92 to 97)	40 to 38.4 (131 to 126)	10,321	1,700
	29.5 to 31 (97 to 102)	38.4 to 36.8 (126 to 121)	15,750	2,100
	31 to 32 (102 to 105)	36.8 to 35.9 (121 to 118)	10,321	1,700
	32 to 33.8 (105 to 111)	35.9 to 34.1 (118 to 112)	17,286	2,200
	33.8 to 37.5 (111 to 123)	34.1 to 30.5 (112 to 100)	19,723	2,350
	37.5 to 45.4 (123 to 149)	30.5 to 22.5 (100 to 74)	25,080	2,650
Lower Sand Stratum	45.4 to 47.5 (149 to 156)	22.5 to 20.4 (74 to 67)	14,286	2,000
Still Branch	47.5 to 65.8 (156 to 216)	20.4 to 2.1 (67 to 7)	9,723	1,650
Congaree	65.8 to 101 (216 to 331)	2.1 to -32.9 (7 to -108)	13,580	1,950
Snapp	101 to 134 (331 to 438)	-32.9 to -65.5 (-108 to -215)	15,009	2,050
Black Mingo	134 to 145 (438 to 477)	-65.5 to -77.4 (-215 to -254)	19,723	2,350
Steel Creek	145 to 179 (477 to 587)	-77.4 to -111 (-254 to -364)	25,080	2,650
Gaillard/Black Creek	179 to 243 (587 to 798)	-111 to -175 (-364 to -575)	29,009	2,850
Pio Nino	243 to 262 (798 to 858)	-175 to -193 (-575 to -635)	29,418	2,870
Cape Fear	262 to 320 (858 to 1,049)	-193 to -251 (-635 to -826)	26,229	2,710
Dunbarton Triassic Basin	320 (1,049)	-251 (-826)		2,710
	333 (1,093)	-265 (-870)		5,300
	403 (1,323)	-335 (-1,100)		7,800

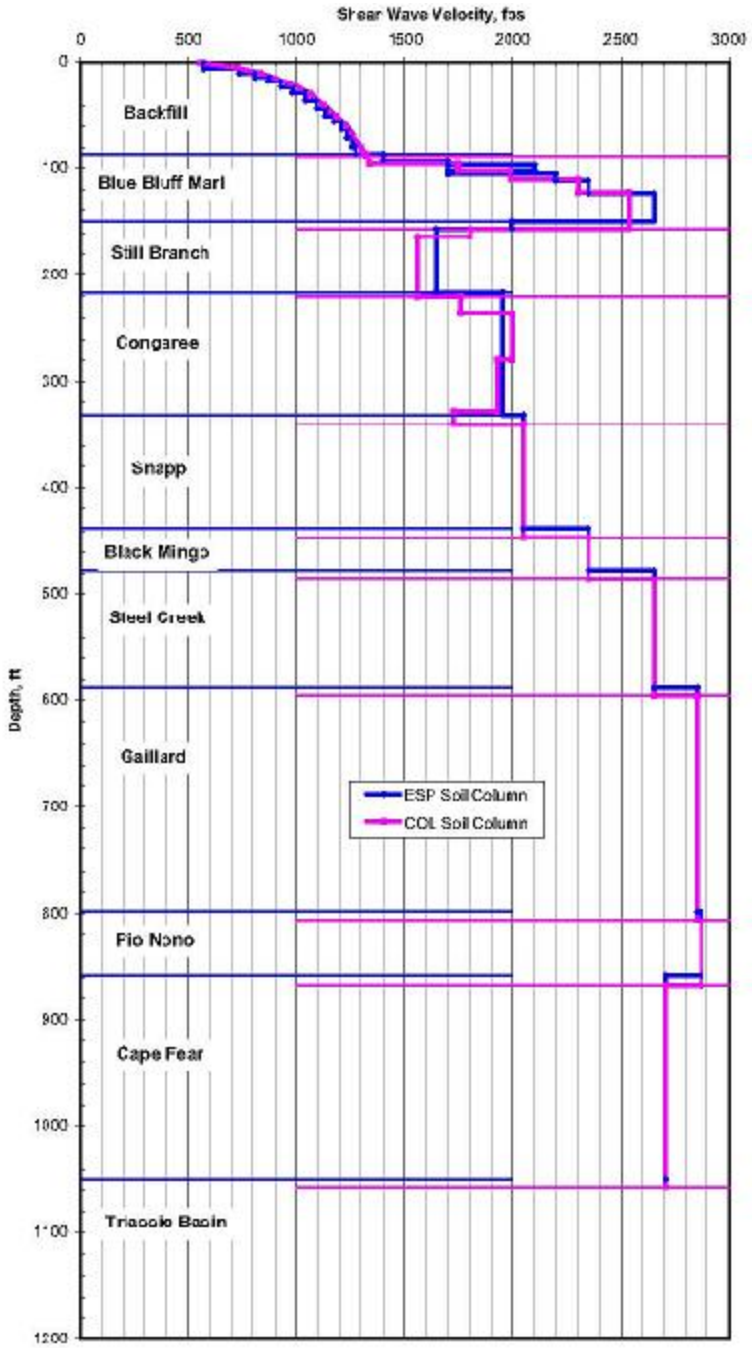


Figure 2.5.4-5 Shear Wave Velocity Profile – ESP and COL Soil Column (SSAR Figure 2.5.4-7a)

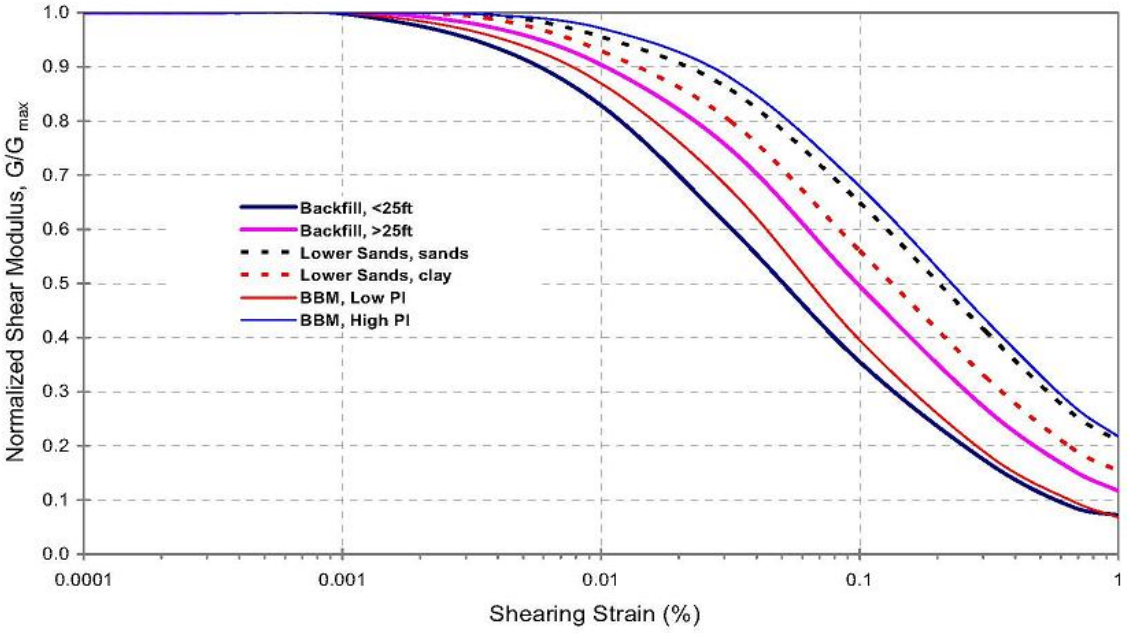


Figure 2.5.4-6 Site-Specific Shear Modulus Reduction Curves (SSAR Figure 2.5.4-9a)

Soil/Rock Amplification/Attenuation Analysis

SSAR Subsection 2.5.4.7.3 describes the use of the SHAKE2000 computer program to determine the site dynamic responses for the soil and rock profiles. The applicant stated that SHAKE2000 used an equivalent linear procedure to account for the non-linearity of the soil by employing an iterative procedure to obtain values for shear modulus and damping that were compatible with the equivalent uniform strain induced in each sublayer. At the beginning of the analysis, the applicant assigned a set of shear modulus and damping value properties to each sub-layer of the soil profile, properties which were used during the analysis to calculate the shear strain induced in each sub-layer. The applicant then modified the shear modulus and damping ratio for each sub-layer based on the shear modulus and the damping ratio versus strain relationships, repeating the analysis until strain-compatible modulus and damping values were achieved.

Comparison of ESP versus COL Soil Column

SSAR Subsection 2.5.4.7.5 compares the subsurface data collected and evaluated during two distinct phases referred to as the ESP and COL investigations, including Phase 1 of the test pad program. The applicant described the ESP investigation as limited in scope but broad in aerial coverage, whereas the COL investigation was extensive in scope but limited to the Units 3 and 4 power block areas. SER Figure 2.5.4-5 presents the stratification and shear wave velocity profiles of the ESP and COL soil columns. The applicant stated that the offset in the soil stratification between the soil columns reflected refinements due to the additional data collected during the COL investigation. The applicant concluded that a comparison of the ESP and COL shear wave velocity profiles indicated good agreement between the data sets and consistency of trends within the strata.

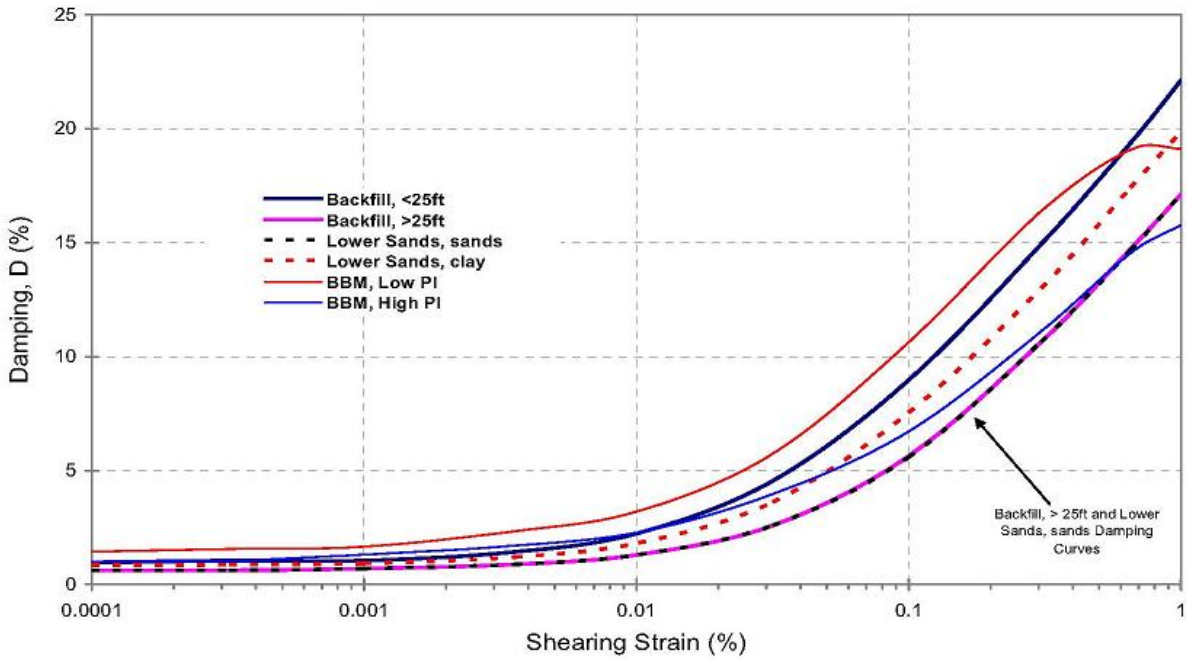


Figure 2.5.4-7 Site-Specific Damping Ratio Curves (SSAR Figure 2.5.4-11a)

In support of the LWA request, the applicant submitted the following information:

Shear Wave Velocity Profile

Soil Shear Wave Velocity. The applicant collected shear wave velocity data from the ESP and COL investigations, and it stated that the ESP data was derived from the backfill shear wave velocity data determined during the previous investigations conducted for VEGP Units 1 and 2, while the COL investigations considered the shear wave velocity data determined for the structural backfill to be used at the VEGP Units 3 and 4 site.

The applicant measured shear wave velocity in the field by the applicant during Phase 1 of the test pad program, as well as through RCTS and other methods from the COL investigations. The applicant used this data, along with laboratory test data, to evaluate the shear wave velocity of the backfill and develop the shear wave velocity profile for the backfill. During the COL investigation, the applicant measured shear wave velocity values from 0 to 27 m (88 ft) in the backfill, 27 to 47.5 m (88 to 156 ft) in the Blue Bluff Marl, 47.5 to 322 m (156 to 1,058 ft) in the Lower Sand Stratum, including the Still Branch, Congaree, and Snapp Formations, and in the Dunbarton Triassic Basin and Paleozoic crystalline rock below 322 m (1,058 ft). The applicant stated that it combined and averaged the data from the six COL profiles and two ESP data profiles to produce SSAR Figure 2.5.4-7a (reproduced as SER Figure 2.5.4-5), an average shear wave velocity profile for the data. The applicant stated that the figure illustrates the relationship and similarity between the ESP and COL data sets.

Table 2.5.4-4 Summary of Site-specific Modulus Reduction and Damping Ratio Values

Stratum	Backfill				Blue Bluff Marl				Lower Sands			
Sub strata	< 7.6 m (25 ft)		> 7.6 m (25 ft)		Low PI		High PI		Sands		Clay (Congaree/Snapp)	
Shear Strain (%)	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio
0.00010	1	0.97	1	0.62	1	1.44	1	1	1	0.62	1	0.86
0.00032	1	1.05	1	0.62	1	1.56	1	1.05	1	0.62	1	0.87
0.00100	0.998	1.05	1.003	0.7	1	1.67	1	1.32	1.001	0.7	1.001	0.93
0.00359	0.942	1.44	0.975	0.89	0.96	2.34	0.9965	1.71	0.997	0.89	0.99	1.21
0.01019	0.826	2.26	0.902	1.3	0.867	3.23	0.97	2.3	0.954	1.32	0.928	1.8
0.03170	0.603	4.55	0.748	2.6	0.673	5.75	0.88	3.97	0.828	2.6	0.8	3.62
0.10000	0.355	8.97	0.495	5.64	0.395	10.63	0.679	6.715	0.649	5.59	0.56	7.54
0.30690	0.172	14.94	0.269	10.65	0.187	16.39	0.433	11.115	0.411	10.65	0.327	13
0.65313	0.089	19.38	0.158	14.73	0.1	19.08	0.2785	14.545	0.263	14.68	0.198	17.42
1.00000	0.072	22.12	0.117	17.11	0.068	19.12	0.217	15.77	0.209	17.11	0.154	19.87

Variation of Shear Modulus and Damping with Shear Strain

1. Shear Modulus (COL Analysis). In addition to the information summarized from this section in support of the ESP application, the applicant also included the following information in support of the LWA request. SSAR Subsection 2.5.4.7.2.1.2 describes the variation of shear modulus with shear strain as determined during the COL analysis at the VEGP site. The applicant developed the site-specific dynamic shear modulus reduction curves from the results of RCTS tests on Blue Bluff Marl and Lower Sand strata samples, as well as on samples from the proposed borrow materials. As part of the COL analysis, the applicant also tested five bulk soil samples from test pits in the proposed borrow sources. The tests conducted by the applicant included percent fines (8 to 25 percent), moisture-density and index testing on the samples. The applicant stated that RCTS tests were performed on the bulk samples at two different levels of compaction (at 95 percent and 97 percent, or at 95 percent and 100 percent), using confining pressures based on representative depths throughout the proposed 27 m (90 ft) backfill soil column. The applicant concluded that the results disclosed little variation based on the level of compaction. The applicant then plotted the shear modulus reduction data against shearing strain, overlaid the data on the EPRI curves for depth for granular soils, and concluded that the site-specific data followed trends consistent with the EPRI relationships for depth for granular soils.

2. Damping (COL Analysis). In addition to the information summarized from this section in support of the ESP application, the applicant also included the following information in support of the LWA request. SSAR Subsection 2.5.4.7.2.2.2 describes the development of the site-specific damping curves from the RCTS test results performed on samples from the Blue Bluff Marl, the Lower Sand Stratum, and the proposed borrow materials for compacted backfill. The applicant stated that it developed site-specific damping curves for the borrow material for samples under low confining pressure (less than 7.5 m (25 ft) deep) and for samples under higher confining pressures (more than 7.5 m (25 ft) deep) based on the similarity of the EPRI curves for depth for granular soils.

Two-Dimensional Effects Site Response Analysis (Bathtub Model)

SSAR Subsection 2.5.4.7.4 states that the model for the site dynamic response analysis, as discussed in SSAR Section 2.5.2.5, depicting the backfill above the Blue Bluff Marl as a continuum, did not account for the extent of the excavation and backfill or any impacts of the Upper Sand Stratum on site response. Therefore, the applicant stated that it evaluated these impacts by considering the site response with both the Upper Sand Stratum in place and replaced by backfill. According to the applicant, the average shear wave profile of the stratum was developed and used to characterize shear wave velocity of the Upper Sand. The applicant provided a more detailed discussion of these analyses and results in SSAR Section 2.5.2.9.2.

2.5.4.1.8 Liquefaction Potential

SSAR Section 2.5.4.8 describes soil liquefaction as the process where loose, saturated, granular deposits lose a significant portion of their shear strength due to the buildup of pore pressure as a result of cyclic loading such as that caused by an earthquake. The applicant stated that multiple factors contributed to liquefaction potential, including geologic age, state of soil saturation, density, grain size distribution, plasticity, and intensity and duration of earthquakes. The applicant stated that, in general, when the following criteria are met,

liquefaction can occur: 1) the design ground acceleration is high, 2) the soil is saturated (i.e., the soil is close to or below the water table), and 3) the site soils are sands or silty sands in a loose or medium dense condition.

In support of the ESP application, the applicant submitted the following information:

At the VEGP site, the applicant identified the Upper Sand Stratum, consisting of sands of varying fines content, as meeting all three criteria. According to the applicant, liquefaction was not a concern in either the Blue Bluff Marl or the Lower Sand Stratum, although the applicant addressed the liquefaction potential of the coarse-grained materials within the Blue Bluff Marl. Due to the potential susceptibility of the Upper Sand Stratum to liquefaction, the applicant completely removed the entire potion of the Upper Sand Stratum during construction of VEGP Units 1 and 2, and replaced it with engineered backfill. The applicant stated that it planned for a similar removal and replacement procedure during construction of VEGP Units 3 and 4.

Acceptable Factor of Safety Against Liquefaction

The applicant used Regulatory Guide 1.198 (RG1.198) as a guide for liquefaction analysis. RG 1.198 considers factors of safety (FS) less than or equal to 1.1 against liquefaction to be low, FS between 1.1 and 1.4 to be moderate, and FS equal to or greater than 1.4 to be high.

Previous Liquefaction Analyses

SSAR Subsection 2.5.4.8.2 describes the applicant's evaluation of the liquefaction potential of the Upper Sand Stratum performed during the VEGP Units 1 and 2 investigations. The applicant determined that the Upper Sand Stratum below the groundwater table was susceptible to liquefaction when it was subjected to the maximum SSE acceleration of 0.2g developed for Units 1 and 2. To account for this potential, the applicant removed the Upper Sand Stratum to an approximate El. of 39.5 to 41 m (130 to 135 ft) in the Units 1 and 2 power block area and replaced it with compacted structural backfill. The applicant evaluated, using cyclic strength data from test specimens, the liquefaction potential of the compacted structural backfill in the power block area and determined an FS against liquefaction of 1.9 to 2.0. The applicant concluded that this was an adequate factor of safety against liquefaction for the compacted backfill for VEGP Units 1 and 2.

Liquefaction Analyses Performed for the ESP Application

SSAR Subsection 2.5.4.8.3 describes the liquefaction analyses performed for the strata at the VEGP site as part of the ESP application, including the Upper Sand, Blue Bluff Marl, and compacted backfill.

1. Liquefaction Analyses of the Upper Sands. Based on the previous investigations and excavations for VEGP Units 1 and 2, as well as on the proximity of proposed Units 3 and 4, the applicant stated that it did not perform a liquefaction study as part of the ESP investigation because the unit would be completely removed and replaced with select compacted non-liquefiable structural backfill up to plant grade within the footprint of the power block.

2. Liquefaction Analyses of the Blue Bluff Marl. The applicant identified the Blue Bluff Marl as a cemented, overconsolidated, calcareous, fine-grained silt and clay material that exhibited a high factor of safety against liquefaction; however, the applicant stated that since it found

some lenses of silty fine sand during the COL investigation, additional analyses were performed. The applicant stated that it evaluated the data from SPT, CPT, and shear wave velocity measurements, with the SPT measurement method being the most well developed and well recognized. The applicant calculated the cyclic stress ratio (CSR), a measure of the stress imparted to the soils by the ground motion; then the cyclic resistance ratio (CRR), a measure of the resistance of soils to the ground motion; and finally used the ratio of the CRR to the CSR to determine the FS.

a. Liquefaction Potential Based on SPT Data. The applicant presented SPT N60-values versus elevation for the 70 COL investigation borings in the VEGP Units 3 and 4 power block area and stated that the results were indicative of non-liquefiable coarse-grained soil samples. The applicant stated that of eight soil samples it analyzed, three were potentially liquefiable, with calculated FSs against liquefaction of 1.43, 1.75, and 2.19, and in all cases, greater than 1.1. Therefore, the applicant concluded the FS against liquefaction in the Blue Bluff Marl was adequate based on the SPT data.

b. Liquefaction Potential Based on Shear Wave Velocity Data. The applicant stated that it measured shear wave velocity (Vs) data in the Blue Bluff Marl by P-S logging in six power block area borings during the COL investigation to evaluate the potential for liquefaction. Following the recommendations in Youd et al, the applicant stated that it corrected the shear wave velocity values for overburden (Vs1), and calculated Vs1 values from 253 to 508 m/s (830 to 1666 fps). Based on the relationship between Vs1, cyclic resistance ratio (CRR), and liquefaction presented by Youd et al., the applicant concluded that the Blue Bluff Marl was non-liquefiable.

Liquefaction Conclusions

Based on its analysis of the potential for liquefaction, the applicant concluded that the only potentially liquefiable rock was the portion of the Upper Sand Stratum below the groundwater table. The applicant stated that for this reason, the Upper Sand Stratum was removed and replaced with compacted structural backfill during construction of Units 1 and 2 and that the same would be done during construction of Units 3 and 4. Through various analyses, the applicant concluded that the liquefaction potential of the compacted structural backfill material, consisting of materials and using methods similar to those for VEGP Units 1 and 2, was not a concern. Finally, the applicant determined that the FS against liquefaction of the Blue Bluff Marl (greater than 1.1) was adequate.

In support of the LWA request, the applicant provided the following information:

Liquefaction Analyses of the Compacted Backfill. In SSAR Subsection 2.5.4.8.3.3, the applicant stated that the structural backfill would be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 1557, and that the backfill materials, construction, and field compaction methods would be consistent with those used during construction of Units 1 and 2. The applicant evaluated the properties of backfill from the proposed borrow sources during Phase 1 of the test pad program through field and laboratory testing of the materials, and by consistent comparison with results from Units 1 and 2, and concluded that for the design basis earthquake, liquefaction was not a concern for the compacted backfill at Units 3 and 4.

2.5.4.1.9 Earthquake Design Basis

SSAR Sections 2.5.2.6 and 2.5.2.7 discuss in detail the Safe Shutdown Earthquake (SSE). SSAR Section 2.5.2.8 discusses the Operating Basis Earthquake (OBE).

2.5.4.1.10 Static Stability

In support of the ESP application, the applicant submitted the following information:

SSAR Section 2.5.4.10 describes the two scenarios used for the bearing capacity and settlement analyses for VEGP Units 3 and 4. The first scenario, as identified by the applicant, was the Containment and Auxiliary Building foundations, which would be constructed at about El. 55 m (180 ft) msl, a level that corresponded to a depth of 12 m (40 ft) below the final grade of El. 67 m (220 ft) msl, and 15 to 18 m (50 to 60 ft) above the top of Blue Bluff Marl bearing stratum based on the ESP subsurface investigation. The second scenario was the construction of the other foundations in the power block area, which the applicant stated would be placed at depths of about 1.2 m (4 ft) below final grade. Based on the results of the ESP and COL investigations and Phase I of the test pad program, the applicant determined that the soils supporting the nuclear island did not exhibit extreme variations in subgrade stiffness and considered the site to be uniform.

Bearing Capacity

For calculation purposes, the applicant modeled the containment building mat as a circle with a diameter of about 43 m (142 ft) placed at a depth of 12 m (39.5 ft) below finished grade, while other structures would be founded at an approximate depth of 1.2 m (4 ft) below grade. The applicant assumed that all structures in the power block area would be founded on a 27 meter (90 feet) thick layer of structural backfill compacted to a minimum of 95 percent.

Settlement Analysis

The applicant noted that, based on previous site experiences, the total settlement for large mat foundations that support major power plant structures can exceed the limit of 5.08 cm (2 inches) suggested in geotechnical literature. The applicant stated that the settlements of VEGP Units 1 and 2 foundations were from 6.8 to 8.1 cm (2.7 to 3.2 in) for containment buildings, 2.8 to 4.8 cm (2.7 to 3.2 in) for the control building, 7.4 to 8.4 cm (2.9 to 3.3 in) for the auxiliary building, and 6.35 to 9.1 cm (2.5 to 3.6 in) for the cooling towers, all of which were significantly below the maximum design values. The applicant also provided the ratio of measured to predicted settlement for these structures, which ranged from less than 0.50 to about 0.75, which indicated that the subsurface soils were stiffer than anticipated.

The applicant also acknowledged that differential settlement between buildings could affect the pipe connections between those buildings, and therefore it measured differential settlements between the basemats of Units 1 and 2 and reported that they were generally within the limit of 1.9 cm (0.75 in) suggested in geotechnical literature and smaller than the design limit. The applicant noted that the settlements were essentially elastic in that they took place during construction of the units and reflected the elastic nature of the compacted backfill, the heavily overconsolidated Blue Bluff Marl, and the underlying Lower Sand Stratum. The results of laboratory consolidation tests that the applicant conducted on relatively undisturbed samples

from the Blue Bluff Marl and Lower Sand Strata confirmed that the elastic behavior and very stiff and dense nature of the strata. Furthermore, the applicant confirmed the very dense nature and the expected performance under load of compacted backfill would be similar to VEGP Units 1 and 2 based on the results from the test pad program. The applicant concluded that settlement could be limited to one inch while differential settlement between footings could be limited to 1.27 cm (½ inch) for footings supporting smaller structures. As an additional strategy, the applicant planned to install piping as late in the construction schedule as practicable and install pipe supports only when construction of the structure to which the pipe connected was near completion.

> Displacement Monitoring. The applicant described plans to develop a detailed instrumentation plan to monitor heave in subsurface soils due to excavation, changes in pore pressures due to excavation and dewatering, and settlement due to construction of the structures. This plan will also include displacement monitoring at depth in order to estimate and confirm moduli of the subsurface soils. The applicant stated that instrumentation would be regularly monitored, including conventional survey, electronic instrumentation, and remote telemetry, where practical. Finally, the applicant stated its intention to place particular emphasis on differential movement and structure tilt. The applicant will develop the plan prior to construction activities.

In support of the LWA request, the applicant provided the following information:

The applicant stated that an earthwork specification for compacted backfill would be developed after Phase 2 of the test pad program was completed. The Phase II test pad program was completed by the applicant in July 2008 and the results used by the applicant to develop draft construction specifications and structural backfill placement procedures. The applicant stated that its final soils specification and backfill implementing procedures are to be finalized in accordance with its quality program, which would be approved as part of the LWA request, prior to the start of any construction activities authorized by the LWA. The applicant also stated that a coefficient of friction of 0.45 against the concrete foundation for the proposed sand and silty sand compacted backfill materials was expected to be achieved, and a site-specific evaluation was conducted and presented by the applicant in Appendix 2.5E of the revised SSAR. The staff's evaluation of the coefficient of friction against sliding is discussed in SER Section 3.8.

Bearing Capacity

Allowable static bearing capacity values were based on Terzaghi's bearing capacity equations using an internal angle of friction of 36 degrees for the compacted backfill as developed by the applicant from field and laboratory testing of the borrow materials during the COL investigation and Phase 1 of the test pad program. With an FS of 3.0, the applicant determined that the site conditions provided an allowable bearing pressure of 1,627 kPa (34 ksf) under static loading conditions for the containment and auxiliary buildings. The allowable bearing capacity values for foundations placed on compacted fills at depths of about 1.2 m (4 ft) below finished grade are shown on SSAR Figure 2.5.4-13.

The applicant also evaluated the allowable bearing capacity of the containment and auxiliary buildings under dynamic loading conditions, again basing its analysis methods on Terzaghi's bearing capacity equation for general shear using seismic bearing capacity factors and equations for local shear. With an FS of 2.25, the applicant concluded that site conditions provided an allowable bearing pressure of 2,011 kPa (42,000 psf) under dynamic loading conditions.

Settlement Analysis

The applicant performed a detailed settlement analysis for VEGP Units 3 and 4 using elastic properties similar to those used in the analysis for VEGP Units 1 and 2. In the analysis, the applicant incorporated excavation, dewatering, and a timeline of construction to estimate basemat displacement time histories. According to the applicant, the results of the analysis indicated that for the assumed loads, the predicted total settlements ranged from about 5.08 to 7.62 cm (2 to 3 in), with a tilt of approximately 0.63 cm (¼ in) in 15 m (50 ft), a differential settlement between structures of less than 2.54 cm (1 in), and the predicted heave due to foundation excavation ranged from about 2.54 to 6.35 cm (1 to 2 ½ in). The applicant noted that the results were similar to the movements measured for Units 1 and 2.

2.5.4.1.11 Design Criteria

SSAR Section 2.5.4.11 summarizes the design criteria provided in the AP1000 DCD, Revision 15, and covered in various sections of the SSAR. The applicant summarized the geotechnical criteria, except for the criteria that pertain to structural design (e.g., wall rotation, sliding, or overturning), which is discussed in Section 3.8 of this SER. As noted by the applicant in SSAR Section 2.5.4.8, the acceptable factor of safety (FS) against liquefaction of site soils was greater than or equal to 1.1. SSAR Section 2.5.4.10 specifies bearing capacity criteria, including the minimum FS of 3 when applied to bearing capacity equations and against breakout failure due to uplift on buried piping. For soils, an FS of 2.25 can be used when dynamic or transient loading conditions apply. SSAR Section 2.5.5.2 specifies that the minimum acceptable long-term static FS against slope stability failure is 1.5. SSAR Section 2.5.5.3 states that the minimum acceptable long-term seismic FS against slope stability failure is 1.1.

2.5.4.1.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 describes the techniques employed by the applicant to improve the subsurface conditions. For the ESP and COL investigations, the applicant did not consider any ground improvement techniques beyond the removal and replacement of the Upper Sand Stratum, while the test pad program defined the materials and methods for the backfill that would replace the Upper Sand Stratum. The applicant also described plans to improve surficial areas outside the power block excavation through densification with heavy vibratory rollers, and other ground improvement methods, such as the use of piles, as warranted.

2.5.4.2 Regulatory Basis

The applicable regulatory requirements for reviewing the applicant's discussion of stability of subsurface materials and foundations are:

1. 10 CFR 50.55a, "Codes and Standards," requires that structures, systems, and components be designed, fabricated, erected, constructed, tested and inspected to quality standards commensurate with the importance of the safety function to be performed.

2. 10 CFR Part 50, Appendix A, General Design Criterion 1 (GDC 1), "Quality Standards and Records," requires that structures, systems and components important to safety be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. It also requires that appropriate records

of the design, fabrication, erection, and testing of structures, systems, and components important to safety be maintained by or under the control of the nuclear power unit licensee throughout the life of the unit.

3. 10 CFR Part 50, Appendix A, General Design Criterion 2 (GDC 2), "Design Bases for Protection Against Natural Phenomena," as it relates to consideration of the most severe of the natural phenomena that have been historically reported for the site and surrounding area, with sufficient margin for the limited accuracy, quantity, and period of time in which the historical data have been accumulated.

4. 10 CFR Part 50, Appendix B, "Quality Assurance Criteria for Nuclear Power Plants and Fuel Processing Plants," establishes quality assurance requirements for the design, construction, and operation of those structures, systems, and components of nuclear power plants that prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.

5. 10 CFR Part 50, Appendix S, "Earthquake Engineering Criteria for Nuclear Power Plants," as it applies to the design of nuclear power plant structures, systems, and components important to safety to withstand the effects of earthquakes.

6. 10 CFR Part 100, "Reactor Site Criteria," provides the criteria that guide the evaluation of the suitability of proposed sites for nuclear power and testing reactors.

7. 10 CFR 100.23, "Geologic and Seismic Criteria," provides the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

The related acceptance criteria are described in SRP Section 2.5.4:

1. Geologic Features: In meeting the requirements of 10 CFR Parts 50 and 100, the section defining geologic features is acceptable if the discussions, maps, and profiles of the site stratigraphy, lithology, structural geology, geologic history, and engineering geology are complete and are supported by site investigations sufficiently detailed to obtain an unambiguous representation of the geology.

2. Properties of Subsurface Materials: In meeting the requirements of 10 CFR Parts 50 and 100, the description of properties of underlying materials is considered acceptable if state-of-the-art methods are used to determine the static and dynamic engineering properties of all foundation soils and rocks in the site area.

3. Foundation Interfaces: In meeting the requirements of 10 CFR Parts 50 and 100, the discussion of the relationship of foundations and underlying materials is acceptable if it includes (1) a plot plan or plans showing the locations of all site explorations, such as borings, trenches, seismic lines, piezometers, geologic profiles, and excavations with the locations of the safety-related facilities superimposed thereon; (2) profiles illustrating the detailed relationship of the foundations of all seismic Category I and other safety-related facilities to the subsurface materials; (3) logs of core borings and test pits; and (4) logs and maps of exploratory trenches in the application for a COL.

4. Geophysical Surveys: In meeting the requirements of 10 CFR 100.23, the presentation of the dynamic characteristics of soil or rock is acceptable if geophysical investigations have been performed at the site and the results obtained therefrom are presented in detail.

5. Excavation and Backfill: In meeting the requirements of 10 CFR Part 50, the presentation of the data concerning excavation, backfill, and earthwork analyses is acceptable if: (1) the sources and quantities of backfill and borrow are identified and are shown to have been adequately investigated by borings, pits, and laboratory property and strength testing (dynamic and static) and these data are included, interpreted, and summarized; (2) the extent (horizontally and vertically) of all Category I excavations, fills, and slopes are clearly shown on plot plans and profiles; (3) compaction specifications and embankment and foundation designs are justified by field and laboratory tests and analyses to ensure stability and reliable performance; (4) the impact of compaction methods are incorporated into the structural design of the plant facilities; (5) quality control methods are discussed and the quality assurance program described and referenced; (6) control of groundwater during excavation to preclude degradation of foundation materials and properties is described and referenced.

6. Ground Water Conditions: In meeting the requirements of 10 CFR Parts 50 and 100, the analysis of groundwater conditions is acceptable if the following are included in this subsection or cross-referenced to the appropriate subsections in SRP Section 2.4 of the SAR: (1) discussion of critical cases of groundwater conditions relative to the foundation settlement and stability of the safety-related facilities of the nuclear power plant; (2) plans for dewatering during construction and the impact of the dewatering on temporary and permanent structures; (3) analysis and interpretation of seepage and potential piping conditions during construction; (4) records of field and laboratory permeability tests as well as dewatering induced settlements; (5) history of groundwater fluctuations as determined by periodic monitoring of 16 local wells and piezometers.

7. Response of Soil and Rock to Dynamic Loading: In meeting the requirements of 10 CFRParts 50 and 100, descriptions of the response of soil and rock to dynamic loading are acceptable if: (1) an investigation has been conducted and discussed to determine the effects of prior earthquakes on the soils and rocks in the vicinity of the site; (2) field seismic surveys (surface refraction and reflection and in-hole and cross-hole seismic explorations) have been accomplished and the data presented and interpreted to develop bounding P and S wave velocity profiles; (3) dynamic tests have been performed in the laboratory on undisturbed samples of the foundation soil and rock sufficient to develop strain-dependent modulus reduction and hysterietic damping properties of the soils and the results included.

8. Liquefaction Potential: In meeting the requirements of 10 CFR Parts 50 and 100, if the foundation materials at the site adjacent to and under Category I structures and facilities are saturated soils and the water table is above bedrock, then an analysis of the liquefaction potential at the site is required.

10. Static Stability: In meeting the requirements of 10 CFR Parts 50 and 100, the discussions of static analyses are acceptable if the stability of all safety-related facilities has been analyzed from a static stability standpoint including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, and lateral loading conditions.

11. Design Criteria: In meeting the requirements of 10 CFR Part 50, the discussion of criteria and design methods is acceptable if the criteria used for the design, the design methods employed, and the factors of safety obtained in the design analyses are described and a list of references presented.

12. Techniques to Improve Subsurface Conditions: In meeting the requirements of 10 CFR Part 50, the discussion of techniques to improve subsurface conditions is acceptable if plans, summaries of specifications, and methods of quality control are described for all techniques to be used to improve foundation conditions (such as grouting, vibroflotation, dental work, rock bolting, or anchors).

In addition, the geologic characteristics should be consistent with appropriate sections from: Regulatory Guide 1.27, "Ultimate Heat Sink for Nuclear Power Plants," Regulatory Guide 1.28, "Quality Assurance Program Requirements (Design and Construction)," Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants," Regulatory Guide 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites," and Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)."

2.5.4.3 Technical Evaluation

This section discusses the staff's evaluation of the geotechnical investigations conducted by the applicant to evaluate the stability and determine the static and dynamic engineering properties of the subsurface materials and foundations at the site of VEGP Units 3 and 4, in particular with respect to the specific LWA activities requested. The applicant presented technical information in SSAR Section 2.5.4 resulting from field and laboratory investigations, data gathered during the ESP phase site investigations, and additional field and laboratory data from a COL level investigation in support of the LWA request. The applicant used the subsurface material properties from its field and laboratory testing to evaluate the site geotechnical conditions and to derive the design values for the ESP, LWA request, and COL application. The staff also identified, summarized and considered the applicant's responses to Requests for Additional Information (RAIs) and Open Items from the SER with Open Items.

2.5.4.3.1 Description of Site Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Section 2.5.1.1 for a description of the regional and site geology. Section 2.5.1.3 of this SER presents the staff's evaluation of the regional and site geology.

2.5.4.3.2 Properties of Subsurface Materials

The staff focused its review of SSAR Section 2.5.4.2 on the applicant's description of the subsurface materials, field investigations and laboratory testing, and the static and dynamic engineering properties of the subsurface materials at the VEGP site. The applicant stated that the soils encountered during the ESP investigation, and during subsequent investigations supporting the LWA request and COL application, constitute alluvial and Coastal Plain deposits and were divided into three groups for stability of subsurface materials and foundation purposes; Group 1, the Upper Sand Stratum or Barnwell Group, which would be removed and replaced with structural backfill; Group 2, the Blue Bluff Marl Bearing Stratum or Lisbon

Formation, which is the load bearing layer at the site; and Group 3, the Lower Sand Stratum, consisting of several formations. The Dunbarton Triassic (206 to 24 million years ago [mya]) basin rock, and the Paleozoic (543 to 248 mya) crystalline rock underlie the soil layers at the site. The applicant determined the static and dynamic properties of the three principal soil groups and compacted structural backfill through field investigations and laboratory testing performed in accordance with RG 1.138. The applicant performed grain size distribution (gradation), Atterberg Limits, natural moisture content, unit weight, and triaxial shear laboratory tests. The applicant concluded that the engineering properties obtained from the subsurface investigations and laboratory testing program were similar to those obtained from the previous VEGP Units 1 and 2 investigations. SSAR Table 2.5.4-1 summarizes the geotechnical features of the strata and their corresponding engineering properties as determined during the aforementioned investigations.

The staff's evaluation of the information provided in support of the ESP application is as follows:

In RAI 2.5.4-1, the staff asked the applicant to clarify the discrepancy in different SSAR sections on the number of borings drilled during the ESP field investigation. The applicant explained in its response that in one section it referred to the total number of borings as 14, which included the two borings without any sampling. In other SSAR sections, the applicant did not include these 2 additional borings. With this clarification, the staff considers RAI 2.5.4-1 resolved.

Geotechnical Parameters of the Lower Sand Stratum and the Blue Bluff Marl

In RAI 2.5.4-3, the staff asked the applicant to provide justification for developing geotechnical parameters for the Blue Bluff Marl and Lower Sand Stratum (the main load-bearing layers) using only the data from four borings with no significant sampling in the Lower Sand Stratum. In its response, the applicant stated that three ESP borings completely penetrated the Blue Bluff Marl and another nine borings extended partially into the marl. Among the three, borings B-1002 and B-1004 penetrated through the marl into the Still Branch and Congaree Formations and boring B-1003 went as deep as 407.8 meters (1,338 ft) into the bedrock. The applicant obtained a total of 58 SPT N-values and corresponding samples, as well as 12 tube samples from the Blue Bluff Marl and the Lower Sand Stratum, and performed P-S velocity logging in the three borings that penetrated the marl. In addition to its ESP investigation, the applicant stated that it considered the soil engineering properties from the previous investigations of Units 1 and 2.

From its review of SSAR Section 2.5.4 and the applicant's response to this and other RAIs, the staff found in the SER with Open Items that the applicant relied more on the previous investigations for the existing Units 1 and 2 than on its ESP field investigations to obtain geotechnical parameters for the ESP site. The staff determined that, while the applicant could use data from the previous investigations as a reference to support the current site characterization, the applicant should not have relied on the previous data to demonstrate the suitability of the ESP site because those data were generated by following different regulatory requirements, regulatory guidelines, and industry standards, and by using different investigation technologies. In addition, soil property variation between the two sites made reliance on the previous data inappropriate. Therefore, the staff concluded that the applicant did not conduct sufficient field and laboratory tests to reliably determine the subsurface soil static and dynamic properties for the soils beneath the Blue Bluff Marl at the ESP site. This was identified in the SER with Open Items as Open Item 2.5-11.

In response to Open Item 2.5-11, the applicant stated that the ESP investigations were intended for limited study of the site conducted in accordance with RS-002, "Processing Applications for

Early Site Permits," and following the example of other recently accepted ESP studies. However, the applicant indicated that additional investigations were ongoing at the site as part of the COL investigation, including 68 power block borings, 42 of which penetrated the Blue Bluff Marl, as well as geophysical and laboratory testing, all of which were included in later revisions of the SSAR. The staff reviewed the guidelines of RS-002, as well as the additional borings and analyses conducted as part of the COL investigation and described in Revision 4 of the SSAR. Based on the inclusion of additional borings, which followed the guidance presented in RG 1.1.32 and RG 1.138, and which penetrated the load-bearing Blue Bluff Marl, the staff concludes that the applicant conducted sufficient field and laboratory tests at the site of VEGP Units 3 and 4 to adequately determine the static and dynamic property values included in Revision 4 of the SSAR. Based on this conclusion, the staff considers Open Item 2.5-11 closed. Furthermore, the closure of Open Item 2.5-11 also resolves the portions of RAI 2.5.4-3 that relate to the properties of subsurface materials at the site of VEGP Units 3 and 4.

In RAI 2.5.4-3, the staff also asked the applicant to explain the low SPT blow count values (as low as 9 bpf) in the Lower Sand Stratum below the Blue Bluff Marl, because low SPT blow count values often indicate the presence of soft soil layers. For comparison, the average blow count for the same layer is about 60 bpf. The applicant explained that this low SPT N-value (9 bpf) in the Lower Sand Stratum could be due to the existence of disturbed materials at the bottom of the drill hole because other geophysical measurements at the same depth showed no physical or strength abnormalities. After reviewing the applicant's response, the staff agreed that the disturbed materials at the bottom of the drill hole may have caused this anomalously low SPT value in the Lower Sand Stratum. However, because the Lower Sand Stratum is one of the load-bearing layers and the applicant was also committed to performing more borings during the COL stage, the staff considered that obtaining additional data on the Blue Bluff Marl and Lower Sand Stratum during the COL stage to confirm the absence of soft materials in these load-bearing layers would be acceptable. Accordingly, in the SER with Open Items, the staff identified this as COL Action Item 2.5-1.

However, in the revised SSAR, the applicant incorporated significant information obtained during the COL site investigations. The applicant included the results of additional subsurface borings, test pits, and SPTs. The staff reviewed this information and determines that none of the additional data provided as part of the applicant's COL investigation results suggests the presence of a soft material within the load-bearing layers at the VEGP Units 3 and 4 site. The inclusion of this information in the revised SSAR addresses the needs of COL Action Item 2.5-1. Therefore, the staff concludes that COL Action Item 2.5-1 is no longer necessary.

The staff considered the existence of the very low SPT N-values measured from the ESP field tests, and in RAI 2.5.4-3(c), asked the applicant to explain whether there were any indications of soft zones in the Upper Sand Stratum, such as those encountered at the SRS. In its response to RAI 2.5.4.-3(c), the applicant stated that it encountered "soft zones" with SPT N–values of 5 bpf in the Upper Sands at ESP boreholes B-1001, B-1004, B-1005, and B-1006. The applicant also stated that if these kinds of soil are saturated with water they would liquefy during certain seismic events, which may result in surface settlement of several inches. The applicant then referred to its RAI 2.5.4-2(a) response, which provided further details about the extent of the soil replacement in the power block area that would occur during the COL stage.

After reviewing the applicant's response to RAI 2.5.4-3, the staff concluded in the SER with Open Items that, because the extent of the excavation and backfill will be limited in both the vertical and horizontal directions at the ESP site, it was not clear from the response that the purpose of the placement of backfill material is to eliminate the existence of such soft zones

located outside the foundation area. Although these soft zones are outside of the immediate foundation area, these soft zones can still have potential adverse impacts on the foundation and the structures of the nuclear power plant. In its response, the applicant committed to take six more deep borings (250 ft to 400 ft deep) during the COL subsurface investigation. Although this information was not necessary at the ESP stage to determine whether 10 CFR Part 100 is satisfied, the issue of confirming the locations of the soft zones and evaluating the potential impact of the soft zones on the foundation and structures was identified as COL Action Item 2.5-2 in the SER with Open Items.

However, in the revised SSAR, the applicant included the additional boring logs and data obtained as part of its COL site investigations, which the staff reviewed. The summary of this additional information can be found in Section 2.5.4.2.2 and 2.5.4.2.3, where the applicant stated that an additional 174 borings were completed as part of the COL investigations. The applicant used these additional borings to confirm the locations of soft zones within the Upper Sand Stratum at the Unit 3 site, to evaluate the potential impact these zones would have on the stability of the plant foundations and safety-related structures, and to verify the ESP characterization of the Upper and Lower Sands, as well as to further validate the ESP characterization of the Blue Bluff Marl. Using this information, the applicant confirmed that the Upper Sand Stratum is too variable and potentially unstable a stratum and further supported the applicant's decision to completely remove the material. Since this information further confirmed the locations of soft zones within the site area, and addressed the minimum number of borings as requested by COL Action Item 2.5-2, the staff finds that COL Action Item 2.5-2 is no longer necessary.

Effective Angle of Internal Friction

In RAI 2.5.4-9, the staff asked the applicant to clarify how the effective angle of internal friction was determined for the soils underlying the ESP site. The applicant responded that it estimated the effective angle of internal friction of 34E using an empirical correlation associated with SPT N-values (Bowles 1982). From its review of the applicant's response, the staff considered that the internal friction angle calculated based on SPT N-values varies significantly, depending on the correlations used. For example, for N-values between 10 and 40, the corresponding soil internal friction angle values vary from 30E to 36E (Peck 1974) or from 35E to 40E (Bowles 1982). More importantly, the N-values measured for the ESP site are all below 20 (from 3 to 19), according to SER Table 2.5.4-3. Therefore, the use of a friction angle of 34E based on an N-value of 25 for the Upper Sand Stratum appeared to be inappropriate. In the SER with Open Items, the staff concluded that the applicant did not provide reliable effective angles of internal friction for the subsurface soils because it did not have sufficient SPT N-values from the ESP investigation to support its calculation. The internal friction angle for the subsurface soils is one of the input parameters in calculating bearing capacity and settlement, as well as liquefaction potential. Therefore, in the SER with Open Items, the issue regarding the effective angles of internal friction for the subsurface soils was designated as Open Item 2.5-14.

The applicant responded to Open Item 2.5-14 by stating that the effective angle of internal friction of the subsurface soils was estimated based on empirical correlations associated with SPT N-values. Furthermore, the applicant summarized the measured SPT N-values, noting that a large number of values were recorded in the Upper Sand Stratum, which would be removed during construction. Some N-values measured below the Upper Sand did not achieve a full 12 inches of penetration, which the applicant attributed to either the high relative density of the material encountered or the intact nature of the in-situ material. The applicant updated the

SSAR to incorporate the additional COL investigation data, such as N-values and shear strength testing, which was used to verify the effective angle of internal friction.

The staff reviewed the response to Open Item 2.5-14, focusing its review on the additional data provided in the revised SSAR. In the revised SSAR, the applicant provided the effective angle of internal friction for both the Upper and Lower Sand Strata (34 and 41 degrees, respectively). The applicant used an empirical correlation associated with the average SPT N-values (Bowles 1982) from the ESP investigation, based on N60 equals 25 bpf, which the staff agrees is an acceptable method by which to determine the effective angle of internal friction. Based on the inclusion of the effective angles of internal friction in the revised SSAR, which were determined using an acceptable method of correlation to the empirical averages of Bowles, the staff considers Open Item 2.5-14 closed. The closure of Open Item 2.5.4-14 also resolves RAI 2.5.4-9.

High Strain Elastic Modulus

In RAI 2.5.4-11, the staff asked the applicant to explain: (1) why it used the Davie and Lewis' (1988) relationship to estimate the high strain elastic modulus (E) for the Upper and Lower Sand Strata underlying the ESP site; (2) what the consensus is about using the Davie and Lewis relationship between SPT and E; and (3) the extent of the application of the Davie and Lewis relationship. In response to RAI 2.5.4-11, the applicant stated that Bechtel used the Davie and Lewis relationship extensively to estimate settlement when compared to observed settlements for a wide range of foundation sizes on granular materials from clean sands to silty sands to gravels, such as the medium-dense, silty sand of the Upper Sand Stratum and the very dense silty sand of the Lower Sand Stratum. Therefore, the applicant believed that the Davie and Lewis relationship is applicable to the Lower Sands. In addition, the applicant found that the Davie and Lewis relationship provided an E value that was closer to the median value of five different relationships for both sand strata than were the four other E and N (the SPT N-value) relationships detailed in SER Table 2.5.4-5, which is taken from the applicant's response to RAI 2.5.4-11. The applicant also implied that Davie and Lewis' relationship provided reasonable predictions of settlement when compared to measured settlements, and with a reasonable consensus.

Table 2.5.4-5 - Summary of Calculation of Elastic Modulus E

Reference	Relationship	E, ksf	
		N = 25 bpf	N = 62 bpf
Bowles (1987)	E = 10 (N + 15) ksf	400	770
D'Appolonia et al. (1970)	E = 432 + 21.2N ksf	962	1,746
Parry (1971)	E = 100N ksf	2,500	6,200
Schmertman (1970) and Schmertman et al. (1978)	E = 30N to 50N ksf	750 to 1,250	1,860 to 3,100
Yoshida and Yoshinaka (1972)	E = 42N ksf	1,050	2,604
Median		**1,006**	**2,232**
Davie and Lewis (1988)	E = 36N ksf	900	2,232
Note: The references shown above are cited in Davie and Lewis (1988) and are listed at the end of the response to this RAI.			

Based on its review of the applicant's response to RAI 2.5.4-11, the staff concurs with the applicant's conclusion about the applicability of the Davie and Lewis' relationship in estimating elastic modulus. However, the applicant needed to use appropriate SPT N-values to obtain a

reasonable E value. Since the N-values obtained from the ESP investigation and the design undrained shear strength values determined by the applicant for the ESP soils are not reliable for very limited data, the staff determined in the SER with Open Items that the applicant did not have sufficient site-specific data to justify the determination of the design parameter E for the Upper and Lower Sand Strata. Therefore, in the SER with Open Items, the issue of using appropriate SPT N-values to determine a reasonable elastic modulus value for the Upper and Lower Sand Strata was designated as Open Item 2.5-16.

In response to Open Item 2.5-16, the applicant referenced the guidance of RS-002 regarding the determination of the engineering properties of the soil and rock strata underlying the site. The applicant stated that the elastic modulus was derived from representative data collected during the ESP site investigation and the measured SPT N-values from the Lower Sand Stratum. Finally, the applicant conducted additional SPTs and provided the data in the revised SSAR.

The staff focused its review of the response to Open Item 2.5-16 on the additional information provided by the applicant in both the response and the revised SSAR, and on the guidance of RS-002. The applicant provided the derived elastic modulus for each of the subsurface strata at the VEGP Units 3 and 4 site (SSAR Table 2.5.4-1). Based on the inclusion in the revised SSAR of additional SPTs, which indicated the hard to very hard and the dense to very dense natures of the Blue Bluff Marl bearing stratum and the Lower Sand Stratum, respectively, from which the elastic modulus was derived, the staff concludes that the applicant has provided sufficient information to close Open Item 2.5-16. The closure of Open Item 2.5-16 also resolves RAI 2.5.4-11.

Determination of Unit Weight Values

In RAI 2.5.4-12, the staff asked the applicant to explain how unit weight values were determined for different soils and why there was a discrepancy between the average values given in the SSAR text and those listed in SSAR Table 2.5.4-1. The applicant explained in its response that the unit weight values were determined based on the laboratory test during the ESP subsurface investigation. However, the applicant used the average values of unit weight based on VEGP Unit 1 and 2 laboratory test results because there were more test data available, despite results that differed from those obtained from ESP tests. The staff considered that the unit weight values for underlying soils are very basic soil property parameters used in many calculations/analyses. However, the applicant did not have sufficient data to calculate the unit weight values for the ESP subsurface soils and instead used the values from previous investigations. In the SER with Open Items, the staff concluded that it was not acceptable for the applicant to use these previously determined engineering parameters in this manner. Accordingly, this issue was designated as Open Item 2.5-17 in the SER with Open Items.

In response to Open Item 2.5-17, the applicant provided the tabulated unit weight for 15 samples from the Blue Bluff Marl and 3 samples from the Lower Sand Stratum. The number of measurements was limited to be consistent with the scope of the ESP site investigation program as designed by the applicant. Additional unit weight measurements were included by the applicant in the revised SSAR and are provided in Table 2.5.4-1 of this SER.

In its review of the response to Open Item 2.5-17, the staff focused on the additional unit weight measurements provided in the revised SSAR Table 2.5.4-1. The staff also considered the description of these additional unit weight measurements and concludes that a sufficient number of samples was measured and that the value ranges of the samples tested are

consistent for the sand, silt, and clay materials that were tested. Therefore, the staff concludes that the information provided by the applicant in the revised SSAR with respect to the unit weight measurements for the Blue Bluff Marl and Upper and Lower Sand Strata at the site is acceptable and follows the guidelines presented in RG 1.138. Accordingly, the staff considers Open Item 2.5-17 closed. This closure also resolves RAI 2.5.4-12.

Chemical Tests

The staff noted that, in SSAR Section 2.5.4.2.5.3, the applicant stated that chemical tests were not included in the ESP laboratory testing program. The applicant also stated in the SSAR that chemical tests would be required for the backfill materials placed in proximity of planned concrete foundations and buried metal piping, and the applicant committed to conduct these chemical tests in the COL investigation phase. Accordingly, the need to provide chemical test results on the backfill was identified as COL Action Item 2.5-3 in the SER with Open Items. However, in a later revision to the SSAR, the applicant included additional information on the excavation and backfill plans for the site of VEGP Units 3 and 4, including the chemical tests performed on the backfill materials, the results of which are included in SSAR Appendix 2.5E. These plans and tests were evaluated by the staff as part of the information provided in support of the LWA request. Because the application now contains this information in the SSAR, the staff concludes that COL Action Item 2.5-3 is no longer necessary.

Blue Bluff Marl Design Shear Strength

In RAI 2.5.4-7, the staff asked the applicant to explain why the undrained shear strength values (7.2 kPa (150 psf) to 205.9 kPa (4,300 psf)) from the UU tests performed on the Blue Bluff Marl samples were significantly lower than the SSAR specified design value, 478.9 kPa (10,000 psf), and to explain why these values differed substantially from the values (12.0 kPa (250 psf) to 23,946.4 kPa (500,000 psf)) obtained from previous investigations conducted for Units 1 and 2. The staff also asked the applicant to justify the use of a 478.9 kPa (10,000 psf) design value based on the SPT N-values measured during the ESP investigations.

In response to RAI 2.5.4-7, the applicant stated that the laboratory measurements of undrained shear strength for the Blue Bluff Marl (Lisbon Formation) yielded low values because the tests were performed using one confining pressure corresponding to the overburden pressure. The applicant also listed some qualitative factors to explain why these laboratory values were low. These factors included (1) being unable to push the CPTs below the Barnwell Group and into the Lisbon Formation (Blue Bluff Marl), (2) Shelby tubes being unable to penetrate into the Lisbon Formation without being damaged, which indicated that the soils were very hard, and (3) possible disturbance of samples obtained by pitcher barrel due to sampling, storage, and transportation processes. For these reasons, the applicant adopted an undrained shear strength design value for the Blue Bluff Marl from the FSAR for VEGP Units 1 and 2. The applicant further provided empirical correlations between the PI value, SPT N-value, shear wave velocity, and the undrained shear strength to justify the use of the SSAR design value of 478.9 kPa (10,000 psf).

From its review of the applicant's response to RAI 2.5.4-7, the staff found in the SER with Open Items that the qualitative and quantitative information provided by the applicant did not justify the use of the SSAR design strength value of 478.9 kPa (10,000 psf) for the Blue Bluff Marl, based on the following five considerations:

1. The design strength value obtained from the previous investigation for Units 1 and 2 was generated using different regulatory requirements, different industry standards, and different testing technologies. The applicant can use the data or engineering values from the previous investigation as a reference to support the current decision, but may not use the data as a direct input to calculate engineering parameters or previous engineering values directly for the ESP site.

2. As for the qualitative reasoning presented by the applicant, being unable to push the CPT and Shelby tubes through the Blue Bluff Marl does not justify the applicant's use of a design strength value much higher than the values obtained from the testing. According to Appendix 2.5 A to the SSAR, because soil samples collected from the Blue Bluff Marl contain gravels, it is possible that the CPT and Shelby tubes engaged gravels causing it to be difficult for them to push through the soil. Therefore, this factor does not support the adoption of a specific value of 478.93 kPa (10,000 psf) as the design shear strength for the Blue Bluff Marl.

3. If, as the applicant implied, the samples used in the ESP tests were disturbed because of the sampling, storage, and transportation processes, then there would be no reliable ESP laboratory test results to support the determination of the design value for the ESP site.

4. The applicant did not justify the applicability of the empirical correlations used in its response, such as the correlations between the undrained shear strength and PI, N-value, or shear wave velocity. Specifically, Mayne (2006) developed the correlation between shear wave velocity and shear strength from one group of clays, and the applicant used this correlation in its response to RAI 2.5.4-7, but this correlation may not be applicable to the Blue Bluff Marl at the ESP site. Furthermore, Mayne recently recommended another correlation developed by Laval University Group (2007) based on data from three groups of clays. This correlation resulted in a lower shear strength value than the one originally developed by Mayne (2006).

5. Even if an empirical correlation is applicable, the applicant did not use appropriate input parameters. Instead, the applicant used inappropriate input parameters, based on very limited data, and values that vary significantly. For example, the design PI value of 25 is an average value based on 18 data points ranging from 5 to 58, with 3 points above 50. The applicant obtained the N-value 80 from a total of 58 samples; among the samples there were only 23 actual measured N-values, ranging from 27 to 81. The applicant extrapolated the N-values linearly for 35 measurements in which the sampler did not penetrate 12 inches, and most of those data ended up having the cutoff value of 100. As mentioned previously, most of the 35 SPT measurements did not penetrate 12 inches because the samplers were in contact with gravels. Therefore, the average N-value does not meaningfully represent the general soil properties due to the lack of actual measurement and possible gravel engagement during the SPT tests.

Based on the above considerations, the staff concluded in the SER with Open Items that the applicant did not provide sufficient data to reliably derive the undrained shear strength value for the Blue Bluff Marl for the design. Accordingly, this was identified as Open Item 2.5-12 in the SER with Open Items.

In response to Open Item 2.5-12, the applicant stated that SPTs and split-spoon sampling were conducted in almost all the ESP borings in accordance with ASTM D 1586 to provide a measure of the relative density for cohesionless soils and consistency for cohesive soils. The applicant

also described the split-spoon sampling process, in which the sampler is driven into massive in-situ materials, converting the material to coarse-grained soils through the crushing process. The applicant indicated that it is this crushing process that was responsible for the high recorded N-value of the materials sampled. Although the applicant followed the guidance of Appendix X2 of ASTM D 2488 to identify the materials sampled during the ESP investigations, it acknowledged that this method has led to some confusion regarding the presence of gravel-sized particles taken from the borings. The applicant clarified this confusion by stating that gravel-sized particles were the result of the crushing process, and were not reflective of actual gravel encountered in the subsurface. The applicant also described ongoing laboratory tests (grain size distribution, Atterberg Limits, and carbonate content) that confirmed the visual reclassifications of the samples. Finally, the applicant revised the SSAR to include additional field and laboratory test results, which were used to verify the undrained shear strength of the Blue Bluff Marl.

The staff reviewed the information provided in response to Open Item 2.5-12. In particular, the staff focused on the applicant's classification of the crushed material from the split-spoon sampler in accordance with Appendix X2 of ASTM D 2488. The staff evaluated the applicant's explanation that no gravel was encountered in the subsurface, but that gravel-sized particles were produced from the crushing of more massive materials, such as micritic limestone or fossiliferous shale beds, which would explain the isolated occurrence of shell fragments in the subsurface investigations. The staff considers this a more likely explanation for the occurrence of "gravel-sized" particles resulting from sampling of the Blue Bluff Marl as this can happen when attempting to sample very hard material. And although this sampling method can produce "gravel-sized" particles, these "fragments" are not actual gravel and should not have been identified as such by the applicant. The applicant acknowledged this error and, in subsequent review of the sample material, was able to correctly identify the materials as resulting from the crushing of very hard massive materials. The staff also considered the additional field and laboratory tests included by the applicant in the revised SSAR as summarized in this SER. Based on the application of the appropriate ASTM guidance for reclassification of the gravel-sized particles encountered at the site, and the additional field and laboratory test results provided in the revised SSAR, in particular the Atterberg Limits and carbonate content tests indicating the presence of limestones and fossiliferous shales, the staff considers Open Item 2.5-12 closed. The closure of Open Item 2.5-12 also resolves the remaining issue from RAI 2.5.4-7.

In RAI 2.5.4-8, the staff asked the applicant for the following:

1. a description of the previous laboratory testing methods and results which indicate that the Blue Bluff Marl is highly preconsolidated,

2. justification for the assumption of an undrained shear strength of 766.3 kPa (16,000 psf) while the undrained unconsolidated test results yielded values from 7.2 to 205.9 kPa (150 to 4,300 psf).

3. justification for the conclusion that "the pre-consolidation pressure of the Blue Bluff Marl was estimated to be 3,831.4 kPa (80,000 psf)," and

4. justification for the conclusion that "settlements due to loadings from new structures would be small due to this pre-consolidation pressure" for the Blue Bluff Marl.

In its response to RAI 2.5.4-8, the applicant provided the following information:

1. The original data and interpretation were based on laboratory tests performed for VEGP Units 1 and 2, which included 191 one-point UU triaxial tests and 38 consolidation tests. The applicant used vertical pressures that reached 3,065 kPa (64000 psf) to perform consolidation tests for all 38 samples. Most of the test results (void ratio versus vertical effective stress curves) showed very flat curves, which indicated that the preconsolidation pressure had not been achieved.

2. The undrained shear strength of 766 kPa (16,000 psf) was an average value based on VEGP Unit 1 and 2 test data calculated from 185 one-point UU triaxial tests that disclosed undrained shear strength values of less than 2,394.6 kPa (50,000 psf).

3. The applicant used the Skempton (1957) method to estimate the preconsolidation pressure of the Blue Bluff Marl by relating the preconsolidation pressure to the PI value and the undrained shear strength. The applicant concluded that the Lisbon Formation was highly overconsolidated because the calculations showed that the overconsolidation ratios (OCRs) were in the range of 3.6 to 5, and most of the consolidation test results on 38 samples from the Lisbon Formation, reported in Bechtel (1974b), showed very flat curves, which indicated that the preconsolidation pressure exceeded 3,065 kPa (64,000 psf).

4. The applicant also concluded that the settlement due to loadings from new structures would be small based on observation of VEGP Units 1 and 2 and that the settlements would take place during the construction phase.

Based on its review of the applicant's response to RAI 2.5.4-8, the staff found in the SER with Open Items that it was inappropriate to use the average undrained shear strength value for VEGP Units 1 and 2 as an input value to calculate preconsolidation pressure and OCRs for the Blue Bluff Marl at the ESP site because the previous value was obtained based on different regulatory requirements, regulatory guidelines, industry standards, and testing technologies. In addition, the spatial variation of the soil properties also made reliance on the VEGP Units 1 and 2 values inappropriate. Moreover, the previous shear strength value differs significantly from the one obtained during the ESP testing. Therefore, the applicant did not have sufficient sampling and testing results to reliably derive the input undrained shear strength used in calculating the preconsolidation pressure and OCRs of the Blue Bluff Marl. Accordingly, this was designated as Open Item 2.5-13 in the SER with Open Items.

In response to Open Item 2.5-13, the applicant stated that the ESP site investigation was limited in scope due to the depth of knowledge available based on VEGP Units 1 and 2. The applicant also noted that although the ESP borings disclosed field measurement data consistent with the previous investigations, there was some confusion regarding the material descriptions as was discussed in response to Open Item 2.5-12. The applicant clarified this issue in its revision to the SSAR, which also included calculations of preconsolidation pressure and overconsolidation ratios for the Blue Bluff Marl using additional test data from the ESP investigation.

The staff focused its review of Open Item 2.5-13 on the additional information provided in the revised SSAR related to preconsolidation pressure and the OCRs for the load-bearing Blue Bluff Marl. The staff also considered the closure of Open Item 2.5-12 as referenced in the applicant's response to Open Item 2.5-13. Based on the applicant's revisions to the SSAR to include preconsolidation pressure of 3,831 kPa (80,000 psf) and an OCR of 8 for the Blue Bluff Marl based on additional site investigations that indicated that settlements due to loadings from new

structures would be small due to the high preconsolidation pressure, the staff concludes that the applicant has sufficiently addressed the calculations identified in Open Item 2.5-13 and therefore the staff considers Open Item 2.5-13 closed. Furthermore, the closure of Open Item 2.5-13 resolves RAI 2.5.4-8.

In RAI 2.5.4-10, the staff asked the applicant to provide the relative density of the Blue Bluff Marl. The applicant stated in its response that the design value of the undrained shear strength for the soil was 478.9 kPa (10,000 psf) and its preconsolidation pressure could be as high as 3,831 kPa (80,000 psf); therefore, the applicant concluded that the Blue Bluff Marl was highly overconsolidated and behaved as hard clay or soft rock material, not as a granular material. The applicant further stated that relative density does not apply to the Blue Bluff Marl. From its review of the applicant's response, the staff concluded in the SER with Open Items that test data for the Blue Bluff Marl were very limited. As described in the SSAR, the limited laboratory test data showed that the percent fines content ranged from 24 to 77 percent, the moisture content ranged from 14 to 67 percent, and the PI ranged from non-plastic to 58 percent. Each of the above-mentioned parameters does not exclude the possibility of the marl being liquefied. In addition, the undrained unconsolidated tests yielded undrained shear strength values from 7.2 to 205.9 kPa (150 to 4,300 psf), which significantly differ from the design shear strength value of 478.9 kPa (10,000 psf), as indicated in the discussion of RAI 2.5.4-7. Therefore, the applicant's response did not support the conclusion that the Blue Bluff Marl would behave as a hard clay or soft rock material because the applicant did not use the ESP soil engineering values to calculate relative density for the Blue Bluff Marl. Accordingly, the need to demonstrate that the Blue Bluff Marl would behave as a hard clay or soft rock material, and thus not need to be addressed using relative density, was designated as Open Item 2.5-15 in the SER with Open Items.

The applicant's response to Open Item 2.5-15 referenced the response to Open Item 2.5-12 and the confusion in subsurface material description. The applicant also stated that while it is technically correct to identify some Blue Bluff Marl samples as sands and gravels, this description does not accurately indicate the in-situ structure of the marl. The applicant conducted laboratory testing to evaluate the carbonate content of the marl materials previously identified as sands and gravels, which the applicant concluded were indicative of a soft rock or hard clay material with lesser amounts of coarse sand and no determinable gravel present. The applicant further stated that the material that was previously identified as gravel was reclassified as limestone fragments. Again, the applicant included the results of additional data and site investigations in the revised SSAR.

The staff considered both the applicant's response to Open Item 2.5-15 as well as the closure of Open Item 2.5-12, which was referenced therein. Since additional laboratory data and site investigations were provided in the revised SSAR that clarified the composition of the Blue Bluff Marl, and the staff concluded in Open Item 2.5-12 that there was no determinable gravel in the subsurface material, the staff concludes that the applicant has provided a sufficient explanation, including supporting data and analyses, to prove that the marl will behave as a hard clay or soft rock material at the ESP site. Based on the resolution of Open Item 2.5-12 and the additional information regarding to composition of the Blue Bluff Marl in the revised SSAR, the staff considers Open Item 2.5-15 closed. Furthermore, with the closure of Open Item 2.5-15, the staff also considers RAI 2.5.4-10 resolved.

Following the submittal of the revised SSAR and the LWA request, the staff issued further requests for additional information to address the supplemental information. These supplemental RAIs are evaluated throughout the following sections and are identified with an "S."

The staff's evaluation of the information provided in support of the LWA request is as follows:

Field Investigations

Similar to its request in RAI 2.5.4-1, in RAI 2.5.4-1S the staff asked the applicant to 1) clarify how it had arrived at the number of ESP soil borings as 174 and to provide a detailed accounting of these additional borings, and 2) identify how many of the penetrations would be unusable for the site-specific analyses because they were taken through the Upper Sand Stratum material that would be excavated and replaced. In response to RAI 2.5.4-1S, the applicant provided a table that broke the number of borings down by series number, subject (i.e., location within the site or specific structure), and the exact number of borings at the subject location. The table indicates that the applicant completed 40 borings in the Unit 3 power block and cooling tower area, and 37 in the Unit 4 power block and cooling tower area. The remaining 97 borings were distributed across the rest of the site of VEGP Units 3 and 4. With this information, the staff was able to account for the number of total borings and their locations within the site, and the staff accordingly considers Item 1 of RAI 2.5.4-1S resolved. Also in this response, the applicant stated that 70 soil borings were located in the immediate vicinity of the combined power block footprint with exploration depths varying from 6.5 to 128 m (21.5 to 420 ft). The applicant further explained that with the exception of two offset borings, each of these borings was drilled through Upper Sand Stratum and advanced into the Blue Bluff Marl. The applicant further stated that 42 of these 70 borings penetrated the Blue Bluff Marl and advanced into the Lower Sand Stratum. With this information, the staff considers Item 2 of RAI 2.5.4-1S resolved because, as the applicant advanced 68 of the 70 borings through the Upper Sand Stratum and into the underlying layers, almost every boring produced usable site-specific data. However, the applicant's response that only 42 borings penetrated the Blue Bluff Marl led the staff to request additional information identified as RAI 2.5.4-20S.

In RAI 2.5.4-20S, the staff asked the applicant to provide additional information to demonstrate that the 42 borings that penetrated the Blue Bluff Marl were sufficient to satisfy the site foundation criteria contained in Regulatory Guides 1.132 and 1.138, including the boring depth acceptance criteria. The staff also asked for clarification of the statement made in response to RAI 2.5.4-2S that only six of 70 borings penetrated the Lower Sand Stratum.

The applicant responded that, in keeping with RG 1.132, the borings were located beneath and adjacent to structures to provide the maximum aerial coverage, which resulted in a boring at the center of the safety-related structures and uniformly spaced inside and relatively close to the perimeter of the other power block structures. In the response to RAI 2.5.4-20S, the applicant provided a Table 1, Summary of COL Power Block Borings, which summarized the number of borings for each structure in each unit. The guidance in RG 1.132 for the density of site borings is one boring per 929 square meters (10,000 square feet): however, the applicant determined the density of its borings to be one boring per 501 square m (5,400 square ft). Regarding the boring depth acceptance criteria in RG 1.132, Appendix D of the RG states that "dmax, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loading is less than 10 [percent] of the effective in-situ overburden stress." The applicant noted that the foundation that will have the largest dmax is the nuclear island base mat. Based on the AP1000 DCD Revision 15 design bearing pressure under the base mat

of 411 kPa (8.6 ksf), the applicant determined that the nuclear island base mat dmax is on the order of 82 m (270 ft). The applicant noted that three borings were drilled at each unit to a depth of at least 76 m (250 ft), and one boring at Unit 3 was drilled to a depth of 128 m (420 ft) while the deepest boring at Unit 4 was to a depth of 122 m (400 ft). As for other power block structures, the applicant noted that the other structures located in the power block were founded nominally at the surface, and that the exploration depth of the borings for these structures was generally 45.7 m (150 ft).

After considering the clarifications and additional information presented by the applicant concerning the RG 1.132 guidelines for boring spacing, depth, and density, the staff has determined that the applicant's response is sufficient to address the location of borings beneath and adjacent to structures to provide the maximum aerial coverage, the density of required borings, and the minimum depth requirements for boreholes because 1) the applicant exceeded the RG 1.132 guidance for density of site borings, 2) the applicant advanced a boring within each nuclear island power block to a depth well in excess of the RG 1.132 guidance for dmax, and 3) the applicant met the intent of the RG 1.132 guidance for spacing by locating a boring at the center of the safety-related structures and by uniformly spacing other borings around the inside and relatively close to the perimeter of the other powerblock structures.

With respect to the guidelines of RG 1.138, the applicant explained that specific guidance about the number of tests that should be performed was not provided in RG 1.138. In response to the RAI, the applicant provided the staff with a table that summarized the COL power block borings for each structure in each unit. Regarding the applicant's response concerning the laboratory testing guidelines in RG 1.138, the staff agrees with the applicant's statement that the RG does not provide specific guidance about the numbers of laboratory tests that should be performed and that this is most likely because the numbers and types of tests depend on various site-specific factors such as the location of borings with respect to significant structures, the depth of sampling (e.g., it may be within a zone of excavation), the type of sample materials (cohesive, cohesionless, soil or rock), and the sample type (disturbed or undisturbed). The RG states that the focus of laboratory investigations should depend on the design requirements and nature of problems encountered or suspected at the site (i.e., some level of determination about the types and quantities of testing needs to be left to professional judgment by the onsite personnel). The staff determined by its review of the applicant's referenced tables, in particular SSAR Tables 2.5.4-3, 2.5.4-3a, and 2.5.4-4, "Types and Numbers of Laboratory Tests for the ESP and COL Investigations and Summary of Laboratory Tests Performed on Selected Soils Samples", that summarize the laboratory test results performed on ESP boring samples, that the applicant has conducted a laboratory testing program sufficient to adequately characterize the engineering properties of the subsurface materials. The staff reached this determination because the laboratory testing program conducted by the applicant included a variety of conventional index (tests that determine the properties of soils that indicate the type and condition of soils and provide a relationship to structural properties such as strength, compressibility, permeability, swelling potential, e.g., particle size distribution and consistency limits) and geotechnical engineering tests as well as dynamic soil test (RCTS) such that the applicant was able to sufficiently characterize the properties of the site soils for the purpose of evaluating the stability of the site for the applicant's planned construction. Finally, the applicant stated that the listed number of borings penetrating the Lower Sand Stratum was a typographical error. Therefore, based on the applicant's responses to RAIs 2.5.4-1S and 2.5.4-20S, the staff concludes that these RAIs were adequately addressed by the applicant and considers them resolved.

Shear Wave Velocity Profiles

In RAI 2.5.4-4S, the staff requested that the applicant provide an assessment of the in-situ velocity profile through the Upper Sand Stratum. The applicant described the additional laboratory strength testing and shear wave velocity measurements performed in the Upper Sand Stratum in the power block and surrounding areas as part of its COL investigations. Figure 2.5.4-3 of this SER shows the in-situ shear wave velocity profile through the Upper Sand Stratum to the Dunbarton Triassic Basin rock. The applicant provided the test results of the laboratory strength testing, which included 10 consolidated undrained triaxial shear tests from relatively undisturbed Upper Sand Stratum samples, Atterberg Limits and chemical tests, and a plot of shear wave velocity measurements in the stratum. In follow-up RAI 2.5.4-23S, the staff asked the applicant to provide justification as to why the two-dimensional (2D) wave velocity consideration was not considered in the SSI analysis.

The staff reviewed the response to RAI 2.5.4-4S as it related to geotechnical engineering, especially the additional strength and shear wave velocity measurements included in the revised SSAR, and concludes that the applicant provided sufficient information to close the geotechnical engineering aspects of RAI 2.5.4-4S because the additional laboratory test results, particularly the Atterberg Limits, confirmed the variable nature of the Upper Sand Stratum and its corresponding low shear strength. Furthermore, the applicant collected additional shear wave velocity data in the Upper Sand Stratum that displayed values over a large range but generally below the required minimum of 304.8 m/s (1,000 fps), which also confirmed the variable nature of the Upper Sand Stratum materials and further validated the applicant's decision to completely remove this stratum. Since the response to RAI 2.5.4-23S specifically addresses structural engineering aspects at the VEGP Units 3 and 4 site, the staff evaluates the response in Section 3.8 of this SER.

The site characteristic values of shear wave velocities were specified for depth intervals and are given in Appendix A to this SER and SER Tables 2.5.4-6 and 2.5.4-7. The applicant determined these characteristic values from the geophysical surveys completed at the VEGP site. Because the values were determined from the results of the applicant's geophysical surveys, which the staff reviewed and found to be acceptable in Section 2.5.4.3.4 of this SER, the staff concludes that these values are acceptable for use as the site characteristics.

Table 2.5.4-6 Shear Wave Velocity for ESP Site Amplification Analysis

Geologic Formation	Depth (feet)	V_s (fps)
Compacted Backfill	0 to 6	573
	6 to 10	732
	10 to 14	811
	14 to 18	871
	18 to 23	927
	23 to 29	983
	29 to 36	1,040
	36 to 43	1,092
	43 to 50	1,137
	50 to 56	1,175
	56 to 63	1,209
	63 to 71	1,232
	71 to 79	1,253
	79 to 86	1,273
Blue Bluff Marl (Lisbon Formation)	86 to 92	1,400
	92 to 97	1,700
	97 to 102	2,100
	102 to 105	1,700
	105 to 111	2,200
	111 to 123	2,350
	123 to 149	2,650
Lower Sand Stratum (Still Branch)	149 to 156	2,000
	156 to 216	1,650
(Congaree)	216 to 331	1,950
(Snapp)	331 to 438	2,050
(Black Mingo)	438 to 477	2,350
(Steel Creek)	477 to 587	2,650
(Gaillard/Black Creek)	587 to 798	2,850
(Pio Nono)	798 to 858	2,870
(Cape Fear)	858 to 1,049	2,710
Dunbarton Triassic Basin & Paleozoic Crystalline Rock	> 1,049	see Table 2.5.4-11, Part B

Table 2.5.4-6 Continued, Six Alternate Profiles

Part B: Rock Shear-Wave Velocities - Six Alternate Profiles

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,402.5	8,000	8,700
1,402.5 to 1,405	8,005	8,703
1,405 to 1,525	8,059	8,739
> 1,525	9,200	9,200

Rock Vs profile corresponding to the location midway between B-1002 and B-1003.

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,450	8,000	8,700
1,450 to 1,550	8,090	8,760
1,550 to 1,650	8,180	8,820
1,650 to 1,750	8,270	8,880
1,750 to 1,830	8,360	8,940
1,830 1,900	8,414	8,976
> 1,900	9,200	9,200

Rock Vs profile corresponding to the location of B-1003.

	Vs (ft/s)	
Depth (ft)	Gradient #1	Gradient #2
1,049 to 1,100	4,400	4,400
1,100 to 1,150	5,650	5,650
1,150 to 1,225	6,650	6,650
1,225 to 1,337.5	7,600	7,600
1,337.5 to 1,450	8,000	8,700
1,450 to 1,550	8,090	8,760
1,550 to 1,650	8,180	8,820
1,650 to 1,750	8,270	8,880
1,750 to 1,850	8,360	8,940
1,850 to 1,950	8,450	9,000
1,950 to 2,050	8,540	9,060
2,050 to 2,127.5	8,630	9,120
2,127.5 to 2,155	8,679.5	9,153
2,155 to 2,275	8,733.5	9,189
> 2,275	9,200	9,200

Table 2.5.4-7 Shear Wave Velocity for COL Site Amplification Analysis

Geologic Formation	Depth (feet) (ft)	V_s (fps) (fps)
Compacted Backfill	0	550
	5	724
	10	832
	20	975
	30	1064
	40	1130
	50	1183
	60	1228
	70	1267
	80	1302
	85	1318
	86.5	1327
	88	1327
Blue Bluff Marl (Lisbon Formation)	88 to 96	1,341
	96 to 102	1,747
	102 to 110	1,988
	110 to 122	2,300
	122 to 156	2,541
Lower Sand Stratum (Still Branch)	156 to 164	1,820
	164 to 220	1,560
(Congaree)	220 to 236	1,757
	236 to 280	2,000
	280 to 328	1,926
	328 to 340	1,727
(Snapp)	340 to 447	2,050
(Black Mingo)	447 to 486	2,350
(Steel Creek)	486 to 596	2,650
(Gaillard/Black Creek)	596 to 807	2,850
(Pio Nono)	807 to 867	2,870
(Cape Fear)	867 to 1,059	2,710

Geotechnical Properties of the Lower Sand Stratum

In RAI 2.5.4-5S, the staff noted that, during its review of the ESP application, some samples below the Blue Bluff Marl were identified as having extremely low blow counts, which called into question the adequacy of the soil material for settlement and bearing capacity. The staff also noted that, although the applicant indicated through informal discussions that these low blow counts were anomalies, the LWA request did not contain an adequate discussion of this anomalous conclusion. Therefore, the staff requested that the applicant provide the basis for the conclusion that the samples with low blow counts were anomalies.

In response, the applicant stated that 42 borings in the power block area penetrated the Blue Bluff Marl, 611 linear feet of drilling was conducted in the Lower Sand Stratum, and 111 SPT split barrel samples were collected from the Lower Sands. The applicant reported that the average corrected blow count reading in the Lower Sand Stratum was 250 bpm (75 bpf), indicative of a very high relative density. The applicant also stated that, with the exception of one value, all of the N60-values taken in the Lower Sand Stratum were greater than 30 bpf, again indicative of a dense to very dense material, although one N-value from a sample taken in the Still Branch Formation of the Lower Sand Stratum at an elevation of -12.6 to -13.1 m (-41.5 to -43 ft), and from which the split barrel sampler was unable to recover a sample, indicated very loose material. The applicant attempted to take an undisturbed sample (UD-11) from elevation -39.5 to -41.5, but no recovery was obtained in this sample. Since the applicant identified the material above this elevation as light gray sand (SP), the difficulty in sampling this material and the weight of hammer reading was an anomaly in sampling that was attributed to disturbed soil conditions at the bottom of the borehole. The applicant surmised that these conditions were likely the result of a hydrostatic pressure imbalance between the borehole and the in-situ hydrostatic pressure, with the resulting imbalance causing a quick condition to develop in the poorly graded sands at the attempted sampling depth. In such circumstances, the resulting disturbed poorly-graded sand will flow out of the sampler, which makes the material difficult to sample, as the applicant appears to have experienced in its lack of sample recovery at that depth. Overall, the applicant concluded that the SPT N-values behaved as expected by increasing with depth. Based on the applicant's response to RAI 2.5.4-5S and because the applicant encountered no other evidence of soft zones or loose material in the 611 linear feet of drilling conducted in the Lower Sand Stratum, the staff concurs with the applicant's explanation that it likely encountered an anomalous condition during sampling at this depth, as such a condition is not an unusual occurrence when attempting to sample very granular material. Therefore, the staff considers RAI 2.5.4-5S resolved. This explanation also addresses COL Action Item 2.5-2, concerning the location and extent of soft zones, which was resolved earlier in this section of the SER.

Geotechnical Properties of the Blue Bluff Marl

The staff identified multiple RAIs related to the properties of the Blue Bluff Marl (BBM).
In RAI 2.5.4-2S, the staff requested that the applicant provide a description of the borings that penetrated into and through the BBM, of the number and types of samples recovered, as well as of the material underlying the BBM. In response to RAI 2.5.4-2S, the applicant stated that 70 borings were taken in the power block area; 42 of these borings penetrated the BBM, accounting for 863 linear m (2,831 linear ft) of drilling in this stratum. Additionally, seven hundred and forty-two SPT split barrel samples (disturbed samples) were obtained in the BBM, for which the applicant presented figures of the SPT N60 values and shear wave velocity measurements. From these SPT data, the applicant recorded an average N-value of 233 blows

per meter (70 blows per foot) with a median value of 240 bpm (72 bpf); the average N60-value is 96. The applicant stated that nearly all of the SPT N60 values from the BBM were greater than 100 bpm (30 bpf). Additionally, the applicant stated that the number of borings penetrating the underlying Lower Sands (LS) was six of seventy, which accounted for 186 linear meters (611 linear ft) of drilling in this stratum. The number of borings that penetrated the Lower Sands was addressed in follow-up RAI 2.5.4-20S, which was previously discussed earlier in this section of the SER.

In follow-up RAI 2.5.4-21S, the staff requested that the applicant provide clarification of how the formulas provided in the response to RAI 2.5.4-2S were used to obtain corrected SPT blow counts. The applicant responded that the formula included in the response was provided as an explanation of how the measured N-values were interpreted in cases where full penetration of the 0.45 m (18-inch) sampler was not achieved due to the presence of very dense and very hard material, which occurred primarily in the BBM. The applicant clarified its conservative approach, which involved interpreting high measured N-values by recomputing the measured N-values using a simpler more intuitive approach; the applicant performed this recomputing where full penetration of the split barrel sample was not achieved due to very hard or very dense material. The applicant noted that the recomputation would not impact the majority of measured N-values where full penetration of the split barrel sampler was achieved, and where full penetration was not achieved because of the hardness or high relative density of the soil, the majority of computed N-values would be at the capped value of 333 bpm (100 bpf). The staff agrees with the applicant's recomputation of the N-values where the applicant was unable to achieve full penetration due to the very hard nature of the marl stratum, as this only affects a relatively small number of the total values measured, and the capped values are still indicative of a very dense or hard material that is the marl stratum. The recomputed and replotted data was included in the ESP Revision 4 for staff's review. Based on the review of the data presented in response to RAIs 2.5.4-2S and 2.5.4-21S, the staff found that the applicant provided sufficient data to enable the staff to determine that the applicant adequately sampled and tested the BBM Stratum and clarified the method used to correct SPT blow counts. Accordingly, the staff considers RAIs 2.5.4-2S and 2.5.4-21S resolved.

In RAI 2.5.4-3S, the staff asked the applicant to demonstrate how BBM samples were obtained and what degree of disturbance was involved. In response, the applicant stated that soil borings into the BBM were drilled using mud rotary methods and SPT tests; split barrel soil sampling was conducted in accordance with ASTM D 1586, generally at 1.5 m (5 ft) intervals. The applicant noted that many of the split barrel samples obtained from harder layers or lenses within the marl were fractured by the sampling process, and some of these samples had the appearance of angular sands or gravels. The applicant obtained relatively undisturbed (intact) soil samples using a three inch diameter thin-walled Shelby tube sampler in accordance with ASTM D 1587. The applicant stated that, in general, the samples taken in the Upper Sands were obtained through the direct push method, whereas samples taken in the BBM and Lower Sands were obtained using a Pitcher sampler, which is recommended for hard or dense soils and soft rocks, in accordance with ASTM D 6169, due to the very hard/dense nature of these materials. The applicant also stated that undisturbed samples and tubes were inspected, sealed, and transported to the climate-controlled on-site storage area following ASTM D 4220 guidelines, and samples were transported to various off-site testing laboratories according to the applicant-approved subcontractor procedures for sample transportation, including transporting RCTS samples by automobile to Houston, Texas.

In follow-up RAI 2.5.4-22S, the staff asked the applicant to provide a description of the approved transportation procedures used to move RCTS samples from the site to a test facility. In

response, the applicant provided a copy of the applicant-approved subcontractor procedure (work instruction) for transporting undisturbed samples by automobile, which the staff determined provided adequate instructions for handling and securing the samples during transportation, consistent with standard industry and ASTM guidelines. Based on its review of the applicant's response, the staff finds that the applicant demonstrated its use of appropriate material sampling techniques using acceptable industry practices or standards. Therefore, the staff considers RAIs 2.5.4-3S and 2.5.4-22S resolved.

In RAI 2.5.4-6S, the staff asked the applicant to provide the basis for its determination of the design value for cohesion of the BBM of 478 kPa (10,000 psf) and to explain how this value is to be used. The staff indicated that it is important to understand the basis for this evaluation, whether any laboratory test data was available to support the proposed design value, and where in the facility evaluation the parameter would be used.

In response, the applicant reiterated that the design value of 478 kPa (10,000 psf) for cohesion of the BBM was based on evaluating empirical correlations and laboratory test data from the ESP geotechnical investigation that was previously presented in response to RAI 2.5.4-7. The applicant also stated that this design value for cohesion of the BBM was based on evaluating empirical correlations and laboratory test data from the ESP geotechnical investigation, including 15 UU tests. The applicant collected additional data during the COL investigation to verify the design value developed during the ESP investigation. The applicant conducted UU and CU triaxial tests at various confining pressures, with results suggesting that the shear strength of the BBM increased with confining pressure as expected. The applicant stated that the marl is located at an approximate depth of 27 to 50 m (90 to 165 ft) with a design ground water level at a depth of 16.7 m (55 ft), and a range of confining pressures, based on overburden conditions, of 320 to 646 kPa (6,500 and 9,700 psf). The applicant noted that within this range, UU test results yielded minimum shear strength of 81 kPa (1,700 psf) and a maximum of 560 kPa (11,700 psf) while the CU test resulted in a minimum value of 134 kPa (2,800 psf) and a maximum value of 1,541 kPa (32,200 psf) for shear strength at the range of confining pressure. The applicant also noted that previously determined confining pressures corresponded to the upper limit of 766 kPa (16,000 psf) used in conducting the UU and CU triaxial tests, and at the higher confining pressure, the average UU and CU test results are 411 and 713 kPa (8,600 and 14,900 psf), respectively. From a review of the field and laboratory test data, the applicant concluded that, regarding the design undrained strength value of 478 kPa (10,000 psf), UU and CU tests conducted at confining pressures of 766 kPa (16,000 psf), empirical correlation with N-values, and empirical correlation with shear wave velocity, all support the design value of 478 kPa (10,000 psf).

The applicant used undrained shear strength of the marl stratum to evaluate the bearing capacity of the nuclear island, incorporating the shear strength value into the calculation of allowable bearing pressure through superposition, as follows from the RAI response:

$$qo = c \cdot Nc \cdot \zeta c + q \cdot (Nq) \cdot \zeta q + 0.5 \cdot \gamma' \cdot B \cdot N\gamma \cdot \zeta\gamma \qquad (1)$$

where:
- qo = ultimate bearing pressure (ksf)
- c = soil cohesion (ksf)
- q = effective overburden pressure at bottom of foundation level (ksf)
- γ' = effective unit weight of soil (kcf)
- B = foundation width (ft) = 101 ft
- L = foundation length (ft) = 254 ft
- Nc, Nq, Nγ = bearing capacity factor

ζc, ζq, $\zeta \gamma$ = foundation shape factor

In this superposition analysis, the foundation is placed on a "strong" layer (compacted structural fill) that is underlain by a "weaker" layer (BBM). The capacity of the "strong" layer is evaluated alone to obtain qo'. The capacity of the "weaker" layer is evaluated alone to obtain qo". The governing capacity, q, is determined by evaluating the effect of the "weaker" layer on the bearing capacity by the following equation:

qo = qo".exp{0.67.[1+(B/L)].(H/B)} (2)
qa = qoFS, with Factor of Safety (FS) = 3

where: qo" = ultimate bearing pressure of the foundation sitting on the surface of the Blue Bluff Marl (ksf)

H = thickness of compacted structural fill between the bottom of the foundation and the top of the BBM (ft) (H=43.5ft)

qo = ultimate bearing pressure at the foundation level

qa = allowable bearing pressure at the foundation level

For the "strong" (backfill) layer where: Φ = 34o, γ'moist = 120 pcf, γ'sat = 130 pcf

Nc = 42.16	Nq 2 9.44	$N\gamma$ = 41.06
ζc = 1.28	ζq =1.27	$\zeta \gamma$ =0.84
q = 4.74 ksf	γ'= 0.076 kcf	

From equation (1)
qo'=0.0 x 42.16 x 1.28 + 4.74 x (29.44) x 1.27 + 0.5 x 0.076 x 101 x 41.06 x 0.84 ≈ 0 + 177.2 +132.4 = 309.6 ksf

For the "weak" (Blue Bluff Marl) layer where: c = 10 ksf

Nc= 5.14	Nq 1.0	$N\gamma$= 0.0	ζc= 1.08
ζq = 1.0	ζy= 0.84	q = 8.49 ksf	

From equation (1)
qo" = 10 x 5.14 x 1.08 + 8.49 x (1.0) x 1 = 55.5 + 8.5 = 64 ksf.

Through superposition using equation (2), the ultimate bearing pressure at the foundation level is:
qo = 64 x exp{0.67 x [1+(101/254)] x (43.5/101)) = 95.8 ksf

Thus, with a factor of safety of 3 and the su of the BBM = 10 ksf, the allowable bearing pressure at the foundation level is:
qa = 95.8/3 or 31.9 ksf

The applicant explained that it used the same method to evaluate the allowable bearing pressure for other pressures as well. Based on the AP1000 standard design, where foundation pressure is 411 kPa (8,600 psf), the applicant provided additional consideration of contact pressure of the foundation and contact pressure projected to the top of the BBM. The applicant explained its methodology as follows:

Foundation Load = area x foundation pressure = 254ft x 101ft x 8.6ksf = 220,625 kips

Foundation pressure influence at the top of BBM =
Foundation Load / projected area, so
220,625 / (297.5ft x 144.5ft) = 5.1 ksf

Where: projected area = {(L + 2(H x s)) x (W + 2(H x s))}
H = 43.5 ft
s = slope of zone of influence (lv:2h) = 0.5

In conclusion, the influence of the foundation load decreases with depth such that at the top of the BBM, the load has diminished by 41 percent (5.1/8.6). Based on the above, using su = 10 ksf for the BBM:

- With the NI founded on the fill, the FS against bearing failure is 958/5.1 = 18.8
- With the NI founded directly on the BBM, FS = 64/8.6 = 7.4

Using su = 6.5 ksf for the BBM:

- With the NI founded on the fill, the FS = 66.8/5.1 = 13.1
- With the NI founded directly on the BBM, FS = 44.6/8.6 = 5.2

Based on the applicant's response to RAI 2.5.4-6S, including the calculations the applicant presented in its response, the staff concludes that the applicant adequately explained the basis for the determination of the 14.8 kPa (10,000 psf) design value. This conclusion is based on data and assessments provided by the applicant, as verified by the staff's confirmatory calculations and review of the laboratory triaxial test data provided in SSAR Revision 4. Furthermore, based on the applicant's response and review of the calculations presented, the staff concludes that the applicant explained how the 478 kPa (10,000 psf) design value will be used in the calculation of a factor of safety against bearing failure. However, although the staff was able to resolve most issues related to RAI 2.5.4-6S, the staff noted some areas of additional concern. The staff noted that the applicant's response to the RAI addressed only static bearing capacity evaluations for failure conditions; settlement considerations, which normally control the allowable pressures under large rigid basemats, were not included in the calculations. The staff also noted that the response did not address dynamic effects, which are the overwhelming effects on the computed toe pressures, and the staff requested the additional information. In follow-up RAI 2.5.4-24S, the staff requested that the applicant provide information addressing settlement considerations for static bearing capacity evaluations, and dynamic effects on the computed toe pressures.

In response to RAI 2.5.4-24S, the applicant stated that additional static and dynamic bearing capacity evaluations were underway, including localized punching failure of backfill materials supporting the nuclear island. The applicant conducted these assessments as part of the Phase 1 test pad program and used conventional analyses assuming safety factors of 3 and 2, for static and dynamic bearing capacity, respectively. Finally, the applicant evaluated settlement characteristics of the site and included all results in the revised SSAR.

The staff reviewed the applicant's response to RAI 2.5.4-24S, in particular the additional information and evaluations provided in the revised SSAR. The applicant stated that the soils supporting the nuclear islands did not exhibit extreme variations in subgrade stiffness and that the proposed Vogtle site could be considered uniform. The applicant presented in Section 2.5.4.2.2.2 that subsurface data has disclosed that the Blue Bluff Marl has a nearly even top

over the length of the excavation footprints with relatively uniform thickness and consistent properties. Over this will be placed approximately 27.4 m (90 ft) of structural backfill that will be placed and compacted in level uniform lifts or layers. Results of the Phase 1 and 2 test pad program disclosed that the materials proposed for structural backfill have consistent engineering properties including density, shear wave velocity and N-values.

The applicant stated in SSAR Revision 4 that it based its allowable static bearing capacity values on Terzaghi's bearing capacity equations using an internal angle of friction of 36 degrees for the compacted backfill as developed from their field and laboratory testing program during the Phase 1 test pad program and COL investigation. The applicant evaluated the influence of the Blue Bluff Marl on the allowable bearing pressure using procedures outlined by Vesic, procedures which are acceptable to the staff as they are in common use. With a factor of safety of 3.0, the applicant determined that the site conditions provide an allowable bearing pressure of 1,627 kPa (34 ksf) under static loading conditions for the nuclear island. The staff concurs with this determination because the applicant used equations from Terzaghi and procedures from Vesic that are commonly used and widely accepted industry method.

The applicant also evaluated the allowable bearing capacity of the nuclear island under dynamic loading conditions, and again the methods of analysis were based on Terzaghi's bearing capacity equation for general shear using seismic bearing capacity factors from Soubra and Terzaghi's bearing capacity equation for local shear. Using a factor of safety of 2.25, the applicant determined that the site conditions provided an allowable bearing pressure of 2,010 kPa (42 ksf) under dynamic loading conditions for the nuclear island. Both the static and dynamic bearing capacity values are well below the minimums specified in Revision 15 of the AP1000 DCD. The staff concurs with the determination because again the applicant used widely accepted equations and factors for such evaluations.

Finally, the applicant conducted laboratory consolidation tests on relatively undisturbed samples of the Blue Bluff Marl and the Lower Sand Stratum, and the results confirmed the elastic behavior and very stiff and dense nature of the two strata. Also, the applicant's test pad program assessed the properties of the proposed compacted backfill and the results confirmed the very dense nature of the materials and showed that the expected performance under load will be similar to VEGP Units 1 and 2. The applicant performed a detailed settlement analysis using similar elastic properties used for the VEGP Units 1 and 2 and incorporated excavation, dewatering, and construction duration to determine basemat displacement histories. The applicant stated that the results predicted total settlement ranges of from 5.08 to 7.62 cm (2 to 3 inches), with an approximate tilt of .635 cm in 15.24 m (¼ inch in 50 ft), and a differential settlement between structures of less than 2.54 cm (1 inch). The applicant noted that these results are similar to actual movements measured for VEGP Units 1 and 2.

The staff concludes that the applicant provided sufficient information in response to RAI 2.5.4-24S to address both the static and dynamic bearing capacities for the materials supporting the nuclear island as it presented results based on site-specific test results input into equations and factors commonly in use to determine bearing capacities and settlements. Therefore, the staff considers RAI 2.5.4-24S closed. Furthermore, the closure of RAI 2.5.4-24S also resolves RAI 2.5.4-6S.

The staff finds that the applicant conducted a subsurface investigation program consistent with the guidelines presented in RG 1.132 to adequately characterize the subsurface conditions and materials, and it performed laboratory testing consistent with the guidelines presented in RG 1.138 to adequately determine the engineering properties of the subsurface materials and

used the results to perform analysis to predict how the site conditions will support the AP1000 design requirements as presented in Revision 15 of the AP1000 DCD. Based on the information and findings above, including the resolution of RAIs, and the closure of Open Items, the staff concludes that the discussion of the properties of Subsurface Materials is acceptable.

The applicant determined the static and dynamic properties of the three principal soil groups and compacted structural backfill through its field investigations and through laboratory testing performed in accordance with RG 1.138. The staff concludes that the applicant complied with the relevant guidance of RG 1.138.

In Revision 4 of the VEGP SSAR, the applicant included information on the chemical tests performed on the engineered backfill for the VEGP Units 3 and 4 site. These tests are summarized in Section 2.5.4.2.2 of this SER and Subsection 2.5.4.2.5.3 of the SSAR, and included pH, chloride, and sulfate tests. The applicant stated that, due to the high concentration of sulfate in the Upper Sand Stratum, switchyard and borrow area 4, the concrete placed at the site would face mild exposure to sulfate attack. However, since the most potentially corrosive unit, the Upper Sand Stratum, would be completely removed during site excavation, the staff does not consider the exposure to sulfate attack to be a significant issue at the VEGP Units 3 and 4 site. Since the applicant included the results of chemical tests as part of the revised SSAR, the staff concludes that COL Action Item 2.5-3, as identified in the SER with Open Items, is no longer needed.

The staff concludes that the applicant's description of the subsurface materials was acceptable in that 1) the applicant, following the guidance of RG 1.132 and RG 1.138, investigated and tested the subsurface materials to determine that the soils encountered were alluvial and coastal plain sediments and characterized the soils as sands with silt and clay, the clay marl bearing layer, and underlying coarse to fine sand with interbedded thin seams; and 2) the applicant obtained sufficient undisturbed samples to allow for the adequate characterization of each of these soil groups and determine the extent, thickness, hardness and density, consistency, strength, and static design properties. The applicant also provided sufficient information in the form of plots, plans, and boring logs; and laboratory test results and summaries that enabled the staff to determine that the applicant had adequately characterized the subsurface soils and rock materials and determined their engineering and design properties. Therefore, the staff concludes that the applicant's description of the subsurface materials and their properties at the site of VEGP Units 3 and 4, per the information obtained from the ESP, COL, and LWA investigations, is acceptable. This conclusion is based on the information and findings above, including the resolution of RAIs and Open Items, and the addition of information to the revised SSAR that rendered COL Action Items unnecessary.

2.5.4.3.3 Exploration

The staff's evaluation of the information provided in support of the ESP application is as follows:

Section 2.5.4.3 of NUREG-0800 directs the staff to compare the applicant's plot plans and profiles of Seismic Category I facilities with the subsurface profile and material properties. Based on the comparison, the staff can determine whether (1) the applicant performed sufficient exploration of the subsurface materials and (2) the applicant's foundation design assumptions contain an adequate margin of safety.

In RAI 2.5.4-20, the staff asked the applicant to justify why it did not provide the relationship of foundations to the underlying materials in the form of plot plans and profiles, the foundation

stability with respect to ground water conditions, and a detailed dewatering plan. In its response, the applicant stated that it would provide this information as part of a COL application once more details become available regarding the foundation and site interaction. The staff concurs with the applicant that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied. Accordingly, in the SER with Open Items, this was identified as COL Action Item 2.5-4. However, later revisions of the SSAR by the applicant included details of the foundation and site interaction, such as plot plans and profiles showing the relationship of the foundations in relation to the underlying materials, in particular boring location plans, boring logs and subsurface profiles, site cross-sections, shear wave velocity measurements and profiles, shear modulus and damping curves, and power block excavation sections. The applicant also provided sufficiently detailed discussions of the ground water conditions, including liquefaction analyses, and provided details about its proposed dewatering system. Accordingly, the staff concludes that the inclusion of COL Action Item 2.5-4 is no longer necessary.

MACTEC Reports

In SSAR Section 2.5.4.3, the applicant heavily referenced a MACTEC report included as an appendix to the application. In RAI 2.5.4-17S, the staff asked the applicant to provide a description of the refraction microtremor (ReMi) testing method used for site geophysical testing as discussed in the MACTEC Report. The staff specifically requested information detailing the application of this method in determining S- and P-wave velocity profiles; the staff also asked the applicant to provide a justification to demonstrate the adequacy of using these data to determine site properties and the resulting impact on response analysis.

The applicant responded to RAI 2.5.4-17S by stating that ReMi testing was conducted in the power block areas for Units 1 and 2, and in the footprint area for Units 3 and 4. The applicant also stated that the original intent was to establish a shear wave velocity profile using this data; however, during collection, it became apparent that the vibration frequency of the existing plant equipment was interfering with the results. After attempts to overcome the interference were unsuccessful in the field, the applicant consulted with Dr. K.H. Stokoe to review the results, who expressed doubt that the results represented the true profile. Therefore, the applicant concluded that the ReMi testing results should not be considered in the COL geophysical survey. The staff reviewed the applicant's explanation of the ReMi testing at the site, including the summary provided in Revision 4 of the SSAR. The staff concurs with the applicant and Dr. Stokoe's assessment that the test results do not truly represent the shear wave velocity profile at the site. The staff concludes that the applicant has provided sufficient information to clarify RAI 2.5.4-17S, and therefore the staff considers the RAI resolved because the applicant did not use the suspect test results.

In RAI 2.5.4-18S, the staff again referred to the MACTEC report, which indicated that Dr. K.H. Stokoe would review the RCTS data generated for appropriate use in the site evaluations. The staff asked for a description of the details, depth, and completeness of Dr. Stokoe's review. The applicant responded by clarifying that RCTS testing is performed by Fugro Consultants at their Houston, Texas facility, Dr. Stokoe was involved in the initial set-up and review of that facility. The applicant also clarified Dr. Stokoe's review role in that Dr. Stokoe reviewed each RCTS draft report to assure quality of the results. Dr. Stokoe also reviewed the laboratory procedures and setup prior to the commencement of RCTS testing. Additionally, the applicant stated that the geotechnical engineering contractor that was used, MACTEC, independently audited the Fugro facility and conducted surveillances of RCTS testing in progress.

The staff reviewed the applicant's response to RAI 2.5.4-18S, particularly the assurances from the applicant that the review of RCTS data by Dr. Stokoe, the foremost expert on the RCTS test method, would ensure that the quality of data generated was appropriate for use in site evaluations. The staff considered Dr. Stokoe's involvement in the initial setup and review of the Fugro RCTS testing facility and concludes that, based on the experience and expertise of Dr. Stokoe, the depth and completeness of Dr. Stokoe's review should ensure that quality information has been generated because Dr. Stokoe is the foremost expert on the RCTS test method. Furthermore, the staff concludes that the independent audit by the applicant's contractor, the leading expert on the test method in question, would further ensure quality of data. Therefore, the staff concludes that sufficient information and details were provided by the applicant to close RAI 2.5.4-18S.

The staff's evaluation of information provided in support of the LWA request is as follows:

In Revision 4 of the SSAR, the applicant provided additional figures of the plot plans and subsurface material profiles. The staff reviewed these figures and determined that because the applicant conducted its program following the guidelines presented in RG 1.132, and because the foundation design assumptions contain an adequate margin of safety consistent with regulatory guidelines and accepted industry practices, such as those developed by the U.S. Army Corps of Engineers (USACE) and delineated in the USACE Manual, Engineering and Design – Slope Stability, EM 1110-2-1902, Office of the Chief of Engineers, the applicant performed sufficient exploration of the subsurface materials. This information removed the need for COL Action Item 2.5-4, which the staff previously identified in the SER with Open Items.

The staff concludes that, based on the information and findings above, including the resolution of RAIs and Open Items, and the addition of information to the revised SSAR that rendered COL Action Items unnecessary, the discussion of the exploration of the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable for approval of both the ESP application and LWA request.

2.5.4.3.4 Geophysical Surveys

The staff focused its review of SSAR Section 2.5.4.3 on the adequacy of the applicant's geophysical investigations to determine the soil and rock dynamic properties. The applicant conducted three down-hole seismic CPT tests and five suspension P-S velocity tests during the ESP site investigation. The applicant compared the soil and rock dynamic properties obtained from these tests with the results from previous geophysical surveys conducted for Units 1 and 2.

In RAI 2.5.4-3, the staff asked the applicant to explain how the base case shear wave velocity profile was developed based on only 12 borings, since most of the borings did not go deeper than 91.4 meters (300 ft). The staff asked additional questions as part of RAI 2.5.4-3, which was discussed and evaluated in Section 2.5.4.3.2 of this SER. In response to RAI 2.5.4-3, the applicant stated that the base case shear wave velocity profile was developed in association with the Lisbon Formation (Blue Bluff Marl), Still Branch Formation, and the upper portion of the Congaree Formation based on the results of the three suspension P-S velocity logging tests performed at the ESP site. One of the suspension P-S velocity logging tests extended into bedrock below the Lower Sand Stratum, and the applicant used those results to derive the base case shear wave velocity profile below the top of the Congaree Formation. The applicant explained that the randomization model captures the uncertainty in the base case shear wave velocity profile for the in-situ soils. The applicant used logarithmic standard deviation of shear wave velocity as a function of depth, which was set to values obtained from soil randomization

performed at SRS. After reviewing the applicant's response, however, the staff found that shear wave velocities vary significantly among the three profiles (ESP, VEGP, Units 1 and 2 and SRS), with most terminating at a depth from 85.34 to 60.96 meters (280 to 300 ft)), and lower shear wave velocities measured from down-hole seismic tests than from the suspension P-S velocity measurements. Furthermore, the shear wave velocities from previous investigations were relatively lower than those obtained from the ESP investigations. Therefore, in the SER with Open Items, the staff concluded that the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile. This issue was identified in the SER with Open Items as Open Item 2.5-18.

In response to Open Item 2.5-18, the applicant stated that the shear wave velocity provided in the ESP was based on site-specific data from velocity measurements taken in the footprint of the ESP site. The applicant also described the development of the velocity profile, which used down-hole seismic CPT data and P-S velocity logging data for elevations above the BBM and P-S suspension logging measurements for elevations below and including the marl. The applicant gave consideration to profiles developed at nearby sites, such as Units 1 and 2 and SRS. However, although the profiles were consistent, they were not incorporated by the applicant into the ESP profiles. The applicant used additional data to re-evaluate the ESP profile following more detailed site investigations, and the applicant included these evaluations in the revised SSAR.

The staff focused its review on the additional information provided by the applicant in the revised SSAR, which included shear wave velocity profiles derived from the down-hole seismic CPT data, P-S velocity logging data, and P-S suspension logging measurements. The staff finds that the applicant's shear wave velocity testing through the ESP and COL subsurface investigations and during the 2 Phase test pad program demonstrated that the site and compacted structural backfill will support the DCD's required minimum shear wave velocity. Based on these revised profiles, illustrated in Figures 2.5.4-3 and 2.5.4-5 of this SER, the staff concludes that the applicant provided shear wave velocity profiles, derived from the results of ESP site investigations, that were sufficient to address the concerns of Open Item 2.5-18. Therefore, the staff considers Open Item 2.5-18 closed. Furthermore, the closure of Open Item 2.5-18 resolves the remaining portion of RAI 2.5.4-3 as it relates to geophysical investigations at the site of VEGP Units 3 and 4.

Based on the review of SSAR Section 2.5.4.4 and the applicant's response to RAI 2.5.4-3, described above, the staff concluded that although the applicant used various methods to determine compressional and shear wave velocities, including some of the latest technologies recommended in RG 1.132, the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile nor to address the velocity difference from different methods. However, in Revision 4 of the SSAR, the applicant provided additional information on the shear wave velocity measurements, including the use of multiple methods such as suspension P-S velocity tests, down-hole seismic tests with cone penetrometers, and, although unsuccessful, ReMi testing. Based on the review of SSAR 2.5.4.4 and the applicant's responses to the RAIs, the staff concludes that the applicant adequately determined the dynamic properties of soil and rock through its geophysical surveys at the site of VEGP Units 3 and 4 because the applicant conducted its exploration program following the guidelines in RG 1.132, which included fieldwork and laboratory testing performed under an approved quality program in accordance with approved industry standards and practices.

The staff concludes, based on the information and findings detailed above, including the resolution of RAIs and Open Items, that the discussion of the geophysical survey at the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable for approval of the ESP application and LWA request.

2.5.4.3.5 Excavation and Backfill

The staff reviewed SSAR Section 2.5.4.5, focusing on the applicant's description of anticipated foundation excavations for safety-related structures, backfills, and slopes; excavation methods and stability; backfill sources and quality control; and control of ground water during excavation. The applicant stated that the Upper Sand Stratum would be removed and replaced with Seismic Category I backfill from the top of the BBM to the bottom of the containment and auxiliary buildings at a depth of about 12.19 meters (40 ft) below the final grade. Backfilling would continue up around those structures to final grade. The excavation would be open-cut, with slopes no steeper than 2:1 (horizontal-to-vertical ratio). The applicant indicated that the guidelines used for VEGP Units 1 and 2 would be followed during the development excavation and backfill plans at the COL phase.

The staff's evaluation of the information provided in support of the ESP application is as follows:

Extent of and Plans for Excavation

Since there was no specific description of the excavation plans in the first revision of the SSAR, in RAI 2.5.4-2, the staff asked the applicant to clarify whether the excavation and backfill would only cover the footprint of the power block or would instead extend to a certain distance beyond the foundation footprint. In response to RAI 2.5.4-2, the applicant explained that safety-related footprints of the future Units 3 and 4 would have two respective backfilled excavations, and those excavations would extend beyond their respective power block footprints. The applicant established the minimum lateral extent of each excavation by determining the stress zone as defined by a 1:1 (horizontal-to-vertical) slope ratio, extending from the bottom of the turbine, containment, and auxiliary building foundations. The approximate bottom of the foundation elevations would be 65.8 meters (216 ft) above msl for the turbine building, 54.9 meters (180 ft) above msl for the containment, and 39.6 meters (130 ft) above msl to the top of the Lisbon Formation (Blue Bluff Marl) for the auxiliary buildings. The stress zone at the top of the Lisbon Formation would extend approximately 26.2 meters (86 ft) horizontally beyond the footprint of the power block structures. The applicant considered the turbine building foundation to be the governing factor of this horizontal extension (highest foundation); therefore, the 26.2-m (86-ft) extension was conservatively set for all four sides of the excavation. The applicant planned to backfill the entire excavation, including the power block footprint, stress zone, and areas beyond the stress zone, using compacted structural fill.

Due to the concern of a possible backfill impact on the seismic response evaluation of the site and structures, in RAI 2.5.4-2, the staff also asked the applicant whether it would implement the seismic hazard calculations to the free-ground surface, including the Barnwell Group in the base case site soil column, if the site excavations were not to extend significant distances to the side of the plant. In addition, the staff asked the applicant to explain the basis for its column analysis that presumed uniform backfill in all horizontal directions, while the actual excavation and backfill would extend only to the immediate vicinity of the plant. In its response, the applicant stated that the site excavations would extend to significant horizontal distances from the structures. With the base of the excavation extending approximately 26.2 meters (86 ft) outside of the building footprint, and with the excavation side slope ratio at 2:1(horizontal to vertical), the

structural backfill would extend more than 54.9 meters (180 ft) beyond the containment and auxiliary buildings at their foundation level and would extend more than 76.2 meters (250 ft) beyond the edge of the turbine building at its foundation level.

Since there was no specific description regarding the backfill compaction control, in RAI 2.5.4-2, the staff also asked the applicant to explain how compaction control would be implemented if the backfill was to contain as much as 25 percent fines content. In its response, the applicant stated that sand and silty sand with no more than 25 percent fines was obtained from onsite sources for use as backfill, as structural backfill for Units 1 and 2, and that it would use the same structural backfill criterion for Units 3 and 4. The applicant would also implement compaction controls for placement of the backfill through an independent soil testing firm. This testing firm would maintain an onsite soils testing laboratory to control the quality of the backfill material and the degree of compaction, and to monitor the compaction through field density tests performed at a minimum frequency of one test per 928 square meters (10,000 square ft) per lift of placed compacted backfill. In addition, the applicant committed to develop more detailed testing compaction control criteria during the COL phase. The applicant met this commitment through the testing performed during its Phase 1 and 2 test pad backfill program. At the time the SER with Open Items was issued, no site excavation or backfill had been performed; therefore, the staff considered this design-related information immaterial to determining whether 10 CFR Part 100 is satisfied at the ESP stage. Subsequently, the applicant performed additional subsurface investigations and laboratory testing to gather additional ESP and later COL data, which the applicant used to develop the later revisions of the SSAR and also defined the LWA portion of the activities to be the removal of the Upper Sand Stratum and excavation to the top of the Blue Bluff Marl bearing layer, placement of structural backfill to the bottom of the nuclear island foundation, installation of the concrete working surface mudmat and waterproofing membrane, installation of the MSE walls and accompanying waterproofing membrane around the perimeter of the nuclear islands, and backfilling around the outside perimeter of the MSE walls up to final plant grade.

After reviewing the responses from the applicant to RAI 2.5.4-2, the staff, in the SER with Open Items, concluded that, although the applicant provided more information on the extent of excavation, backfill material, and its compaction control, the applicant needed to consider some related issues during the COL stage including: (1) the stress zone described in the applicant's response to RAI 2.5.4-2 was based on normal static stress evaluations, but the applicant needed to consider both static and dynamic load induced stresses; and (2) since the applicant indicated that excavations would extend from about 26.2 meters (86 ft) outside of the building footprint with 2:1 (horizontal-to-vertical) side slope ratios and then extend away from the power block, the applicant needed to include the backfill material placed in and around the power block structures in the structural model when evaluating SSI, as indicated in the currently revised Section 3.7 of NUREG-0800. Thus, in the SER with Open Items, the applicant's commitment to provide detailed excavation and backfill plans during the COL stage was identified as COL Action Item 2.5-5.

Revision 4 of the SSAR contains detailed information on the excavation and backfill plans for the VEGP Units 3 and 4 site. The summary of these plans can be found in Section 2.5.4.1.5 of this SER. The applicant included discussions of the extent of excavations, methods and stability of excavations, backfill design and sources, quality control and ITAAC, groundwater control, and retaining wall plans. This information specifically fulfilled the level of detail specified by COL Action Item 2.5-5. Therefore, COL Action Item 2.5-5 is no longer necessary.

Geotechnical Parameters of Backfill Materials

Because the applicant did not describe the determination of shear wave velocity for the backfill, in RAI 2.5.4-4, the staff asked the applicant to explain how it would determine shear wave velocity values at depths of 15.2 meters (50 ft) and deeper for the backfill materials and whether it considered the effects of confinement. In its response, the applicant reiterated the statement of SSAR Section 2.5.2.5.1.2.1.1:

> Shear-wave velocity was not measured for the compacted backfill during the ESP subsurface investigation (APPENDIX 2.5A). Interpolated values based on measurements made on backfill for existing Units 1 and 2 (Bechtel 1984) are used instead.

The applicant also clarified that the measurements made of backfill soil for existing Units 1 and 2 were laboratory measurements using resonant column tests. The applicant developed shear wave velocity profiles for the backfill using equations presented in the response.

After reviewing the response to RAI 2.5.4-4, the staff found in the SER with Open Items that the applicant attempted to apply the estimated shear wave velocity from the backfill for the existing units to the backfill for the ESP site. But the equation used in the estimation dated back to the 1960s and there was significant variability, or uncertainty, for the parameter K2 in the equation. The calculation also did not account for confinement effects. Since the ability to show that the backfill meets the minimum shear wave velocity requirement with minimum in-situ variability is a major concern in the COL phase, and the procedures presented in the SSAR did not provide such information, the staff determined in the SER with Open Items that additional information to address the backfill shear wave velocity should be submitted in the COL application. Accordingly, this was identified as COL Action Item 2.5-6 in the SER with Open Items.

SSAR Revision 4 includes information on the applicant's test pad program, which was used to produce the site-specific data necessary to develop a shear wave velocity profile for the engineered backfill at the site. The applicant included the results of the test pad program in the revised SSAR, and the engineering properties, including shear wave velocity are found in Table 2.5.4-1 of this SER. The staff agrees that this information specifically addresses the needs of COL Action Item 2.5-6 because the information is specifically related to the actual materials the applicant planned to use for structural backfill and the shear wave velocity profile was developed for these proposed site-specific materials.

In summary, based on a review of SSAR Section 2.5.4.5 and the applicant's responses to RAI 2.5.4-2 and RAI 2.5.4-4 described above, the staff determined that the applicant did not initially provide detailed information on excavation and backfill plans due to the limited knowledge of the exact location of reactors and fill materials. Regulatory Position C.6 of RG 1.132 recognizes that there may be limitations on the extent of geologic mapping that may be performed prior to a site being approved under the 10 CFR Part 52 licensing procedures. To address this need for construction mapping, in the SER with Open Items, the staff proposed the inclusion of a permit condition requiring that the ESP holder or an applicant referencing the ESP perform geologic mapping of future excavations for safety-related structures, evaluate any unforeseen geologic features that are encountered, and notify the NRC no later than 30 days before any excavations for safety-related structures are open for NRC's examination and evaluation. Accordingly, this was identified as Permit Condition 2. However, geologic mapping of excavations was included within the scope of the LWA request, as was the evaluation of any unforeseen geologic features that may be encountered. Since this information is included within

the scope of the LWA request, the staff concludes that Permit Condition 2 is no longer necessary.

The staff's evaluation of the information provided in support of the LWA request is as follows:

Subsequent revisions to the SSAR included additional information for the staff to review regarding the excavation and backfill plans proposed in the LWA request for VEGP Units 3 and 4. During the review of the revised SSAR, the staff identified several areas requiring additional information.

Geotechnical Parameters of Backfill Materials

In RAI 2.5.4-7S, the staff requested that the applicant provide a discussion of the required shear wave velocity condition that needs to be met to ensure the backfill soil will satisfy the analysis criteria used for the SSI calculations of the AP1000 standard design. The staff asked that this discussion refer to both the minimum shear wave velocity and the acceptable variability of the measure velocity over the nuclear island footprint.

The applicant responded by stating that a description of the borrow sources could be found in its response to RAI 2.5.4-10S. The applicant also described the general backfill design program for Units 3 and 4 as being modeled after the program that was used for the existing units, and which included a limiting fines content of no more than 25 percent passing the No. 200 sieve (0.075 mm); the Proctor test was utilized as the compaction standard. Furthermore, the applicant provided a detailed description of the two-phase backfill test pad program, which was used to develop the site-specific backfill design to satisfy the standard plant design siting criteria in Revision 15 of the AP1000 DCD and to develop placement and compaction methodologies for the construction program. The applicant stated plans to use the results of these two phases to finalize the details of the backfill construction program, including material properties criteria, construction methods, compaction methods and requirements, and testing protocol, before describing the phases of the program in greater detail:

> Phase 1 will entail a test pad, constructed below grade, approximately [6 m] 20 feet thick using on site borrow from the switchyard area borrow source. The backfill will be placed in [15.24 cm] 6 inch loose lifts and compacted to 95 percent of the maximum dry density as determined by ASTM D 1557. The placement of the backfill will be comprehensively monitored and tested. During backfill placement, field testing will include compaction and shear wave velocity testing utilizing surface wave methods (SASW). Parallel testing will be performed in the laboratory for density, grain size, moisture, and plasticity. On completion of test pad construction, SPT borings will be drilled through the test pad and sampled continuously in the backfill and at [1.5 m] 5-foot intervals to a depth of [6 m] 20 feet in the in-situ soil. Shear wave velocity will be measured in the test pad using cross-hole techniques in accordance with ASTM D4428. Shear wave velocity measurements will also be taken at the finished surface of the test pad using surface wave methods. Results of the test pad field and laboratory measurements will be used to develop expected shear wave velocity characteristics of the backfill.

The applicant concluded by stating that the description of the shear wave velocity data developed during Phase 1 would be evaluated against the assumed shear wave and soil degradation characteristics of the backfill used in SSAR Revision 2, and if significant differences

were found, the SSAR would be revised. The applicant noted only minor differences and revised the shear wave profiles in the SSAR accordingly. The applicant later included the RCTS test results in Revision 4 of the SSAR. Finally, the applicant stated that the results of Phase 2 of the test pad program would be used to develop procedures, in accordance with the applicant's quality control program, to ensure that the backfill was placed as specified by design requirements, to minimize variability of backfill, and to achieve acceptable results as required by the AP1000 standard plant design.

During the review of the applicant's response to RAI 2.5.4-7S, the staff considered the information provided and, in follow-up RAI 2.5.4-25S, asked the applicant to explain how the limitation of 25 percent fines was selected, how different the fines content could be to still be acceptable, and how the acceptable ranges of fines were defined for the Phase I Test Pad program and the production of backfill. RAI 2.5.4-28S, which is discussed later in this section, also relates to the two-phase test pad program for backfill. In response to RAI 2.5.4-25S, the applicant stated that, based on studies, tests and analyses of the structural backfill used for Units 1 and 2, the maximum percent fines to minimize potential settlement of the backfill was 25 percent. The applicant also developed the grain size distribution envelope that met the prescribed criteria outlined in the SSAR for the proposed materials parameters, such as percent fines, and included the results of the settlement calculations using the geotechnical properties of the backfill in Revision 4 of the SSAR.

The staff also reviewed the explanation of the percent fines for the backfill and concludes that the use of 25 percent fines will minimize settlement of the backfill at the site of VEGP Units 3 and 4, because the proposed backfill materials are very similar to those used for Units 1 and 2, in which the materials performed acceptably, and 25 percent fines is a widely-accepted industry value for sands and silty sands. Therefore, the staff considers RAI 2.5.4-25S resolved. The staff considered the detailed description provided in response to RAI 2.5.4-7S, including the details and implementation of the two-phase backfill test pad program and inclusion of the subsequent test results in Revision 4 of the SSAR. The applicant was able to demonstrate through the Phase 1 and 2 test pad programs that, by keeping the fines content to less than 25 percent, placing and compacting the proposed materials to at least 95 percent of the modified ASTM D 1557 standard, and performing laboratory testing to verify moisture content, that the grain size distribution of the sands and silty sands did not fall outside of the proposed grain size envelope; therefore, structural backfill materials would meet the requirement for minimum shear wave velocity. The applicant verified this information through in situ testing of the placed and compacted backfill materials, and shear wave velocity testing utilizing the SASW method at various times during the construction of the test pad and again upon completion of the test pad. These test results indicated that, by employing uniform and consistent soil placement and compaction methods, as demonstrated by the applicant during the Phase 2 portion of the test pad program, the final compacted materials will meet the requirement for shear wave velocity. Based on this additional information, in conjunction with the resolution of RAI 2.5.4-25S, the staff considers RAI 2.5.4-7S resolved.

Similar to the issue the staff addressed in RAI 2.5.4-7S, in RAI 2.5.4-14S, the staff requested that the applicant provide a discussion of how velocity testing of the compacted backfill would be performed and what assurances would be provided to ensure, in the completed condition, that the resultant velocities will meet target velocity requirements. In response to this RAI, the applicant referred to the velocity testing of compacted backfill that would be performed as part of the two-phase backfill test pad program described in the response to RAI 2.5.4-7S. The applicant also stated that "assuring the in-placed backfill meets the backfill design and construction requirements will provide the assurance that the shear wave velocity profile of the

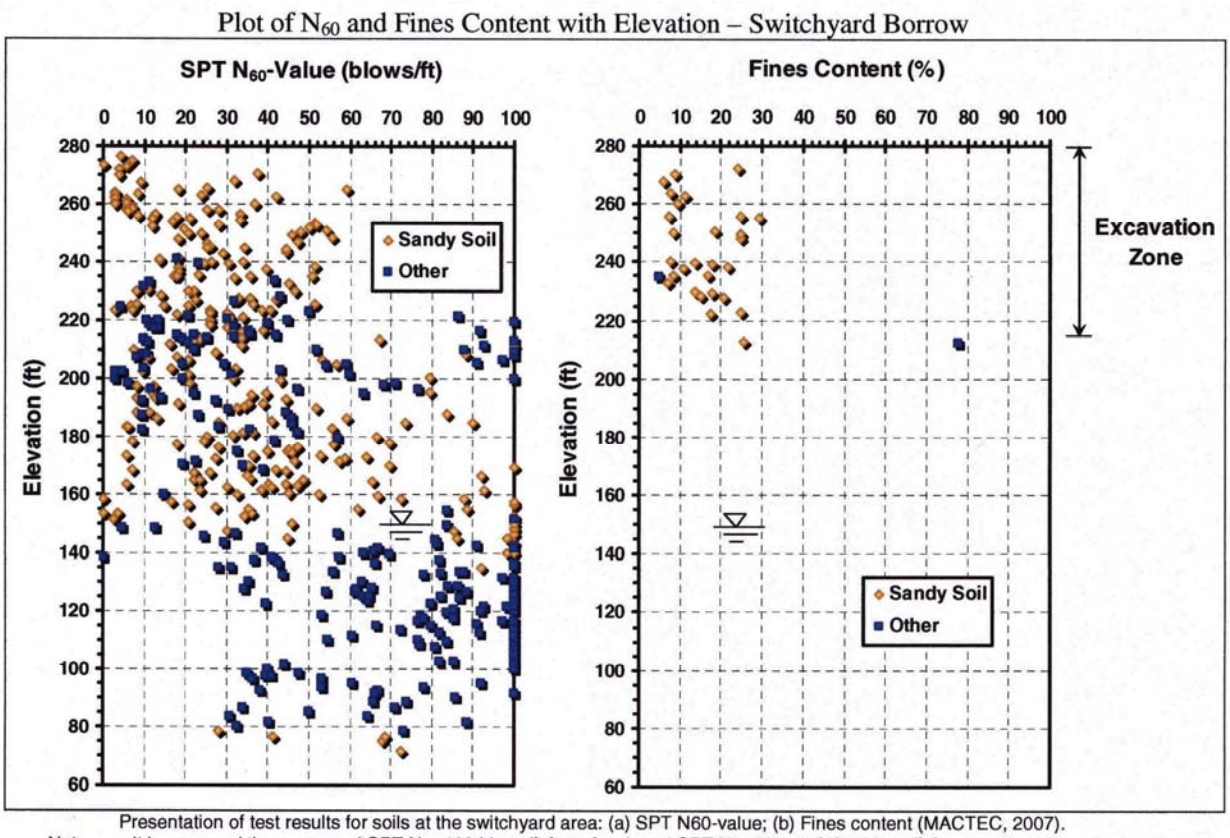
in-place backfill falls within an acceptable range consistent with the appropriate requirements stated in the Westinghouse DCD and the Vogtle site-specific analyses including the development of the GMRS and FIRS and the soil-structure interaction analyses."

The staff reviewed the applicant's response and the backfill test program described in response to RAI 2.5.4-7S. The staff observed significant portions of both Phase 1 and 2 of the test pad program and actual in situ SASW shear wave velocity testing conducted on the compacted backfill and reviewed laboratory test results, as documented in the trip reports from the staff's December 2007 and July 2008 visits to the VEGP site (ML080110651 and ML082280539). Based on the staff's observation of the applicant's structural backfill placement and compaction methodologies, the applicant's SASW shear wave velocity testing and results, and the applicant's proposed soil specifications arrived at through laboratory testing, the staff concludes that the applicant has provided assurance that, during construction activities, if the applicant meets its soils specification and follows its backfill placement and compaction procedures as determined during the two-phase test pad program, the applicable soil density and shear wave velocity requirements will be met as specified in the proposed backfill ITAAC presented in SER Section 2.5.4.1.5. Based on the resolution of RAI 2.5.4-7S and the acceptable shear wave velocity results presented in the revised SSAR and reviewed by the staff, as well as the assurances that the soil density and shear wave velocity requirements will be met and confirmed through ITAAC, the staff concludes that the applicant supplied sufficient information to resolve RAI 2.5.4-14S. The staff's further evaluation of the proposed backfill ITAAC is provided below in this section of the SER.

Volume and Sources of Backfill Materials

In SSAR Section 2.5.4.5.3, the applicant stated that the volume of material to be excavated at the site was approximately 2.98 million (M) cubic meters (3.9M cubic yards), which will require 2.90 M cubic meters (3.8 M cubic yards) of structural backfill. The applicant further stated that only 30 percent of the excavated material will be available for reuse as structural backfill. In RAI 2.5.4-10S, the staff asked the applicant to perform additional investigations and testing at both horizontal and vertical intervals sufficient to determine the material variability of the remaining 70 percent of borrowed soil that will be used for backfill.

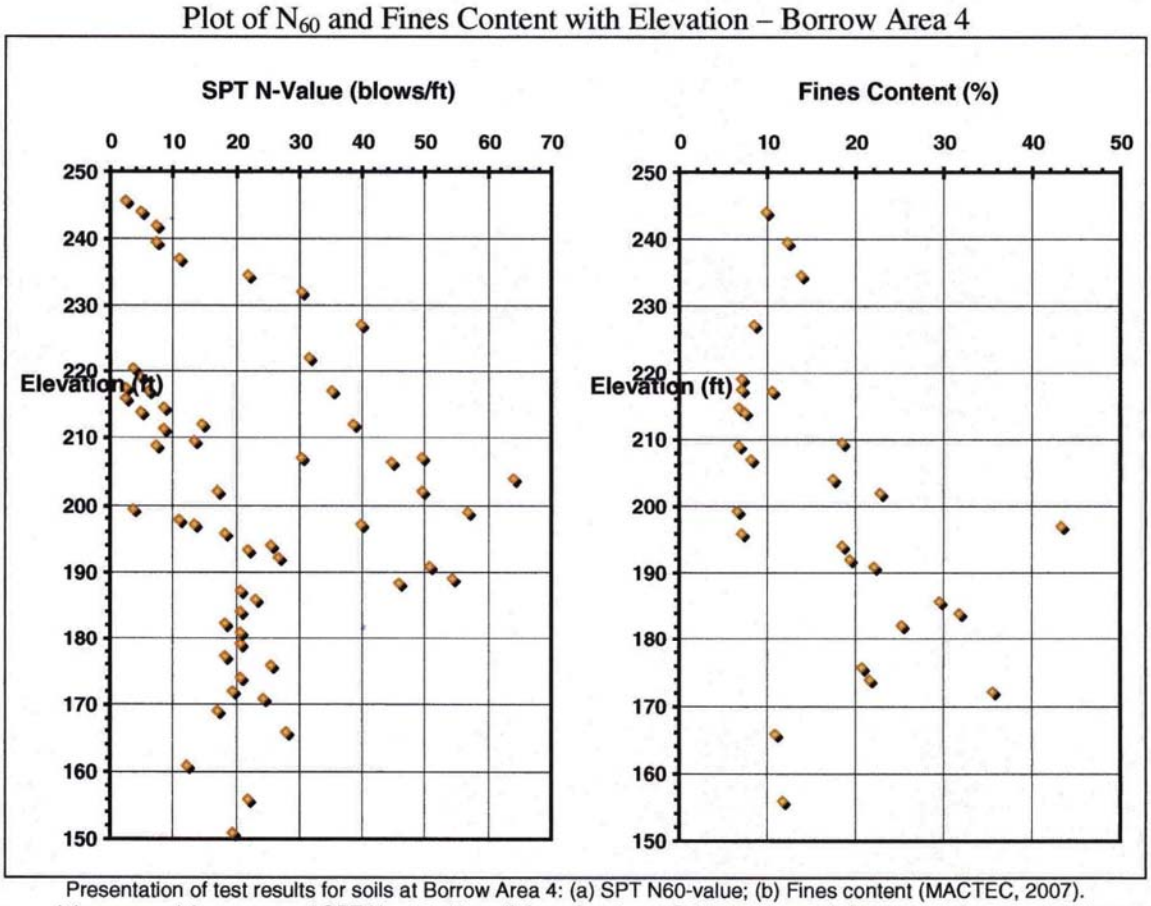
In response to this request, the applicant reiterated its previous conclusion that sufficient borrow material was identified at the site and that no additional investigations or testing was necessary. The applicant summarized the COL level investigation at the switchyard borrow area, including the results of 15 SPT borings that were drilled through these materials and five excavated test pits. Grain size, chemical tests, and compaction tests were part of the laboratory investigation described by the applicant for the borrow materials, an investigation which identified 1.9 million cubic meters (2.5 million cubic yards) of suitable borrow material. Again, the applicant referred to the backfill test pad program described in its response to RAI 2.5.4-7S for additional information on tests to be conducted on the borrow materials. Finally, the applicant described plans for investigations at an alternative borrow source, Borrow Area 4, which included four SPT borings and three test pits, and included preliminary comparison plats of N60 and Fines Content between the Switchyard Borrow area and Borrow Area 4 (SER Figures 2.5.4-8 and -9). However, in reviewing this response, the staff noted that survey results and/or figures were not provided to justify that sufficient material exists at the various borrow sources.



Presentation of test results for soils at the switchyard area: (a) SPT N60-value; (b) Fines content (MACTEC, 2007).

Note: It is assumed the measured SPT N = 100 blows/ft for refusal, and SPT N = 50 + 50/Δ/10 blows/ft for recorded blow counts of 50/(Δ ft). The energy-corrected SPT N_{60} does not exceed 100 blows/ft.

Figure 2.5.4-8 Plot of N60 and Fines Content with Elevation for Switchyard Borrow.



Presentation of test results for soils at Borrow Area 4: (a) SPT N60-value; (b) Fines content (MACTEC, 2007).

Note: It is assumed the measured SPT N = 100 blows/ft for refusal, and SPT N = 50 + 50/Δ/10 blows/ft for recorded blow counts of 50/(Δ ft). The energy-corrected SPT N_{60} does not exceed 100 blows/ft.

Figure 2.5.4-9 Plot of N60 and Fines Content with Elevation for Borrow Area 4

Accordingly, follow-up RAI 2.5.4-27S requested that the applicant provide clarification and justification of the quantity of suitable material in the switchyard area stockpiles, as well as describe how the percentage of reusable material excavated at the site was determined to be 30 percent. In response, the applicant stated that of the 2.75 million cubic meters (3.6 million cubic yards) of backfill required, two-thirds will come from the switchyard area and one-third will come from the power block excavations. The applicant also identified 1.5 million cubic meters (2.0 cubic yards) of additional borrow material available at Borrow Area 4 and from the power block excavation. Details of the two major sources of backfill, the switchyard and power block areas, were provided by the applicant as follows:

> Switchyard Area: A detailed geotechnical investigation of the switchyard area was performed to confirm the suitability of the material in this area for use as backfill. As discussed in SSAR Section 2.5.4.5.4, the subsurface conditions in this area were explored with 15 SPT borings and five test pits during the COL investigation. Laboratory testing was conducted on representative samples to determine their engineering characteristics and to assess their suitability for use as backfill. These data, along with the backfill criteria as discussed in SSAR

Section 2.5.4.5.3, were used to estimate the horizontal and vertical extent of suitable borrow material in the switchyard area.

The field and laboratory test data from the switchyard borrow area borings were compiled onto logs of the borings. This information was used to develop subsurface profiles through the switchyard area as shown on Figures RAI 2.5.4-27-2a and 2b [SER Figure 2.5.4-10]. The material identified as suitable for use as backfill is identified as the Sands 1 Belt on these profiles down to the rough grade excavation surface.

The volume of suitable borrow material was calculated using CADD. The surfaces of the suitable materials were projected from the profiles onto a 3-D plot of the borrow area and the volume of suitable material was determined to be approximately 2,400,000 cubic yards.

The surfaces of the Sands 1 Belt of suitable borrow material are relatively horizontal (not undulating); therefore, segregation of the suitable material from un-suitable material is not expected to be an issue.

Power Block Area: Field and laboratory data were used to develop subsurface profiles in the power block excavation area. A total of 70 SPT borings in this area were considered, along with borings outside this footprint to add additional data and clarity to interpretation of the subsurface conditions.

Engineering judgment was used to correlate the layers of suitable borrow material identified in the borings for use in developing 3-D CADD surfaces. The Sands 1, Sands 2, and Sands 3 layers constitute suitable borrow material. The total quantity of this borrow material in the excavation calculated using CADD is approximately 2,000,000 cubic yards, see Figures RAI 2.5.4-27-1a, 1b [SER Figures 2.5.4-11], and 1c [SER Figure 2.5.4-12].

Prior to utilization of the subsurface data from the borings, approximately 30 percent of excavation materials were judged to be suitable material for backfill. However, analysis of the subsurface data indicated that over 50 percent of the material was suitable. For estimating purposes, the original conservative estimate of approximately 30 percent (1,200,000 cubic yards) has been maintained for use as backfill. The remaining 800,000 cubic yards of suitable borrow material will be segregated and stockpiled for potential future use.

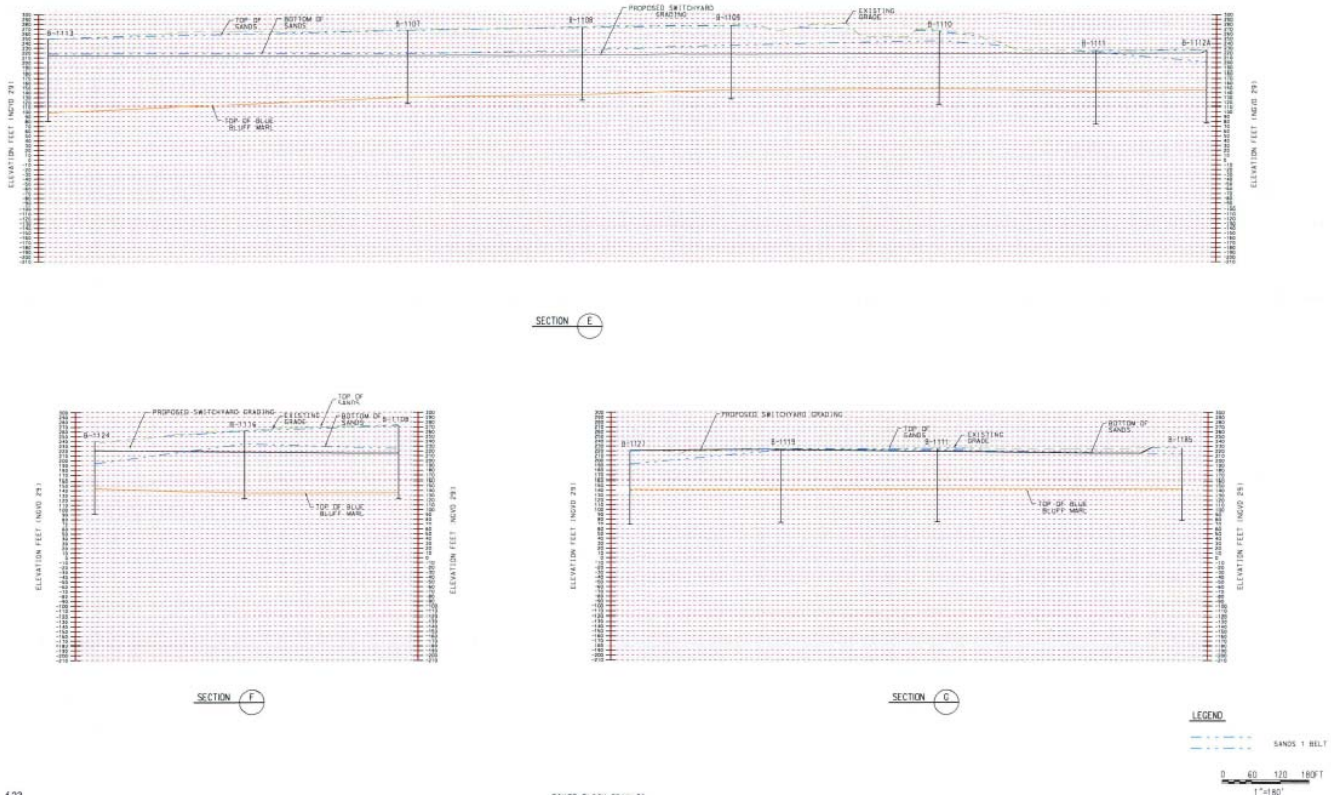


Figure 2.5.4-10 Switchyard Profiles Section E, F and G (Taken from RAI Response Letter #10)

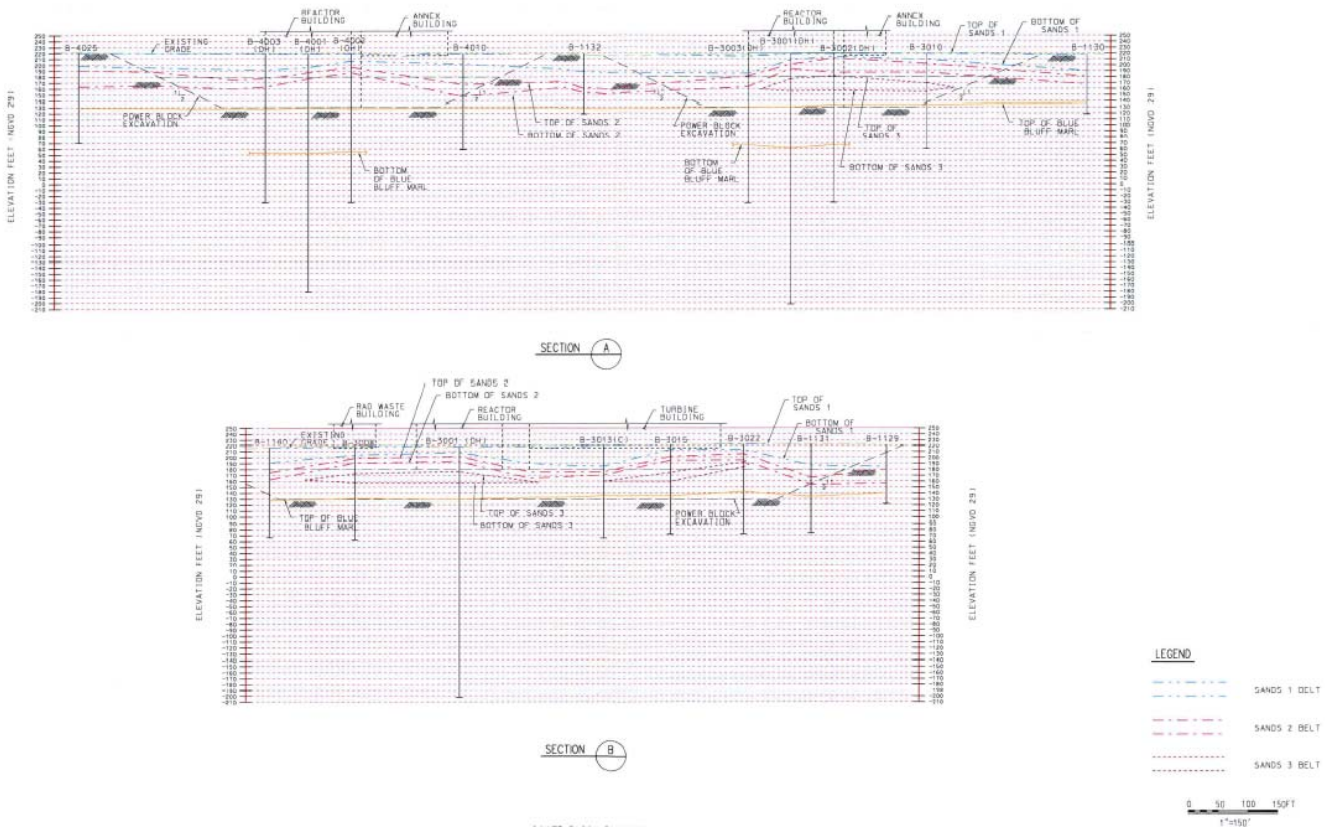


Figure 2.5.4-11 Power Block Profiles Sections A and B (Taken from RAI Response Letter #10)

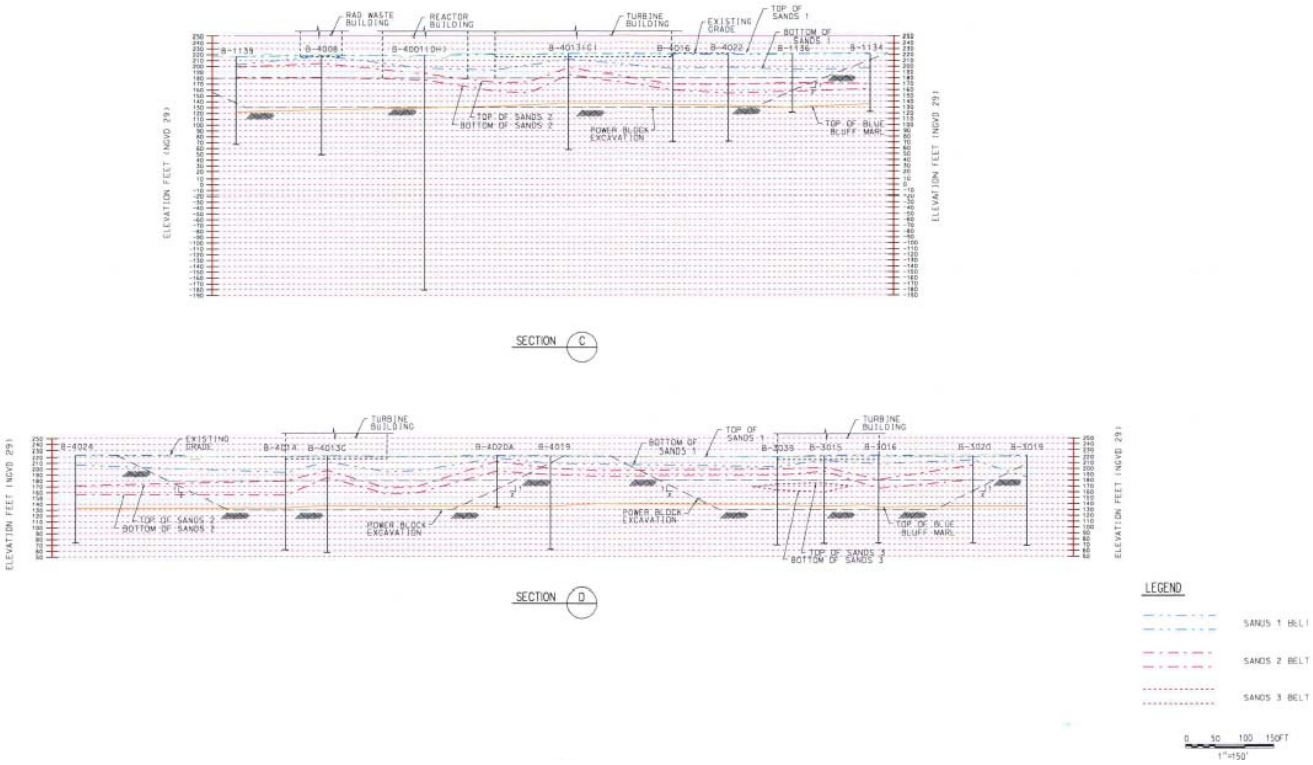


Figure 2.5.4-12 Power Block Profiles Sections C and D (Taken from RAI Response Letter #10)

The staff reviewed the information regarding the determination of borrow material availability at the VEGP site, including the site maps provided to support the applicant's conclusion that sufficient borrow material exists in two areas at the site to be used as structural backfill. Based on its review, the staff concurs with the applicant that sufficient borrow material is available at the site based on the applicant's exploration through borings and laboratory testing to adequately determine the horizontal and vertical extent of acceptable materials, and the applicant's use of computer-aided design and drafting (CADD) to calculate the volume of suitable materials. As such, the staff concludes that RAI 2.5.4-27S is resolved. Given this resolution of RAI 2.5.4-27S, combined with the applicant's description of laboratory tests to determine the variability of borrow material at the site provided in response to RAI 2.5.4-10S, the staff agrees with the applicant's subsurface investigation and laboratory results and the method of calculating material quantities, and concludes that the applicant provided sufficient information to describe the variability and availability of borrow material at the site of VEGP Units 3 and 4. Therefore, the staff considers RAI 2.5.4-10S to be resolved.

In SSAR Subsection 2.5.4.5.3, the staff reviewed the information provided regarding the control of the uniformity of the backfill. Included in this review, the staff considered any plans regarding grain size tests, maximum dry density and optimum water content. In RAI 2.5.4-11S, the staff asked the applicant to ensure that the backfill underneath and to the sides of the nuclear island satisfies the AP1000 SSI criteria by providing a description of the program needed to assure the correlation of grain size distribution of the borrow material, and the corresponding maximum dry density and associated shear wave velocity is defined.

The applicant responded to RAI 2.5.4-11S by referring to the two-phase test pad backfill program that was described in the response to RAI 2.5.4-7S, which evaluated the range of acceptable backfill material properties at the site, including the maximum dry density and optimum water content for backfill, properties related to the grain size distribution, density, and shear wave velocity. According to the applicant, the test program would also specify the material property and field and laboratory testing criteria to ensure that the material would conform to the AP1000 standard plant criteria included in Revision 15 to the DCD.

The staff reviewed the backfill test program described in response to RAI 2.5.4-7S and referenced in response to RAI 2.5.4-11S, focusing its review on the correlation of grain size distribution, maximum dry density and shear wave velocity. The staff concluded, through review of the applicant's laboratory test results and results of the two-phased test pad program, that the applicant thoroughly characterized the material properties of the proposed structural backfill materials according to the guidance presented in RG 1.138. With the field density and shear wave velocity testing conducted during the two-phase test pad program, the applicant demonstrated that the soil placement and compaction methodology developed during the test pad program will ensure that soil specifications, resulting from its laboratory and field testing, will result in a uniformly placed and compacted backfill program that will meet the standard plant criteria in AP1000, as considered by the staff in review of the applicant's response and activities to address the RAI condition. Accordingly, the staff concludes that the applicant's plans to use the results of the test pad program to determine the final material properties and soil specifications for the compacted backfill are sufficient to ensure the appropriate correlation between material properties and soil specifications from the laboratory and field testing at the site, as well as to ensure conformance with the standard plant criteria. Therefore, the staff considers RAI 2.5.4-11S resolved.

Flowable Fill

In SSAR Subsection 2.5.4.5.3, the applicant indicated that a flowable fill would be used in place of compacted backfill to a very limited extent. In RAI 2.5.4-12S, the staff asked the applicant to specify: 1) the target properties of this material; 2) the required uniformity of the target properties; 3) the relationship of the flowable fill to the remainder of the compacted backfill; and 4) the potential extent of the material's use. The applicant provided a four-part response that addressed each portion of the RAI individually.

With respect to the target properties of the flowable fill, the applicant provided both the expected unit weight (1,922 to 2,242 kg/m^3 (120 to 140 pcf)) and the shear wave velocity, which would be determined empirically following the equation Vs = (G0/p)0.5 where Vs = shear wave velocity, p is soil density determined from unit weight of soil, and G0 is shear modulus. The applicant then addressed the required uniformity of the properties, which it indicated would be adjusted to meet the strength requirements of the particular application. In an effort to maintain uniformity, the applicant described plans to produce the flowable fill in a ready-mixed concrete batch plant and transport the fill material using standard concrete mixing trucks to minimize the potential for component separation. The applicant also plans that most uses of the flowable fill at the site would be well removed from safety-related structures of the proposed units, but regardless of its eventual use, all flowable fill constituents, mix design, and placement will be controlled by widely-used industry specifications and procedures, the uses and locations of which will be documented on drawings. Regarding the relationship between flowable fill and the compacted backfill, the applicant stated that the flowable fill would have a higher load-bearing capacity, higher unconfined compressive strength, and greater bearing strength than the compacted backfill. Finally, the applicant addressed the potential extent of flowable fill at the VEGP Units 3 and 4 site, noting that flowable fill would be used where placement, compaction, and testing of compacted backfill was difficult. As was stated in response to the uniformity of the flowable fill, the applicant stated that flowable fill will be used at locations where the placement of soil backfill would be difficult or impractical to place and that those applications would be around piping, sewer and utility trenches, pipe bedding and slope stabilization well removed from the safety-related structures of the AP1000 units. Some potential locations where flowable fill may be used, as identified by the applicant, included the backfilling of sewer and utility trenches, road base, pipe bedding, and slope stabilization.

The staff considered the target properties and uniformity of the flowable fill, as well as the relationship to compacted backfill and potential extent of flowable fill at the site, provided in response to RAI 2.5.4-12S. The staff concludes that the applicant adequately addressed all aspects of the RAI by explaining the inclusion of target properties, its plans to maintain uniformity of fill, flowable fill's relationship to other backfill materials, and the extent of its usage at the site as described above; the staff therefore considers RAI 2.5.4-12S resolved, because while any use of flowable fill will be determined later, it will be controlled by specifications, procedures and drawings in accordance with the applicant's approved quality program.

Compaction of Backfill

SSAR Subsection 2.5.4.5.3 describes the classification of the backfill soils, including the percent compaction for each of the two categories. The applicant stated that the Seismic Category 1 backfill would be compacted to an average of 97 percent compaction, with no more than 10 percent of field compaction below 95 percent of the maximum dry density, while the Seismic Category 2 backfill would be compacted to an average of 93 percent, also with no more than 10 percent of field compaction below 95 percent. In RAI 2.5.4-8S, the staff asked the applicant to: a) correlate between density and velocity to ensure site characteristics and backfill requirements are met; b) justify how the 93 percent compaction minimum under Seismic

Category I structures would not adversely impact soil density to the point the shear wave velocity falls below the minimum requirement; and c) justify how the average dry density of Seismic Category 2 backfill will meet the 95 percent compaction requirement that no more than 10 percent would fail below 95 percent.

The applicant provided a three-part response to RAI 2.5.4-8S, each part addressing one aspect of the RAI. First, the applicant responded to the correlation between velocity and backfill design and construction requirements. The applicant stated that this correlation was based on the two-phase backfill and test pad program described in response to RAI 2.5.4-7S. The program resulted in detailed design and construction parameters, including backfill selection criteria, placement techniques, compaction methods and requirements, and testing protocol, which the applicant then used to assure the expected shear wave velocity profile would be achieved. In response to the second part of the RAI, regarding minimum compaction requirements of the backfill, the applicant revised the backfill compaction specification to a single compaction requirement for both Seismic Category 1 and 2 backfill. The applicant stated that the criteria were revised to be 95 percent of the maximum dry density per the modified Proctor compaction standard as described and determined in accordance with ASTM standard D 1557, which should provide uniformity in placement and strength of the backfill. Finally, the applicant justified the average dry density of Seismic Category 2 backfill by stating that the same compaction requirements of Seismic Category 1 backfill would be applied to Seismic Category 2 and the 93 percent compaction requirement would be deleted; density for all backfill will be as required and verified by the backfill ITAAC presented in Section 2.5.4.1.5 of this SER and evaluated in the following section of this SER.

The staff focused its review of this additional information on the correlation of density and velocity, and the revision of the Seismic Category 2 backfill criteria to mirror that of Seismic Category 1. The staff noted that the change in the compaction and density requirements of Seismic Category 2 backfill to match the engineering criteria of Seismic Category 1 results in location being the only difference between Seismic Category 1 and 2 backfill. That is, Seismic Category 1 backfill will be beneath the Seismic Category 1 (safety-related) structures, and Seismic Category 2 backfill, although engineered to the same criteria as Seismic Category 1, will be beneath the Seismic Category 2 (non-safety-related) structures. The staff concludes that the applicant's plan to utilize the backfill and test pad program described in response to RAI 2.5.4-7S to correlate shear wave velocity to density is an acceptable plan to address the required correlation because shear wave velocity and density are functions of each other, i.e., the denser a material is generally, the higher the shear wave velocity. Furthermore, the staff concludes that the revision of Seismic Category 2 requirements to reflect the compaction requirements of Seismic Category 1 backfill is sufficient to address the compaction concerns raised for Seismic Category 2 backfill because both materials will be placed and compacted to an industry accepted minimum density in accordance with the backfill ITAAC evaluated in the following section of this SER. Based on these conclusions, the staff considers RAI 2.5.4-8S resolved.

SSAR Subsection 2.5.4.5.3 states that the two categories of backfill will be compacted to the Proctor density requirements given based on tests performed at a density of one test per 929 square meters (10,000 square feet). In RAI 2.5.4-9S, the staff requested that the applicant provide the basis for using a testing density of one test per 929 square meters (10,000 square ft) of lift and to explain how this distribution will provide assurance of adequate uniformity of shear wave velocity as used in the SSI analyses of the AP1000 standard design. The applicant responded by describing an evaluation that, with respect to justifying the testing frequency for performing field density testing of engineered backfill, would use the recommendations of ASME

NQA-1-2004. The applicant revised the ESP application to conform to the testing frequency recommended by the aforementioned ASME code. Once again, the applicant referenced the backfill testing program described in response to RAI 2.5.4-7S and stated that the use of the ASME code for quality assurance requirements would provide an acceptable and consistent industry testing frequency for the development of the final construction specifications. The staff considers the applicant's utilization of ASME NQA-1-2004 as the recommended testing frequency for mass earthwork at nuclear facilities to be a suitable testing frequency for the density tests to assure uniformity of shear wave velocity as applied to the SSI analyses of the AP1000 standard design. In follow up RAI 2.5.4-26S, the staff requested that the applicant provide further clarification of how the ASME standard referenced in the response to RAI 2.5.4-9S will be implemented, and to provide justification of the testing density and how the applicant will ensure adequate uniformity of shear wave velocity.

In response to this supplemental request, the applicant stated that both the 152 cubic meter (200 cubic yard) criteria and lift criteria will be applied and that the backfill testing program will provide the necessary assurance that the backfill will achieve the required shear wave velocity at the nuclear island foundation. The applicant further stated that the testing density for mass earthwork was consistent with the guidance of NRC Inspection Manual, Inspection Procedure 88131, which references the test frequency (testing density) of ASME NQA-1 initially cited by the applicant. The applicant then described the testing frequency in greater detail, stating that "early during placement of the production backfill, the frequency of field density testing is expected to exceed the minimum frequency until sufficient data are developed to document that the required degree of compaction is consistently being achieved, based on field engineering judgment." The applicant also made comparisons to the frequency of testing for the MOX facility at the Savannah River Site and the National Enrichment Facility in New Mexico. The applicant concluded that a higher frequency of in-place testing was required depending on the size of the area; six nuclear tests per lift for areas between 1858 and 5574 m^2 (20,000 and 60,000 ft^2), four tests per lift for areas between 929 and 1858 m^2 (10,000 and 20,000 ft^2), and three tests per lift for smaller areas.

During the review of RAI 2.5.4-26S, the staff focused its review on the applicability of ASME NQA-1 to nuclear power plant sites. The staff agrees with the use of the criteria from the inspection manual as it specifies testing frequencies consistent with those used successfully at other nuclear facilities. Based on the applicant's reliance on the code in question in the NRC Inspection Manual, Inspection Procedure 88131, as well as the comparison to other facilities handling special nuclear material, and the applicant's proposed backfill ITAAC, evaluated in the following section of this SER, whereby it will prepare final reports documenting the minimum 95 percent compaction and shear wave velocity equal to or greater than 304.8 m/s (1,000 fps) requirements, the staff concludes that the applicant adequately justified the testing density used to resolve RAI 2.5.4-26S. With the resolution of RAI 2.5.4-26S, the staff also considers RAI 2.5.4-9S resolved.

Backfill ITAAC, Test Pad Program and MSE

While reviewing the excavation and backfill section for the VEGP Units 3 and 4 site, the staff also considered the applicant's discussions of its proposed ITAAC for backfill soil, which is provided in table 2.5.4-2 from Section 2.5.4.1.5 of this SER.

In RAI 2.5.4-15S, the staff asked the applicant to address the following four issues: 1) include the requirement of minimum shear wave velocity of 304 m/sec (1,000 ft/sec) in the Design Requirement; 2) provide a detailed description of the testing program for the placement of the backfill materials as part of the inspections and tests; 3) describe the report that is referenced in the Acceptance Criteria; and 4) include the minimum shear wave velocity of 304 m/sec (1,000 ft/sec) in the Acceptance Criteria.

In its response, the applicant addressed all four issues simultaneously by stating that SSAR Subsection 2.5.4.5.3.2 would be updated to provide additional discussion of the design of engineered backfill. In Revision 4, the applicant revised the SSAR to include a description of the test pad program and RCTS testing to provide assurances that the minimum shear wave velocity would be met. With respect to the backfill ITAAC, the application stated that the conformance to shear wave velocity would be demonstrated through the test pad program and not through the ITAAC process. In reviewing this information, the staff determined that it was not inherently clear whether the normal variability would be sufficiently evaluated without adequate shear wave velocity testing. Accordingly, in follow-up RAI 2.5.4-28S, the staff asked the applicant to justify the adequacy of the production backfill test program to estimate the average velocities of placed soils and their variability.

The applicant replied by referring the staff to the response given for RAI 2.5.4-19S and to a structural backfill evaluation report it submitted with the RAI responses. In RAI 2.5.4-19S, the staff asked the applicant to address two issues related to MSE wall backfill placement and footing construction. On the first issue, the staff asked the applicant to provide information on how the procedures modified from Phase I of the test pad program and revised compaction procedures from Phase II would be developed, to indicate whether a section of the MSE wall would be included in Phase II, and if so, to explain how compaction around the wall would be accomplished. The staff also requested confirmation from the applicant that the procedures developed at the end of Phase II would be used during the placement of production backfill. Finally, the staff asked for information on how the soil wave velocity testing would be accomplished during the placement of the production backfill in and around the final nuclear island configuration.

In response to the first issue, the applicant stated that Phase II of the test pad program would focus on the establishment of placement procedures and equipment to be combined with the Phase I results to develop backfill specifications and procedures, including frequency and type of quality control testing. Based on preliminary testing as part of Phase I of the test pad program, the applicant concluded that shear wave velocity testing during production fill placement would not be necessary since the results of the test pad program indicated that proper controls on backfill gradation and compaction would result in a homogenous fill with minimum shear wave velocity meeting the criteria of the AP1000 DCD. The staff reviewed this information, particularly the conclusion that shear wave velocity testing would not be needed during placement of fill because the applicant intends to use its specific backfill placement and compaction procedures developed during the test pad program, in conjunction with its laboratory testing program, to control the structural backfill gradation and compaction density to produce a homogeneous soil backfill foundation that will result in a minimum shear wave velocity at the foundation level of the NI that meets the AP1000 DCD criteria. Thus, because the applicant will verify and document the shear wave velocity as required by ITAAC, the staff concludes that the applicant provided sufficient information to resolve the first issue of RAI 2.5.4-19S.

On the second issue of RAI 2.5.4-19S, the staff requested that the applicant describe in detail the concrete footer that will be installed at the start of construction of the MSE wall. The staff noted that this description should include such parameters as concrete mix design, and reinforcing steel sizes so that the staff could determine the adequacy of the design. The applicant responded that the MSE wall is an internally stabilized system of panels that would act as forms for pouring the nuclear island structures. In order for the panels to be erected, the applicant explained that a thin leveling pad, or footer, is needed to provide a stable working surface from which the panels can be erected. The applicant stated the specifications of the footer, including the 28-day concrete strength, which would be 17 MPa (2,500 psi) or above, and the dead-load pressure of the wall (less than 275 kPa (40 psi)). The applicant further stated that reinforcing steel will not be needed since the pad will be confined by its neighboring elements and shrinkage will be negligible. Finally, the applicant provided the profile dimensions of the footer (15.24 cm wide by 30.48 cm deep (12 in by 6 in)), stated the length to be equal to that of the MSE wall, and specified that the concrete mix would be designed in accordance with the governing ACI code. The staff reviewed these specifications, including the use of the governing ACI code for the concrete mix and concludes that the applicant provided an acceptable level of detail for the staff to determine that the design of the MSE wall footer is adequate because 1) the purpose of the concrete footer is to provide a clean smooth working surface for construction of the MSE wall and as such has no bearing capacity requirements, 2) the applicant stated that the design of the MSE wall considers that the horizontal soil reinforcements at or most near to the wall leveling pad (footer) have full effective pullout length so that the footer takes no or insignificant tension force when lateral pressure is exerted on the MSE wall system, 3) the 28 day compressive strength for the cast in place concrete footer will be a minimum of 17 MPa (2,500 psi) or greater and the dead load pressure exerted by the wall system will be at or less than 275 kPa (40 psi), 4) and the concrete will be designed in accordance with AVI-318, which is the governing code used for all nuclear plant construction, and finally 5) the concrete footer will be allowed to cure to meet its design strength prior to the placement of MSE wall sections. Based on the above, the staff considers the second issue of RAI 2.5.4-19S to be resolved.

With the resolution of these two issues, which relate to geotechnical engineering aspects of the VEGP LWA request, the staff considers the geotechnical engineering aspects of RAI 2.5.4-19S to be resolved. Based on the resolution of these aspects of RAI 2.5.4-19S, which is referenced by RAI 2.5.4-28S, the staff also considers RAI 2.5.4-28S resolved based on the resolution of issue 1 for RAI 2.5.4-19S. Finally, since the resolution of RAI 2.5.4-15S was contingent upon the resolution of RAI 2.5.4-28S, the staff also considers RAI 2.5.4-15S to be resolved as well because the applicant included in the ITAAC for shear wave velocity all four of the items requested by the staff in RAI 2.5.4-15S.

SSAR Subsection 2.5.4.5.5 discusses the quality control program and ITAAC associated with the excavation and backfill at the VEGP Units 3 and 4 site. The applicant stated that a MSE will be used as a form against which the nuclear island structures would be poured; however, it was not obvious to the staff that the backfill immediately behind the MSE wall would be compacted to the same density criteria of the remainder of the fill. Accordingly, in RAI 2.5.4-13S, the staff asked the applicant to provide the procedures for compaction of the backfill immediately adjacent to the MSE wall.

The applicant responded to RAI 2.5.4-13S by stating that with the exception of within five feet of the panels, the backfill will be compacted using a large smooth drum vibratory roller. For the five feet immediately behind the panels of the MSE, the applicant planned to use small single or double-drum vibratory walk-behind rollers, walk behind vibratory plate compactors, and jumping

jack compactors to achieve the requisite compaction. The applicant concluded that using these methods, the compacted fill would meet or exceed the established specifications. The staff reviewed this response, including the numerous tools which might be used to compact the fill adjacent to the MSE wall, and concludes that the applicant provided sufficient information in its response to resolve RAI 2.5.4-13S. The staff further based its conclusion on results from the Phase 2 of the test pad program, portions of which were observed by the staff and audited by Region II staff during the December 2007 and July 2008 visits to the VEGP site as documented in the staff-written trip reports (ML080110651 and ML082280539). During these trips, the staff observed the actual placement methodologies and subsequent field and laboratory test results for structural backfill materials placed adjacent to test portions of constructed MSE wall system. Therefore, the staff concludes that the applicant provided sufficient evidence to resolve RAI 2.5.4-13S.

The applicant described the extent of the excavations, planned backfills, and described its construction slopes, including providing adequate plans and profiles and boring logs supported by laboratory testing following the guidelines of RG 1.138. The applicant also described why and how the Upper Sand Stratum will be removed and replaced with engineered structural backfill, the specifications and locations of which the applicant adequately described in detail as discussed in this section. The applicant also established the design of its Seismic Category 1 and 2 structural backfill materials through analysis and testing, and provided sufficient test results in the form of laboratory test result summaries that adequately characterized the properties of the materials and provided sufficient information to allow the staff to determine material acceptability. The applicant conducted exploration and testing of potential borrow sources to identify backfill material sources, from which it was able to identify and verify that sufficient backfill material was available at the site. As discussed above, the applicant also proposed acceptable ITAAC for the structural backfill compaction density and shear wave velocity requirements and to provide documented evidence that testing is sufficient to verify that the AP1000 DCD requirements have been met. An associated ITAAC, concerning the applicant's approach to securing the waterproof membrane to the mudmat and placing the membrane against the vertical MSE wall, is evaluated in Section 3.8.5 of this SER. Finally, the applicant provided details for the MSE walls that will permit backfilling of the excavations up to plant grade.

Based on the information and findings above, including the resolution of RAIs and Open Items, the staff concludes that the discussion of the excavation and backfill plans at the site of VEGP Units 3 and 4, including the ESP, COL, and LWA investigations, is acceptable, and that the proposed Backfill ITAAC are appropriate. The staff concludes that the geotechnical parameters of minimum soil backfill density of 95 percent as determined by ASTM D 1557, and minimum shear wave velocity of 1000 fps at the bottom of the NI foundation are acceptable criteria because 1) a minimum compaction of 95 percent is the accepted industry standard for nuclear construction, and 2) the minimum shear wave velocity of 1000 fps is as required by the AP1000 DCD. The staff agrees with the applicant's density testing frequency because it will use the ASME NQA-1 industry standard and because the ITAAC will require the applicant's shear wave velocity testing at the bottom of the nuclear island foundation as required by the AP1000 DCD.

2.5.4.3.6 Groundwater Conditions

In SSAR Section 2.5.4.6, the applicant provided some basic groundwater conditions based on the water well observations and a summary of the dewatering plan implemented for VEGP Units 1 and 2. The staff determined that this information is necessary to understand the ground water conditions and potential dewatering plan at the ESP site.

The staff's evaluation of the information provided in support of the ESP application is as follows:

The staff reviewed the groundwater conditions described by the applicant in SSAR Section 2.5.4.6.1. The staff's evaluation of this information can be found in Section 2.4.12 of this SER.

The staff's evaluation of the information provided in support of the LWA request is as follows:

In RAI 2.5.4-6, the staff asked the applicant to explain the dewatering procedures it will use for the construction of the new units. In its response to this RAI, the applicant stated that it would implement the same dewatering program as that developed for the VEGP Units 1 and 2 but with some deviations. The applicant considered the dewatering program deployed at Units 1 and 2 to be successful, and subsurface conditions at the ESP site and at Units 1 and 2 are similar.

After reviewing the applicant's response, the staff concluded that, since the applicant had not yet determined the reactors' location within the ESP site and did not have a site-specific dewatering program, the staff could not evaluate the groundwater conditions as they affect the loading and stability of foundation materials. The staff was also unable to assess the applicant's dewatering plans during construction as well as ground water control throughout the life of the plant. Because the plant specific dewatering program could not be planned until the reactor location is decided, the staff considered that this design-related information was not necessary to determine whether 10 CFR Part 100 is satisfied. Therefore, in the SER with Open Items, the staff identified the need for the submission of groundwater condition evaluations and a detailed dewatering plan during the COL stage as COL Action Item 2.5-7.

However, in the revised SSAR, the applicant described plans for temporary dewatering of the site during the excavation and construction of VEGP Units 3 and 4. These plans are summarized in Section 2.5.4.2.6 of this SER and include the sump-pumping of ditches to remove groundwater during construction at the site. The staff reviewed this information, especially the dewatering plans and groundwater characterization through observation wells, and concludes that due to this additional information, COL Action Item 2.5-7 is no longer necessary.

The staff considered the following information acceptable to meet the criteria of RG 1.132 and 10 CFR Part 100.23: 1) as the staff discusses in SER Section 2.4.12, groundwater conditions at the site were discussed in sufficient detail in SSAR Section 2.4.12, 2) the applicant installed fifteen observation wells at the site for the ESP subsurface investigation and also used an additional 22 existing wells for the groundwater monitoring program, 3) the applicant had a representative number of wells in both the unconfined water table aquifer in the Upper Sand Stratum and in the confined Tertiary aquifer in the Lower Sand Stratum, and concluded that the Blue Bluff Marl is an aquiclude that separates the unconfined WT aquifer and the confined Tertiary aquifer, 4) the applicant was able to determine the groundwater levels in the wells and determine the hydraulic conductivity (k) values, through "slug" testing, 5) the applicant determined that some temporary dewatering of excavations will be required during construction and that, due to the low permeability of the Upper Sand Stratum and Blue Bluff Marl, sumps and pumps would be sufficient for successful construction dewatering, and 6) the applicant determined that groundwater levels for VEGP Units 3 and 4 correspond to design levels for the existing Units 1 and 2. The staff also concludes that the applicant's use of a liner in the sumps and ditches is acceptable, even though the liner material was not specified, since the type of liner material is peripheral to the adequate performance of the liner except in special applications, such as hazmat, which are not involved in the proposed construction dewatering.

The staff considered the criteria of RG 1.132 and 10 CFR Part 100.23 in its review of SSAR Section 2.5.4.6 and, for the above reasons, concludes that the applicant's assessment of groundwater conditions at the site is acceptable.

2.5.4.3.7 Response of Soil and Rock to Dynamic Loading

The staff's review of the information provided in support of the ESP application is as follows::

The staff reviewed SSAR Section 2.5.4.7, focusing on how the applicant developed the base shear wave velocity profile and modeled soil modulus reduction and damping with respect to cyclic shear strain. The applicant derived shear modulus for the soil strata from the relationship relating the unit weight to shear wave velocity, as well as the dynamic shear modulus reduction and damping ratio curves derived from EPRI (EPRI TR-102293 1993). The applicant used the SHAKE2000 (Bechtel 2000) computer program to evaluate the site dynamic responses.

The applicant derived ESP soil shear modulus degradation and damping curves from the curves developed by EPRI (1993). In RAI 2.5.4-5, the staff asked the applicant to justify its application of the EPRI curves to fine-grained soils. In response, the applicant stated that EPRI (1993) developed degradation curves for soils from gravels to high plasticity clays, and thus it was appropriate to apply the curves to fine-grained soils. EPRI (1993) presented fine-grained soils in Figures 7.A-16 (shear modulus reduction curves) and 7.A-17 (damping ratio curves) in terms of soil plasticity and required the use of the plasticity index . The applicant referred the staff to its response to RAI 2.5.4-17 for more details on how it derived the degradation curves from the EPRI (1993) curves. The applicant further indicated that the soil degradation relationships for fine-grained soil (and coarse-grained soils) used in the SSAR would be verified by laboratory testing during the COL subsurface investigation. Figures 2.5.4-6 and -7 of this SER present the site-specific shear modulus and damping ratio curves, respectively.

After reviewing the applicant's response and references, the staff determined that although Section 7A.6 of the EPRI (1993) report recommends the modulus degradation and hysteretic damping strain-dependent curves for generic CEUS sites, these curves are intended for gravelly sands to low plasticity silty or sandy clays and should not be applied to either very gravelly or very clayey deposits. The curves presented in the report for silts and clays of high plasticity are significantly different from those for sandy soils. In its response to RAI 2.5.4-10, however, the applicant indicated that the BBM "is described as hard, slightly sandy, cemented calcareous clay, and with less than 50 [percent] fine material," which was different from the type of materials for which the curves were intended. Therefore, the staff concluded that the applicant did not adequately explain why it was appropriate to apply those relationships to the silt and clay soils at the ESP site. The report further stated that, while the generic curves are appropriate for preliminary site studies, one should use site-specific data for final evaluations. In conclusion, the staff agreed with the applicant that it needed to verify the soil modulus degradation and damping curves. However, the staff concluded that this verification should not wait until the COL stage. Without site-specific soil modulus degradation and damping curves, the determination of site-specific GMRS (SSE) is inadequate. In the SER with Open Items, the need to provide site-specific soil degradation and damping ratio curves for the site-specific soil amplification calculation discussed in SER Section 2.5.2 was identified as Open Item 2.5-19.

The applicant responded to Open Item 2.5-19 by stating that site-specific soil degradation and damping ratio curves were not developed as part of the ESP investigations at the VEGP Units 3 and 4 site. The applicant also referenced its responses to RAIs 2.5.4-5 and 2.5.4-17 with

respect to the applicability of the generic EPRI curves to the materials at the VEGP site, stating that in addition to the EPRI curves, soil degradation and damping ratio curves from the adjacent SRS were also included in the analysis. Finally, the applicant stated that the data determined from the EPRI and SRS curves would be confirmed after RCTS testing was completed during the COL investigation. The staff considered this justification, including with respect to the assertion that the use of both generic and adjacent curves was sufficient, as well as the applicant's plans to confirm these conclusions during the COL phase of site investigations. Because the applicant confirmed the EPRI and SRS curves through RCTS testing performed as part of the COL investigations and included that information in the revised SSAR, the staff concludes that the applicant provided sufficient information to satisfy Open Item 2.5-19. Therefore, the staff considers Open Item 2.5-19 closed, which also resolves RAI 2.5.4-17 since the response provides a suitable description of how the soil degradation and damping ratio curves were developed.

The SSAR stated that the applicant used values of shear modulus and damping ratio to extend the EPRI curves beyond the 1 to 3.3 percent strain level. In RAI 2.5.4-13, the staff asked the applicant to justify how it extended the values beyond the 1 percent strain level and to provide a complete description and supporting data. In its response, the applicant stated that, even though it extended the EPRI curves beyond the 1 percent strain level, the maximum strains calculated during the site amplification analyses remained below 1 percent. But the applicant then stated that SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1, and 2.5.4.7.2.2 would be revised, along with associated tables and figures, to show the degradation curves only at a 1 percent or less cyclic shear strain. In light of the applicant's commitment to revise the shear modulus and damping ratio curves back to a 1 percent strain level without extrapolation, the staff concluded that this RAI could not be resolved until the revised SSAR sections were submitted for review. This was identified as Open Item 2.5-20 in the SER with Open Items.

In response to Open Item 2.5-20, the applicant updated the appropriate SSAR sections. The staff reviewed the revised figures and tables, and, based on the revisions to the SSAR and included tables and figures, which reflect the revised degradation curves at 1 percent cyclic shear strain, the staff concludes that the applicant provided sufficient data in the revised tables and figures of SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1, and 2.5.4.7.2.2 to close Open Item 2.5-20. The closure of Open Item 2.5-20 also resolves RAI 2.5.4-13 since it provides the necessary updating of figures and tables referencing the excess percent strain that was previously modeled.

In RAI 2.5.4-17, the staff asked the applicant to provide a complete description, including sample calculations, to show how it derived the shear modulus reduction and damping curves and how it incorporated uncertainties in the site characteristics into the curves' development. The applicant explained in its response that it used the shear wave velocity to calculate the low strain dynamic shear modulus (Gmax) only. The EPRI (1993) curves simply showed the ratio G/Gmax versus cyclic shear strain, regardless of the initial value of Gmax. The shear modulus reduction and damping ratio curves for cohesionless materials were based on confining pressure at depth, or simply depth, but were based on the plasticity index for cohesive material like BBM. The applicant then described how the shear modulus reduction and damping ratio curves were derived from the EPRI (1993) curves for each layer included in the base shear wave velocity profile. The applicant also stated that, "shear modulus reduction and damping curves will be obtained using undisturbed samples collected during the COL subsurface investigation."

In addressing how uncertainties were incorporated, the applicant stated that EPRI shear modulus reduction curves were extended from the strain level of 1 percent to 3 percent and uncertainties were incorporated in the site parameters during the randomization process. SER Figures 2.5.4-6 and 2.5.4-7 show shear modulus reduction and damping ratio curves, respectively, for each layer in the profile. The applicant randomized the shear modulus reduction and damping ratios at one strain level using log-normal distributions with median values given by the corresponding base-case curves and logarithmic standard deviations taken from the statistical summaries obtained by Costantino (1997) for natural soils. For the engineered backfill, the applicant reduced these standard deviations by one-third to account for a more homogeneous soil mass. The applicant also used a hyperbolic parametric form to generate the shear modulus reduction and damping ratios at other strains from the randomized values obtained above. The applicant stated that this approach produced realistic curves with logarithmic standard deviations that approximate the Costantino (1997) values over a wide range of strains. The applicant assumed that the normal random variables associated with the log-normal shear modulus reduction and damping ratios had a correlation coefficient of -0.75.

After reviewing the responses from the applicant, the staff reached the following conclusions:

1. Although the EPRI (1993) curves were up to the 1 percent strain level, the applicant did not provide information on the strain levels associated with the 10^{-4}, 10^{-5}, and 10-6 uniform hazard response spectra (UHRS) at the bedrock in the site response analyses and did not indicate whether the laboratory data developed during the SRS testing program carried to those levels of strain.

2. The adequacy of the equivalent-linear approximations for site response deteriorates as strain levels exceed about 0.5 percent effective shear strain. The applicant did not justify the applicability of the equivalent-linear method used in the SHAKE2000 model analysis if the strain levels were to exceed 1 percent.

3. In its response to RAI 2.5.4-13, the applicant indicated that it would revise the 3.3 percent strain level extrapolation back to 1 percent for the EPRI (1993) modulus reduction and damping curves; however, its response to this RAI indicated otherwise.

4. The applicant needed to demonstrate that it can confidently obtain undisturbed samples for deeper depths (e.g., in the Blue Bluff Marl and lower sands of the Congaree and Lower Snapp formations) for use in site response and SSI studies.

5. The applicant also needed to test disturbed samples of the compacted fill material to estimate appropriate modulus reduction and damping properties for the SSI analysis.

6. Other RAI responses indicated that the applicant used both SRS and EPRI (1993) models in the site response analyses and weighted them equally. Considering that site-specific data are almost always desired over generic models, the applicant needed to evaluate the strain level difference in the surface UHRS at different exceedance levels that result from application of these different models and to justify whether the equal-weighting approach is appropriate.

Based on its review of SSAR Section 2.5.4.7, the related references, and the applicant's responses to RAIs described above, the staff concluded that the applicant did not have sufficient site-specific laboratory data to support the determination of the site response to dynamic loading. Although the applicant committed to provide the site-specific modulus

reduction and damping curves during the COL stage, the staff determined that this issue, raised with a different perspective in RAI 2.5.4-13, needed to be resolved in the ESP application to provide site-specific shear modulus reduction and damping curves for the site SSE determination. Therefore, as stated earlier, resolving this issue was designated as Open Item 2.5-19 in the SER with Open Items; and the evaluation and closure of that Open Item was discussed in more detail above.

The staff's review of the information provided in support of the LWA request is as follows:

In supplemental RAI 2.5.4-16S, the staff asked the applicant to provide further discussions on the comparison of the EPRI 1993 soil degradation models to the SRS models, identify which model is more appropriate for the VEGP site, and explain how significant the models are to both site response and soil structure interaction (SSI) analyses. In its response, the applicant referenced its response to RAI 2.5.4-17 described above. The applicant also stated that both the EPRI and SRS curves were used as inputs into the SHAKE analysis at the VEGP ESP site. Also in the response, the applicant provided additional figures demonstrating the relationship between the EPRI-derived curves and those derived from the SRS data, selecting the SRS curves based on their stratigraphic relationship to the ESP site. Finally, the applicant stated the results of RCTS testing were used to develop site-specific data as well as confirm the derived curves. The staff agrees with the applicant that the SRS curves are more appropriate for the VEGP Units 3 and 4 site since the SRS curves represent a stratigraphy similar to that of the VEGP site. Based on the supplied response, especially the figures provided to compare the EPRI-derived and SRS curves and the selection of the SRS curves based on the stratigraphic correlation to the VEGP site, the staff concludes that the applicant provided the information to resolve RAI 2.5.4-16S.

Based on its review of SSAR Section 2.5.4.7 and the resolution of RAIs and closure of Open Items described above, the staff concludes that the applicant adequately determined the response of the soil and rock underlying the site of VEGP Units 3 and 4 to dynamic loading and that this determination is acceptable for both the ESP application and the LWA request.

2.5.4.3.8 Liquefaction Potential

In its review of SSAR Section 2.5.4.8, the staff evaluated the applicant's description of liquefaction potential and plans for future liquefaction studies at the ESP site. The staff's review focused on the applicant's conclusion that, based on the previous investigations and excavation completed for the VEGP Units 1 and 2, liquefaction would occur only in the Upper Sand Stratum.

The staff's evaluation of the information provided in support of the ESP application is as follows:

In RAI 2.5.4-14, the staff asked the applicant to justify why liquefaction analyses were not performed on the BBM, since the unit has a relatively high variable fines content (24–77 percent) and saturation level (14–67 percent), and a potentially high ground motion level at the site. In response, the applicant first discussed the liquefaction potential for the BBM (Lisbon Formation) based on the material and age. The applicant then examined the field strength and shear wave velocity results to determine whether the marl would liquefy based on these results.

The applicant stated that, although the BBM frequently contained less than 50 percent of fine material, it had the appearance and characteristics of a calcareous claystone or siltstone and was a hard, slightly sandy, cemented calcareous clay. The design undrained shear strength of

the marl was set as 478 kPa (10,000 psf) with a preconsolidation pressure as high as 3,831 kPa (80,000 psf), indicative of a highly overconsolidated material. Although the marl would be below the groundwater table, its compressed structure would prevent it from having the free water characteristic of a saturated granular material. Based on these characteristics, the applicant concluded that the BBM is not a material with liquefaction potential, regardless of the ground motion level. The applicant further indicated that liquefaction resistance would increase markedly with geologic age. Based on Youd et al. (2001), Pleistocene (1.8 mya to 10,000 year) sediments were more resistant, while pre-Pleistocene (older than 1.8 mya) sediments were generally immune to liquefaction. The BBM's age is late middle Eocene (40 to 41 million years old), much older than Pleistocene.

The applicant also stated that, based on Youd et al. (2001), there were thresholds for the N-values, tip resistance, and shear wave velocity beyond which the material was considered nonliquefiable (e.g., a sand with 35 percent or more fines or a soil with a corrected N-value over about 21 is not liquefiable). According to the applicant, of the 58 N-values measured in the marl for the ESP investigation, 5 were below 50, ranging from 27 to 46. Thus, if the marl were a potentially liquefiable material, a liquefaction analysis would be run for these five samples. An initial analysis of these five samples showed factor-of-safety values in excess of the accepted 1.35 value in all cases. All of the CPTs that penetrated into the marl had refusal at or near the top of the stratum; therefore, the applicant concluded that the measured tip resistance showed the material to be nonliquefiable. The applicant also stated that the typical shear wave velocities in the marl ranged from 426 to 807 m/s (1,400 to 2,650 ft/s) but dropped to 301 to 512 m/s (990 to 1,680 ft/s) when corrected for overburden. According to the applicant, Youd et al. (2001) indicated that, for a sand with 35 percent or more fines, soils with a corrected shear wave velocity in excess of about 190.5 m/s (625 ft/s) were nonliquefiable.

The applicant stated that, based on material and age, the BBM does not have the potential to liquefy, and that the CPTs, as well as shear wave velocities, consistently indicated the marl is nonliquefiable material. In addition, the applicant indicated that over 90 percent of the SPT N-values indicated the marl as nonliquefiable material and the remaining N-values showed adequate factors of safety.

After review of the applicant's response, however, the staff was concerned that (1) the general observation of liquefaction occurrence with respect to age and material type did not exclude the liquefaction potential of the BBM because of the limitation of the observations, such as the possible gravel engagement during the SPT and CPT tests; and (2) limited test data, including N-values, tip resistance, and shear wave velocity, could not reliably exclude the liquefaction potential for the BBM. The staff concluded that limited data prevented the applicant from making a conclusion on the liquefaction potential for the BBM; therefore, the staff determined that the applicant did not have sufficient ESP soil property data to confirm that the BBM is not liquefiable. Accordingly, the staff in the SER with Open Items designated this issue as Open Item 2.5-21.

In response to Open Item 2.5-21, the applicant stated that additional boring logs were used to re-characterize the confusion surrounding the presence of hard layers (i.e. gravel) in the BBM that may have yielded anomalously high SPT results. The applicant provided updated boring logs, along with additional laboratory tests, which it stated showed that the BBM was a hard clay or soft rock material and therefore not prone to liquefaction. The applicant incorporated additional boring logs and field and laboratory test data into later revisions of the SSAR. The staff reviewed these additional boring logs and information and concludes that the soil property data support the applicant's conclusion that the BBM was not susceptible to liquefaction. The

staff based its conclusion on the results of the liquefaction potential analyses performed for the application, including liquefaction potential based on SPT data, liquefaction potential based on shear wave velocity data, and liquefaction analyses of the compacted backfill. The applicant also determined that the Blue Bluff Marl is primarily cohesive but has some lenses of coarse grained materials, but these materials have an adequate factor of safety, greater than 1.1, against liquefaction. RG 1.198 states that factors of safety against liquefaction of 1.1 to 1.4 are considered to be moderate. Accordingly, the staff considers that the applicant has demonstrated an adequate factor of safety against liquefaction for the Blue Bluff Marl for Open Item 2.5-21 to be closed. The closure of Open Item 2.5-21 also resolves RAI 2.5.4-14, since the applicant provided the additional information required to confirm the liquefaction potential of the BBM.

The staff identified the site characteristic value for liquefaction potential and determined it should be defined as negligible. Because portions of the soil at the VEGP site are susceptible to liquefaction, the applicant stated that these soils would be either removed and replaced, or physically improved, such that the liquefaction potential is reduced to negligible and the factor of safety against liquefaction is increased to at least 1.1. The staff therefore proposes to include the following condition in any ESP that might be issued in connection with this application: The ESP holder shall either remove and replace, or shall improve, the soils above 26.8 m (88 ft) below the ground surface for soil under or adjacent to Seismic Category 1 structures, to eliminate any liquefaction potential. This is **Permit Condition 1**.

The staff's evaluation of the information provided in support of the LWA request is as follows:

The staff reviewed the information provided by the applicant regarding the liquefaction potential of the backfill materials proposed for use at the site. Based on the properties of the backfill material described in SSAR Section 2.5.4.5.3, and the results of field and laboratory testing, the applicant concluded that, for the design basis earthquake, liquefaction was not a concern within the compacted backfill. Considering the dry density of 95 percent, and the relatively high blow count and shear wave velocity of the compacted backfill, the staff concurs with the applicant's conclusion that liquefaction potential of the compacted backfill was not a concern at the VEGP Units 3 and 4 site. Therefore, the staff concludes that the assessment of the liquefaction potential of the compacted backfill at the site is adequate to satisfy the criteria of 10 CFR Parts 50 and 100 with respect to the liquefaction potential of the materials underlying the Seismic Category 1 structures at the site.

Based on its review of SSAR Section 2.5.4.8 and the resolution of RAIs and closure of Open Items, the staff concludes that the applicant's assessment of the liquefaction potential of the soil and rock underlying the site of Units 3 and 4 is acceptable for both the ESP and LWA applications, subject to Permit Condition 1.

2.5.4.3.9 Earthquake Design Basis

SSAR Sections 2.5.2.6 and 2.5.2.7 present the applicant's derivation of the safe shutdown earthquake (SSE), and Section 2.5.2.8 presents the operating basis earthquake (OBE). Sections 2.5.2.3.6 and 2.5.2.3.8 of this SER provide the staff's evaluation of the applicant's determination of the SSE and OBE. Shear wave velocity profiles, soil modulus reduction, and damping curves described in Section 2.5.4 are critical inputs to the site seismic response and therefore to the SSE and OBE. However, the staff's analysis of these inputs is fully discussed in SER Section 2.5.2.

2.5.4.3.10 Static Stability

In its review of SSAR Section 2.5.4.10, the staff focused on the applicant's evaluation of bearing capacity and settlement of the bearing strata at the ESP site. The applicant used the following assumptions in calculating soil-bearing capacity and structure settlement: (1) placing all safety-related structures on the structural backfill above the Blue Bluff Marl after removal of the Upper Sand Stratum; (2) placing the base of the containment and auxiliary building foundations about 12.19 meters (40 ft) below final grade, or 15.3 to 18.3 meters (50 to 60 ft) above the top of the Blue Bluff Marl Stratum; and (3) placing other foundations in the power block area at depths of about 1.2 meters (4 ft) below final grade. The applicant modeled the containment building mat as a circle with a diameter of about 43.3 meters (142 ft) placed at a depth of 12.0 meters (39.5 ft) below finish grade in the calculations. The applicant determined that the allowable bearing pressure was 1470.3 kPa (30,700 psf) under static loading conditions and 2203 kPa (46,000 psf) under dynamic loading conditions. The settlement under an average bearing pressure of 239.5 kPa (50,000 psf) was 41 mm (1.6 in.).

In RAI 2.5.4-15, the staff asked the following of the applicant:

1. Justify the adoption of the Peck et al. (1974) settlement and differential settlement values as guidelines which suggest total settlement of no more than 50 mm (2 in.), and differential settlement of no more than 19 mm (0.75 in.). For footings that support smaller plant components, the total settlement should be no more than 25 mm (1 in.), and the differential settlement no more than 13 mm (.5 in,).

2. Explain the main causes for exceeding these settlement values at the foundation levels of Units 1 and 2 and whether it would take any measures to prevent settlements and differential settlements for the new units.

3. Justify the use of an average bearing pressure of 239.5 kPa (50000 psf) for the settlement analyses of compacted fills.

In response to this RAI, the applicant stated the following:

1. The geotechnical community has widely accepted and used the Peck et al. (1974) total settlement guidelines of 25 mm (1 in.) for column footings and 50 mm (2 in.) for mats. When limiting foundation settlements to these values, differential settlements are usually very small. The applicant further stated that, even if these settlement values were exceeded, it would not necessarily have adverse effects on structures, especially for large mat foundations which can efficiently distribute structural loads to the soil. The applicant used the VEGP Units 1 and 2 as an example where the measured settlements of the containment buildings ranged from 102 to 109 mm (4 to 4.3 in.)

2. It (the applicant) will not use the settlement guidelines from Peck et al. (1974) for Units 3 and 4. The approach used for Units 3 and 4 consisted of estimating settlements for power block structures and using them as design values. The "VEGP Report on Settlement" prepared by Bechtel in 1986 provides comparisons of measured versus calculated settlements and concludes that the measured values did not exceed calculated or design values. The applicant would reanalyze and employ corrective measures in the event that monitored settlements exceed the design values. The

applicant committed to follow the same approach for Units 3 and 4 and to revise SSAR Sections 2.5.4.10.2 and 2.5.4.11 accordingly in the next revision to the ESP application.

3. It (the applicant) used a bearing pressure value of 239.5 kPa (50,000 psf) in foundation settlement analysis for illustrative purposes because no design value was available during the ESP. The applicant will revise the calculation using design values during the COL application.

After reviewing the responses, the staff concluded the following:

1. A primary concern of potential total and differential settlements is how these settlements compare with what the design of the reactor takes into consideration. It is important to compare the estimated settlements, which are appropriate for evaluation of the acceptability of the site at the ESP stage, with those incorporated into the plant design to evaluate the degree of conservatism because there will be severe impact to the safety of the SSCs once unexpected differential settlements occur.

2. The contact pressures associated with the planned reactor model are of interest and need to be considered at the ESP stage to estimate potential settlement. Since the data for a given reactor facility are available, the applicant incorporated the data into the site evaluation. Based on the above considerations and in lieu of the fact that large settlements were observed at VEGP Units 1 and 2, the staff concludes that the applicant did not demonstrate quantitatively whether the observed large settlement that occurred at the existing VEGP units will occur at the VEGP site and have no impact on the new units. This was identified as COL Action Item 2.5-8 in the SER with Open Items.

In the revised SSAR, the applicant provided additional information on the settlement analysis for the ESP site. These analyses are summarized in Section 2.5.4.2.10 of this SER, and include details on the differential settlement and the application of the elastic properties of VEGP Units 1 and 2 to determine the settlement of Units 3 and 4. The staff reviewed the additional information supplied in Revision 4 and determines that because the applicant provided the information on settlement analysis using differential settlement and the elastic properties of the existing units, the response negates the need to include COL Action Item 2.5-8 in the final safety evaluation report.

In RAI 2.5.4-16, the staff asked the applicant to justify not analyzing the stability of all planned safety-related facilities in terms of bearing capacity, rebound, settlement, and differential settlements with the consideration of dead loads of fills and the reactor facility, as well as the lateral loadings. In its response, the applicant explained that this kind of information is not available at the ESP stage. Based on the applicant's response, the staff concluded that, since the applicant committed to provide more details regarding the bearing capacity, the staff agreed with the applicant that this information will not be available until the COL stage, and considered that this design-related information was not necessary to determine whether 10 CFR Part 100 is satisfied. Accordingly, this issue was designated as COL Action Item 2.5-9 in the SER with Open Items.

Revision 4 of the SSAR incorporates additional site investigation results from the COL stage, including bearing capacity calculations summarized in Section 2.5.4.2.10 of this SER. The staff reviewed this additional information from the COL site investigations, including the influence of the load-bearing layer (Blue Bluff Marl) on the allowable bearing pressure. The staff determined that because the applicant provided additional factors of safety and allowable bearing capacity

details that the applicant determined as part of its COL investigation, the applicant provided adequate information to address concerns identified in COL Action Item 2.5-9. Therefore, the staff concludes that COL Action Item 2.5-9 does not need to be included in the FSER.

In RAI 2.5.4-18, the staff asked the applicant to provide detailed information on its determination of the allowable bearing capacity value. In its response, the applicant provided a detailed description of bearing capacity evaluations based on the Vesic (1975) formula. In addition, the applicant later clarified that the calculated value was net allowable bearing capacity, not the gross bearing capacity; therefore, the formula used in the actual calculation was slightly different from that presented in the reference. From its review of the applicant's response, the staff considered that the Vesic (1975) formula is based on primary assumptions of gross shear failure of soils under the foundation. Although this allowable bearing capacity formulation is applicable for general foundation analysis, the staff considers it inappropriate to use in nuclear power plant foundation design. The control factors of allowable contact pressure for a large and heavy structure typically are not general shear failure but are (1) settlements; (2) allowable pressures used in design of the wall/basemat intersection; and (3) toe pressures developed during potential overturning and sliding of the facility. Based on the above considerations, the staff concluded that the allowable bearing capacity value provided by the applicant is not appropriate when considering the expected governing issues controlling the site evaluation. This was identified as Open Item 2.5-22 in the SER with Open Items.

In response to Open Item 2.5-22, the applicant stated that the bearing and settlement analysis would be completed in late 2007 and would be incorporated in a later revision of the SSAR. When the applicant submitted Revision 4 of the SSAR, the staff reviewed the bearing capacity of the containment and auxiliary buildings, which the applicant stated was 2010 kPa (42 ksf) under dynamic loading conditions with a factor of safety of 2.25 and 1627 kPa (2.25 and 34 ksf) under static loading conditions with a factor of safety of 3.0. These bearing capacity values were identified by the staff as the site characteristic values. The staff also considered the settlement analysis performed by the applicant for the large mat foundations that will support the major power plant structures. The applicant concluded that the settlement at the site would be 5.08 to 7.6 cm (2 to 3 in), with a tilt of approximately 0.63 cm (¼ in) in 15 m (50 ft), a differential settlement between structures of less than 2.54 cm (1 in), and the predicted heave due to foundation excavation ranging from about 2.54 to 6.35 cm (1 to 2 ½ in).

As a result of a staff audit of seismic calculations, the applicant revised SSAR Section 2.5.4.10.1. The applicant evaluated the allowable bearing capacity of the structural backfill under the nuclear island for dynamic loading conditions using both Terzaghi's bearing capacity equation for local shear and Soubra's method with seismic bearing capacity factors, which incorporates Terzaghi's bearing capacity equation for general shear with an internal friction angle of 36° (SNC 2008d). To simulate the potential for higher edge pressures during dynamic loading, the applicant considered three foundation widths corresponding to 10, 25, and 50 percent of the width of the nuclear island basemat. The applicant stated that the results from these two methods compared well with Terzaghi's approach for local shear, providing more conservative values, and it reported the computed average ultimate capacities for the three widths as 4261, 4788, and 5697 kPa (89, 100, and 119 ksf). The applicant reported that using a width of 7.62 m (25 ft) and a factor of safety of 2.25 for site-specific conditions provided an allowable bearing pressure greater than 2010 kPa (42 ksf) under dynamic loading conditions for the nuclear island. The applicant also noted that the value was greater than the DCD requirement of 1675 kPa (35 ksf) for dynamic bearing as well as the Vogtle site-specific maximum dynamic demand of 861 kPa (18 ksf) for the ESP soil profile.

The applicant also evaluated the bearing capacity of the structural backfill in terms of the ratio of the ultimate bearing capacity against structure demand, and stated that this capacity over demand (C/D) ratio provided an alternative measure of the margin of safety against bearing failure (SNC 2008d). The applicant evaluated these C/D ratios for the static and dynamic demand conditions as well as the maximum dynamic demand from the Vogtle site-specific seismic evaluation. The applicant stated that the C/D ratios, 11.9 for DCD static, 2.9 for DCD dynamic, and 5.6 for the site-specific dynamic, were higher than those typically utilized for standard practice. While the results did not account for settlement of the structures, the applicant concluded the significant margin suggested that settlements would be minimal and within the DCD requirements.

Considering: 1) the updated bearing capacities determined for both static and dynamic conditions, which incorporated capacity-over-demand ratios as an alternative measure to the factor of safety against bearing failure; 2) the settlement analysis results, which showed minimal settlement; and 3) the displacement monitoring plans for the VEGP site, the staff concludes that the information provided by the applicant in the revised SSAR addressed the concerns identified in Open Item 2.5-22 and the staff considers the Open Item closed. The closure of Open Item 2.5-22 also resolves RAIs 2.5.4-15, 2.5.4-16 and 2.5.4-18. Based on its review of SSAR Section 2.5.4.10 and the applicant's responses to the RAIs, as described above, the staff further concludes that the applicant provided an adequate assessment of the static stability of the ESP site through the incorporation of data and results for both ESP and COL site investigations. The site characteristics approved by the staff for minimum bearing capacity (static and dynamic) are included in Appendix A. Furthermore, the staff concludes that the applicant provided sufficient information with respect to the static and dynamic stability of the site to satisfy the applicable criteria of 10 CFR Parts 50 and 100.

2.5.4.3.11 Design Criteria

In SSAR Section 2.5.4.11, the applicant provided general geotechnical criteria, such as acceptable factors of safety against liquefaction, allowable bearing capacities, acceptable total and differential settlements, and an acceptable factor of safety against slope stability failure.

The staff's evaluation of the information provided in support of the ESP application is as follows:

The staff reviewed the information provided by the applicant regarding the applicable AP1000 geotechnical design criteria to determine if the applicant conducted an exploration and testing program sufficient to determine whether the site would support the design parameters. The staff focused on 1) the applicant's efforts to determine the ability of the Blue Bluff Marl bearing layer to support the plant structures and whether the overall site geology met site perimeters, 2) the applicant's studies to determine static and dynamic bearing capacity and whether the site properties and properties of the engineered backfill met or exceeded site perimeters and required factors of safety, 3) whether the applicant's studies and backfill designs supported DCD shear wave velocity minimum requirements, and 4) whether the applicant sufficiently analyzed site liquefaction potential. As discussed in the previous sections, the staff concludes that the applicant conducted an exploration and testing program consistent with the guidance presented in RG 1.132, RG 1.138, and RG 1.198 to adequately characterize the site and verify that the site would support the AP1000 design criteria discussed and applied in Section 2.5.4 of this SER.

The staff focused its review on the design criteria, including the factors of safety against specific events, such as liquefaction and loading conditions. The application did not provide structural

design criteria, such as wall rotation, sliding, or overturning. The staff also considered the applicant's incorporation of standard design criteria into the most recent revision (Rev. 4) of the SSAR. Based on the applicant's inclusion of site-specific design criteria, including the factors of safety against events such as liquefaction or loading, the staff considers the applicant's design criteria used in the ESP application to be acceptable, as the applicant has met the applicable standards of 10 CFR Part 50.

The staff's evaluation of the information provided in support of the LWA request is as follows:

The staff reviewed the information provided by the applicant regarding the design criteria required to support the LWA request to excavate, prepare the site, and backfill the proposed plant site to the bottom of the foundation within the nuclear islands and up to plant grade outside the MSE walls. To meet the requirement for the LWA, the applicant needed to characterize the site down to a depth sufficient to support the AP1000 site parameters for bearing capacity, shear wave velocity and liquefaction, and it also needed to develop the site-specific criteria for engineered structural backfill, MSE retaining walls, concrete mudmats, and MSE and concrete mudmat waterproofing materials sufficient to meet the intent of the DCD design for coefficient of friction. As discussed in the preceding sections, the staff concludes that the applicant presented sufficient information for a LWA request because the staff determined that the applicant 1) adequately characterized the site following the guidelines presented in RG 1.132, 2) performed field and laboratory testing following the guidelines presented in RG 1.132 and RG 1.138 to verify that the site and engineered structural backfill support the DCD minimum required shear wave velocity, 3) presented sufficient design details for the concrete mudmat and MSE wall, including constructing a test section for staff observation, and 4) worked with the DCD design organization to determine the proper waterproofing system and minimum required coefficient of friction for the system.

In RAI 2.5.4-19, the staff asked the applicant to justify the omission of additional design criteria and factors of safety (FS). In response, the applicant revised the SSAR to reference the applicable design criteria in the AP1000 DCD, Revision 15. The applicant also stated that the FS against liquefaction should be greater than 1.1; FS of 3 should be applied to bearing capacity equations, but this FS can be reduced to 2.25 when dynamic or transient load conditions apply; and the long-term static and seismic FS against slope stability failure was 1.5 and 1.1, respectively. Because the applicant incorporated the applicable design criteria from Revision 15 of the AP1000 DCD and the revised SSAR to include relevant factors of safety, the staff considers RAI 2.5.4-19 resolved. Furthermore, based on the closure of RAI 2.5.4-19, the staff concludes that the design criteria presented for an ESP at the VEGP Units 3 and 4 site is acceptable to satisfy the requirements of 10 CFR Part 50 because the revised SSAR contained a description and safety assessment of the site and the site evaluation factors identified in Part 100, including the information relative to the materials of construction, general arrangement and approximate dimensions of the facility sufficient to provide reasonable assurance that the final design will satisfy the design bases with adequate margin of safety.

Based on the applicant's inclusion of the design-specific criteria, including the factors of safety against events such as liquefaction or loading, the staff considers the applicant's design criteria to be acceptable for the LWA request, as the applicant has met the applicable standards of 10 CFR Part 50.

2.5.4.3.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 states that no ground improvement techniques were considered beyond the removal and replacement of the Upper Sand Stratum with engineered structural backfill; however, other ground improvement techniques will be considered as necessary. The staff therefore focused its review on the subsurface improvement plans, the most significant of which is the planned removal of the entirety of the Upper Sand Stratum. The staff reviewed the plans for removal of the Upper Sand Stratum, as described in Section 2.5.4.1.5, and for the reasons evaluated in Section 2.5.4.3.5 of this SER, as well as the applicant's consideration of other improvement techniques, as necessary, the staff concludes that the plans for subsurface improvement therefore satisfy the criteria of 10 CFR Part 100. The inclusion of the detailed plans for removal of the Upper Sand Stratum, as well as the applicant's consideration of additional ground improvement techniques make fulfills COL Action Item 2.5-11. Therefore, COL Action Item 2.5-11 is no longer necessary.

2.5.4.4 Conclusions

Based on its review of SSAR Section 2.5.4, related references, and the applicant's responses to the associated RAIs and Open Items described above, the staff concludes:

The applicant conducted a limited ESP investigation to determine the engineering properties of subsurface soils at the ESP site. The applicant supplemented the few field and laboratory tests conducted as part of the ESP investigation to determine static and dynamic and other engineering properties of the underlying soils with information from the subsequent COL investigation. The additional quantity and quality of the test results were sufficient for the applicant to reliably determine the engineering properties of the subsurface materials. Therefore, the staff concludes that the applicant has adequately determined the engineering properties of the subsurface materials.

The applicant provided a site-specific shear wave velocity profile in a situation that assumed the shear wave velocity measured from the down-hole tests was lower than the shear wave velocity obtained from the suspension P-S velocity measurements; the shear wave velocities from previous investigations associated with VEGP Units 1 and 2 were also lower. Additionally, the applicant provided the results of soil dynamic testing on the samples from the ESP site to provide soil modulus reduction and damping curves to feed into the site response study and the site-specific shear wave velocity profile. The applicant also supplemented the SSAR with additional inputs to the development of the shear wave velocity profile and the shear modulus reduction curves. Therefore, the staff concludes that the applicant provided sufficient information to characterize the shear wave velocity profiles, and the shear modulus reduction and damping ratio curves, which are critical input to the site-specific ground motion response spectrum discussed in SER Section 2.5.2, as well as to the soil structure interactions discussed in SER Section 3.8.

The applicant provided an assessment of the liquefaction potential of the BBM, which was the load-bearing unit at the ESP site. Based on the results of extensive SPT and CPTs by the applicant, the staff concurs with the applicant that the BBM is not prone to liquefaction. The applicant also described the excavation and backfill plans, in extensive detail, to support both the ESP application and its LWA request. These plans included the use of a test pad program to better constrain the final engineering properties of the Seismic Category I backfill to be used. The staff concludes that the level of detail provided for the excavation and backfill plans, including quality control and ITAAC, is sufficient to address the requirements of 10 CFR Part 50.

The proposed Units 3 and 4 would be located above the load-bearing strata similar to that underlying the existing units, and the existing units already observed an unusually large settlement (both total and differential). The applicant provided a detailed settlement analysis to ensure that the SSCs for the AP1000 are safe. The staff finds that the applicant adequately demonstrated the stability of the subsurface materials in response to static and dynamic loading conditions at the ESP site. The applicant provided the bearing capacity for the containment and auxiliary buildings at the site, which were given as 2,010 kPa (42 ksf) under dynamic loading conditions with a factor of safety of 107 and 1,627 kPa (2.25 and 34 ksf) under static loading conditions with a factor of safety of 3.0. Based on these bearing capacities and the high factor of safety, the staff concludes that the bearing capacity of the site is acceptable to meet the requirements of 10 CFR Parts 50 and 100 with respect to the static stability of the site. The staff also reviewed the information and data from the applicant's field and laboratory investigations as well as the evaluations of the geotechnical engineering properties of the soils and rock underlying the ESP site. Additionally, the staff made several trips to the site to observe applicant activities and the geotechnical conditions of the site to determine whether the applicant followed the guidance contained in RG 1.132 and other relevant guidance in its ESP and LWA site-specific investigations.

Based on the above findings, the staff concludes that, in support of both the ESP application and LWA request, the applicant conducted sufficient site investigations and performed adequate field and laboratory tests and associated analyses, to provide sufficient information describing soil conditions underlying the ESP site, such as the possible existence of "soft zones" in the foundation-bearing layer. The applicant also demonstrated reliable engineering properties of the soils through the combination of its ESP and COL site investigations. This information was addressed and evaluated by the staff as part of its review of the LWA request. Therefore, the staff concludes that for the information required by the scope of the ESP, the applicant has provided sufficient information to characterize the subsurface materials at the ESP site of VEGP Units 3 and 4. Based on its review of the engineering properties of materials at the ESP site, the assessment of bearing capacity, liquefaction potential, and settlement, as well as the development of a shear wave velocity profile through the site, the staff finds that the applicant has met the requirements of 10 CFR 100.23 in that the applicant adequately demonstrated the overall static and dynamic stability of the site, identified the soil and rock engineering properties through field and laboratory testing, and characterized the soil subsurface profile.

In SSAR Section 2.5.4, the applicant identified the subsurface material properties as ESP site characteristic values. The first site characteristic specifies that there is no liquefaction below approximately 26.8 m (88 ft) below the ground surface. The applicant demonstrated, in SSAR Section 2.5.4.8, that any liquefaction at the ESP site would be limited to the upper 26.8 m (88 ft) of soil. The requirement to remove and replace or otherwise improve the liquefiable soils at the site to eliminate the liquefaction potential is Permit Condition 1. The second site characteristic value specifies a minimum bearing capacity of 1627 kPa (34 ksf) under static loading conditions and 2010 kPa (42 ksf) under dynamic loading conditions. These values are based on the VEGP site soil properties and the results of the applicant's ESP and COL investigations. Finally, the third design parameter specifies minimum S-wave velocities for the depth intervals given in SSAR Tables 2.5.4-11 and 2.5.4-11a. These S-wave velocity values are based on the applicant field geophysical surveys. The staff has reviewed the applicant's suggested site characteristics related to SSAR Section 2.5.4 for the inclusion in an ESP, should one be issued. For the reasons set forth above, the staff agrees with the applicant's proposed site characteristic and the values for those characteristics.

Based on the staff's review of the applicant's information regarding the LWA request, the staff concludes that the applicant conducted sufficient subsurface investigations and performed adequate field and laboratory testing and analyses to support that request. As discussed previously in this section of the SER, much of the information needed for the LWA request was also required for the staff's evaluation of the ESP application. The applicant had to first adequately characterize the proposed site to determine whether the site could support the applicable AP1000 design criteria for the LWA activities. As the staff has stated above, the applicant adequately characterized the site and verified that the site criteria for bearing capacity, liquefaction, and shear wave velocity could be met. The applicant also developed the criteria for the engineered structural backfill materials and verified that these criteria, in conjunction with the geologic site conditions, would further support the DCD design criteria for bearing capacity, liquefaction, and shear wave velocity. As the staff stated above, the applicant did so, following the guidance presented in the applicable Regulatory Guides.

Once the applicant determined that the site and proposed backfill materials would meet the AP1000 design criteria, the applicant determined whether sufficient material was available on-site to backfill the proposed excavation. The applicant also proposed a design for the MSE wall system. As part of the LWA request, the applicant showed the extent and depth of the excavation; disposition of the excavated materials as backfill or spoil; extent of temporary construction slopes and construction dewatering details; preparation of the marl bearing layer for placement of backfill and backfilling to the bottom of the foundation; placement of the MSE walls and nuclear island concrete mudmat working surfaces and waterproofing system; backfilling around the perimeter of the nuclear islands outside of the MSE walls to final plant grade; demonstration of mass and confined backfill placement techniques; and, finally, its demonstration of backfill density, shear wave velocity and, as evaluated in SER Section 3.8.5, waterproofing system friction coefficient, with proposed ITAAC to verify and document that the AP1000 design criteria will be met. Therefore, for the reasons stated above, the staff concludes that the applicant has adequately demonstrated that it has met the applicable LWA requirements associated with the stability of subsurface materials and foundations for the requested LWA activities at the VEGP site.

2.5.5 Stability of Slopes

SSAR Section 2.5.5 describes the applicant's review of existing slopes at the ESP site and the applicant's plan for permanent cut and fill slopes during construction excavation. The applicant also discussed its plans for future slope stability analysis to take place during the design phase. The applicant did not perform slope stability analysis for the ESP site because there is no existing slope and the applicant cannot determine the future slope at the ESP phase.

2.5.5.1 Technical Information in the Application

The applicant stated that, since there were no existing slopes or embankments near the proposed location of VEGP Units 3 and 4, it did not perform a dynamic slope stability analysis. The applicant further stated that the site grading for construction of new units would result in nonsafety-related permanent cut and fill slopes. Permanent cut slopes would have a height of 15.2 meters (50 ft) or less and would be located several hundred meters away from planned or existing safety-related structures. Permanent fill slopes would have a height of 6.1 meters (20 ft) or less and would also be several hundred meters away from planned or existing safety-related structures. During the construction phase, the applicant will remove the soils above the Blue Bluff Marl and replace them with compacted structural fill. The applicant stated

that the construction excavation cut slopes would be temporary (i.e., only during the construction period) and that they will be far away from the safety-related structures of the existing VEGP Units 1 and 2. The applicant committed to perform nonsafety-related permanent slope stability analysis for dynamic and static conditions, as well as excavation cut slope analysis for static conditions during the design stage, to ensure that these slopes will not pose a hazard to the public.

2.5.5.2 Regulatory Basis

SSAR Section 2.5.5 states that the applicant did not perform a slope stability analysis for the ESP site application. However, the applicant stated in SSAR Section 1.8 that it followed the guidance of NUREG-0800, Section 2.5.5, when it described the slope-related issues in SSAR Section 2.5.5. In its review of SSAR Section 2.5.5, the staff considered the regulatory requirements in 10 CFR 100.23(c) and 10 CFR 100.23(d). According to 10 CFR 100.23(c), applicants must investigate the engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site. Pursuant to 10 CFR 100.23(d)(4), applicants must evaluate siting factors such as natural and artificial slope stability.

2.5.5.3 Technical Evaluation

The staff focused its review of SSAR Section 2.5.5 on whether there are any existing or planned new slopes that would adversely affect the safety-related structures of the proposed new units due to any possible loading conditions and/or natural events. After reviewing the information provided by the applicant, the staff concludes that, because there are no existing significant slopes near the proposed ESP site, a detailed slope stability analysis is not necessary at the ESP stage. The staff considers the creation of permanent slopes during construction to be a design-related issue, which must be addressed at the COL stage. However, after reviewing the site construction plan layout and discussions with the applicant, the staff confirmed that the only permanent slopes are not safety-related. Therefore COL action item 2.5-12 is no longer needed.

2.5.5.4 Conclusions

Since there are no safety-related permanent slopes, the applicant did not perform any slope stability analysis. The excavation will create nonsafety-related permanent cut and fill slopes during the new units' construction stage, however, since these slopes are not permanent, they are not part of the staff's review.

2.5.6 Embankments and Dams

SSAR Section 2.5.6 presents a general description of existing and potential new embankments and dams at the ESP site.

2.5.6.1 Technical Information in the Application

SSAR Section 2.5.6 indicates that there are no earth, rock or earth, and rock fill embankments required for plant flood protection or for impounding the cooling water required for the operation of the plant. The applicant indicated that there are three existing nonsafety-related impoundments at the site—Mallard Pond, Debris Basin Dam 1, and Debris Basin Dam 2. The Mallard Pond is located to the north of the proposed switchyard, Debris Basin Dam 1 is located

to the southeast of the proposed cooling towers, and Debris Basin Dam 2 is located to the southwest of the proposed cooling towers. The applicant stated that it would not use the impoundments for plant flood protection or for impounding cooling water for the operation of the plant. The pool level in Mallard Pond is below the elevation of 38.1 meters (125 ft) above msl. In the event of a dam breach at Mallard Pond, the water would drain to the north and away from the proposed new units. The pool levels in Debris Dams 1 and 2 are also below the elevation of 45.7 meters (150 ft) above msl, and, in the event of a dam breach, the water would drain to the south, away from the proposed new units. Therefore, the applicant concluded that there would be no need for embankments or dams for flood protection or for impounding the cooling water at the site.

2.5.6.2 Regulatory Basis

The applicant did not state which regulations SSAR Section 2.5.6 addressed; these topics are covered in NUREG 0800, Sections 2.4.4 and 2.5.5. However, in SSAR Section 1.8, Table 1-2, the applicant stated that it used RG 1.70 for guidance on format and content. Section 2.5.6 of RG 1.70 describes the necessary information and analysis related to the investigation, engineering design, proposed construction, and performance of all embankments used for plant flood protection or for impounding cooling water.

2.5.6.3 Technical Evaluation

In its review of SSAR Section 2.5.6, the staff evaluated the possible impact of a breach of existing embankments and dams on the proposed new units at the ESP site and evaluated the need for construction of any embankments or dams for flood protection. Based on the information provided by the applicant, the staff notes that the proposed finished grade elevation for the new units is approximately 67 meters (220 ft) above msl, and the existing pool levels for the three impoundments are 38.1 meters (125 ft) above msl for Mallard Pond, and 45.7 meters (150 ft) above msl for both Debris Basin Dams 1 and 2. These elevations are all below the proposed finished grade elevation. In addition, as the applicant discussed in Sections 2.4.3 and 2.4.4 of the SSAR, both probable maximum flood elevation (45.8 m (150.13 ft) msl) and the dam break level (54.3 m (178.10 ft) msl) are much lower than the proposed finished grade elevation Therefore, the staff concurs with the applicant's conclusion that no embankments and dams are required.

2.5.6.4 Conclusions

The applicant provided adequate information and analysis in SSAR Section 2.5.6, with reference to Sections 2.4.3 and 2.4.4 of the SSAR, regarding the embankments and dams at the ESP site. The applicant demonstrated that no embankments or dams are needed for flood protection at the ESP site under possible flood and dam breach conditions because of the proposed finished grade elevation.