Sensitivity Studies

The applicant performed two sensitivity studies to demonstrate the appropriateness of the final SSE shown in Figure 2.5.2-6.

The first sensitivity study uses a higher minimum magnitude value for each of the seismic source zones. Currently, the EPRI and LLNL studies use an m_b of 5.0 as the minimum magnitude for calculations, which corresponds to an M_w of 4.6. SSAR Section 2.5.2.6.8 states that there is “abundant evidence that earthquakes with M_w less than 5 do not cause damage to nuclear plant structures and equipment.” An M_w of 5 corresponds to an m_b of 5.4. As such, the applicant reran the PSHA with the lower bound magnitude, m_b, set to 5.4. SSAR Table 2.5-28 shows the lower mean $5x10^{-5}$ spectral accelerations resulting from a higher minimum magnitude value. The lower frequency ground motion is similar to the recommended ground motion spectrum. However, the higher frequency ground motion (i.e., 10 Hz and above) is as much as 20 percent lower than the motion at the same frequency for the performance-based spectrum derived from using a higher minimum magnitude value. The applicant stated that this result demonstrates that the recommended ground motion spectrum incorporates substantial conservatism.

For the second sensitivity study, the applicant revised the uncertainty for the base-case ground motion model to match the uncertainty values of California ground motion models. The uncertainty for CEUS ground motion models, especially for higher frequencies (i.e., 5 Hz and above), exceeds the uncertainty reported for ground motion models based on California strong-motion data. This uncertainty, referred to as aleatory uncertainty, represents the scatter of the observed ground motion about the predicted ground motion. SSAR Section 2.5.2.6.8 states that “it is not obvious that aleatory uncertainties should be higher for ground motions in the eastern U.S. than in California.” Using lower aleatory uncertainty, the applicant reran the PSHA and compared the recommended ground motion spectrum to that obtained by using the lower uncertainty. SSAR Table 2.5-28 shows the resulting ground motion spectrum using the lower aleatory uncertainty values. A comparison between this ground motion spectrum and the recommended ground motion spectrum shows that a fairly significant decrease (about 10 percent) in the selected spectrum would occur if the lower aleatory ground motion uncertainties were used in place of those reported in the 2003 EPRI ground motion study.

Future Modification of the Selected Spectrum

SSAR Section 2.5.2.6.9 describes potential modifications to the selected SSE ground motion spectrum to account for embedment and structure effects. According to the applicant, the COL application would include these modifications. The modifications to the SSE spectrum would account for horizontal and vertical spatial variation and incoherence of the ground motion, as well as scattering effects and soil-structure interaction. Horizontal spatial variation in ground motion is more prominent for structures with large plan dimensions and would reduce the input into the structure at high frequencies. SSAR Section 2.5.2.6.9 states that this occurs because the presence of large structures modifies the ground motion input to the base mat and that the modifications become significant at higher frequencies, especially above 10 Hz. The applicant concluded that the SSE spectrum is “an unrealistic input for analysis and design of structures,” and, “in order to obtain a realistic design spectrum, the Engineering Design Spectrum (EDS), factors must be considered that affect the shape of the spectrum experienced by structures with large base mats, such as those typical of nuclear power plants." The applicant referred to this "realistic design spectrum" as an engineering design spectrum (EDS).

2.5.2.1.7 Operating-Basis Earthquake

SSAR Section 2.5.2.7 describes the establishment of the operating-basis earthquake (OBE) ground motion for the ESP site. Rather than performing a detailed analysis, the applicant decided to establish the OBE earthquake spectrum as one-third of the SSE spectrum, in accordance with Appendix S to 10 CFR Part 50.

2.5.2.2 Regulatory Evaluation

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 SSAR Section 2.5.2 conforms to the requirements of 10 CFR 50.34, "Contents of Applications; Technical Information," Appendix S to 10 CFR Part 50, and 10 CFR 100.23. The applicant further stated in Section 1.8 that it developed this information in accordance with the guidance presented in NUREG-0800, Revision 3, Section 2.5.2; RGs 1.70 and 1.165; and DG-1105, "Site Investigations for Foundations of Nuclear Power Plants." (RG 1.198, of the same title, issued November 2003, superseded DG-1105 since the applicant submitted the SSAR.).

In its review of the application, 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 geologic, seismologic, 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 NUREG-0800 provides guidance concerning the evaluation of the proposed SSE ground motion, and RG 1.165 provides guidance regarding the use of PSHA to address the uncertainties inherent in the estimation of ground motion at the ESP site. 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.

2.5.2.3 Technical Evaluation

This section of the SER provides the staff's evaluation of the seismological, geological, and geotechnical investigations the applicant conducted to determine the SSE ground motion 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 they moved closer to the ESP site. The SSE is based upon a detailed evaluation of earthquake potential, taking into account regional and local geology, Quaternary 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 SSE spectrum. The SSE represents the design earthquake ground motion at the site and the vibratory ground motion for which certain nuclear power plant SSCs must be designed to remain functional. According to RG 1.165, applicants may develop the vibratory design ground motion for a new nuclear power plant using either the EPRI or LLNL probabilistic seismic hazard analyses for the CEUS. However, RG 1.165 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.

2.5.2.3.1 Seismicity

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 region. The historical earthquake catalog used in the original EPRI analysis was complete through 1984. Therefore, in addition to reevaluating the EPRI seismicity catalog, the applicant added seismicity data for the time period from 1985 through 2001.

The staff reviewed both the original EPRI seismicity catalog and the update to the catalog. The applicant added 30 more earthquakes to the regional catalog for the ESP site. Figure 2.5.2-7 depicts the earthquake epicenters in the region surrounding the ESP site. The more recent events since 1984 are shown as solid dots. The cluster of seismicity to the south-southwest of the ESP site is from the CVSZ.

Because the applicant used the EPRI seismicity catalog, which is part of the 1989 EPRI seismic hazard study that the NRC endorsed, the staff concludes that the seismicity catalog used by the applicant is complete and accurate through 1984. The staff compared the applicant's update of the regional seismicity catalog with its own listing of recent earthquakes and did not identify any significant omissions. Accordingly, the staff concludes that the applicant accurately updated the regional seismicity.

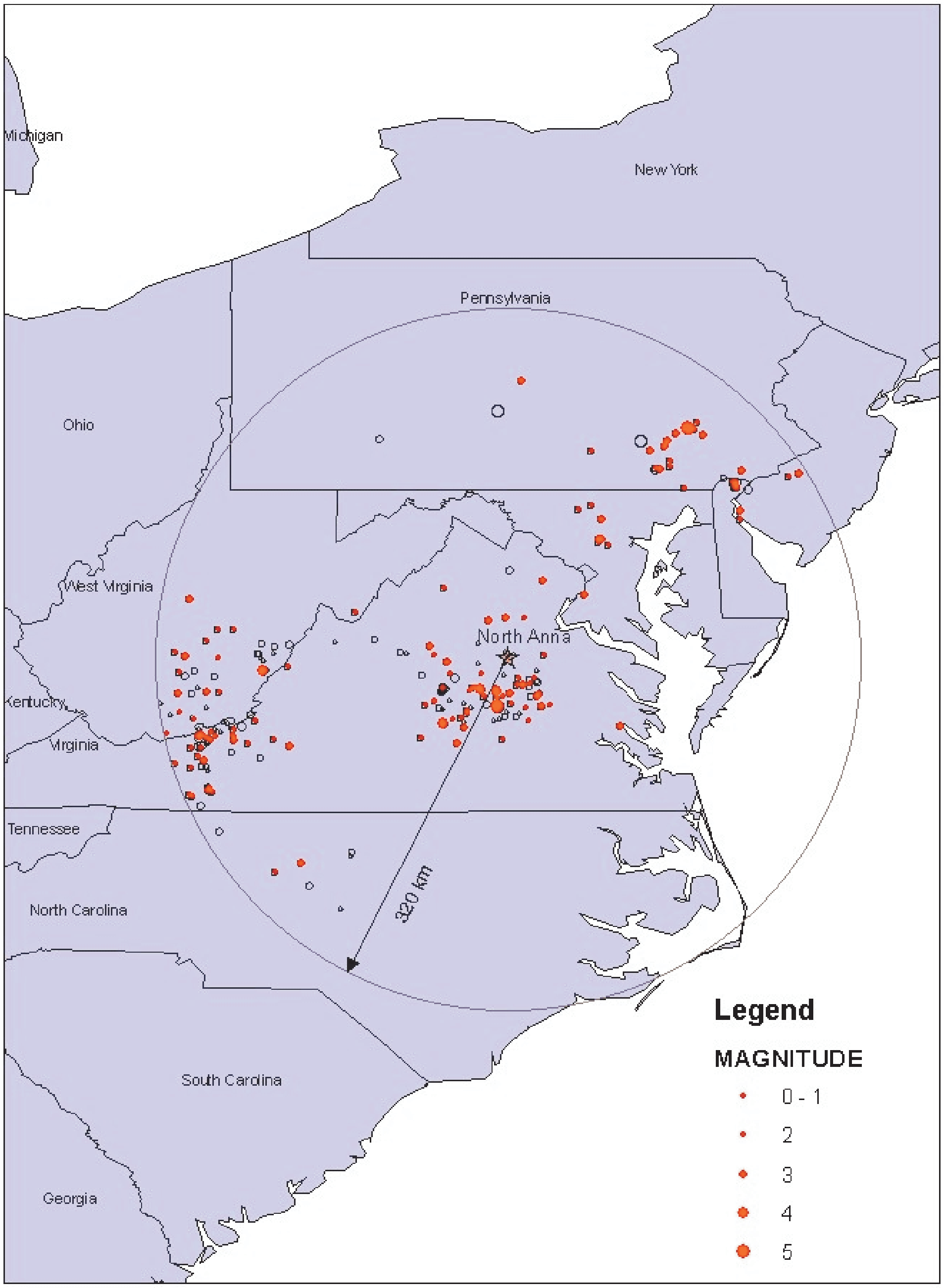

Figure 2.5.2-7 Regional seismicity for ESP site

2.5.2.3.2 Geologic and Tectonic Characteristics of the Site and Region

The staff focused its review of SSAR Section 2.5.2.2 on the applicant's characterization of potential seismic sources in the region surrounding the ESP site. The applicant evaluated recently published studies to determine if the seismic source models used for the 1989 EPRI study needed updating. The applicant concluded that no new information would suggest potentially significant modifications to the EPRI seismic source model, with the following three exceptions:

(1) the newly postulated ECFS
(2) the smaller recurrence interval for the Charleston seismic source zone
(3) the smaller recurrence interval for the NMSZ

In RAI 2.5.2-4(a), the staff asked the applicant to provide additional seismicity parameters beyond those shown in SSAR Tables 2.5-5 through 2.5-11 for the seismic sources surrounding the ESP site. In response to RAI 2.5.2-4(a), the applicant provided the recurrence parameters ("a" and "b" values) used in the EPRI study for the latitude and longitude degree cell encompassing the ESP site region. Because RG 1.165 endorsed the EPRI PSHA methodology and results, the staff used the information the applicant provided in response to RAI 2.5.2-4(a) to determine if any of the seismicity parameters should be updated. In particular, the staff asked the applicant in RAI 2.5.2-4(b) to justify its decision to not update M_{max} values assigned to the CVSZ by the 1989 EPRI ESTs considering the 1994 EPRI study, "Seismotectonic Interpretation and Conclusion from the Stable Continental Region Database." In its response to RAI 2.5.2-4(b), the applicant stated that EPRI initiated the 1994 EPRI study in the mid-1980s specifically for use by the EPRI teams in their development of the EPRI seismic source model. Each of the EPRI teams had access to the preliminary source zone geometry drawn from the 1994 EPRI study in their 1989 seismic source models. Because the M_{max} values used by the EPRI teams generally encompass the M_{max} values recommended by the 1994 EPRI study, the staff concludes that the applicant adequately characterized the seismic source zones, particularly the CVSZ, surrounding the ESP site. Section 2.5.2.1.6 of this SER summarizes the applicant's revisions to SSAR Section 2.5.2 resulting from RAI 2.5.2-4.

In RAI 2.5.2-7, the staff noted that some of the EPRI ESTs did not include the CVSZ as a specific source and asked the applicant to describe how the modern and historical seismicity of the CVSZ is distributed among either a specific source zone or a background source zone. In its response, the applicant described the source model used by each of the six EPRI teams to characterize the CVSZ. The staff reviewed each of the source models for the CVSZ that the applicant provided in its response to ensure that it had adequately characterized the seismic activity of the CVSZ. Each of the EPRI ESTs included the seismicity within the CVSZ as either a specific seismic source zone or as part of a background seismic source zone, and the staff concludes that these source models are acceptable in this respect.

Based on its review of SSAR Section 2.5.2.2 and the applicant's responses to the RAIs, as set forth above, the staff concludes that the applicant adequately investigated and characterized the regional seismic sources. The staff concludes that the 1989 EPRI seismic source models, with the exceptions noted above, remain valid for the ESP site. In addition, the staff concludes that the applicant identified those source zones that may warrant updating based on the results of its sensitivity studies which are presented in SSAR Section 2.5.2.6.

2.5.2.3.3 Correlation of Earthquake Activity with Seismic Sources

The staff focused its review of SSAR Section 2.5.2 on the applicant's efforts to correlate seismicity with known geologic features. Based on a comparison of the updated earthquake catalog to the EPRI catalog, the applicant concluded that none of the earthquakes within the site region can be associated with a known geologic structure. In addition, the applicant concluded that the updated catalog does not show a unique cluster of seismicity that would suggest a new seismic source outside of the EPRI seismic source model. Since the seismicity in the region surrounding the ESP site (see SSAR Figure 2.5-2) is not narrowly focused along any known faults or fault zones, the applicant used areal seismic source zones to characterize the seismic hazard for the ESP site. EPRI teams developed these areal source zones in the mid-1980s.

The staff compared the applicant's seismicity maps with its own and concludes that the applicant has adequately investigated the correlation of earthquake activity with known geologic sources. In particular, the staff plotted the epicenters of the most recent earthquakes surrounding the site (see SER Figure 2.5.2-4) and concurs with the applicant's conclusion that there are no new seismic sources that were not included in the 1989 EPRI seismic source model.

2.5.2.3.4 Probabilistic Seismic Hazard Analysis and Controlling Earthquakes

To evaluate the applicant's PSHA and controlling earthquakes, the staff reviewed the information presented in SSAR Sections 2.5.2.4 and 2.5.2.6. In SSAR Section 2.5.2.4, the applicant reproduced the 1989 EPRI PSHA using the 1989 seismic sources, 1989 ground motion models, and current PSHA computer program. The applicant concluded that its current PSHA computer program accurately models the 1989 EPRI results for the ESP site location.

For its PSHA, the applicant considered (1) a new regional earthquake catalog, (2) new M_{max} information, (3) new seismic source characterizations, and (4) new ground motion models. Based on PSHA sensitivity studies, which incorporate each of these four items, the applicant concluded that the more recent characterization of the Charleston seismic source recurrence interval and the new ground motion models result in significant changes to the PSHA for the ESP site.

For Revision 3 (September 2004) of Section 2.5.2 of the SSAR, the applicant repeated its deaggregation of the PSHA results to determine the controlling earthquake magnitudes and distances. Although the applicant used the same reference probability (mean 5×10^{-5}), the most recent deaggregation uses the mean PSHA hazard results rather than the median hazard results to calculate the controlling earthquakes. Because the mean hazard curves are higher than the median curves, the applicant's use of the mean curves is conservative.

In its response to RAI 2.5.2-5, the applicant explained how it incorporated the alternative characterization of the Charleston seismic source into the final PSHA. It stated that the alternative characterization of the Charleston source was evaluated both independently and additively to conservatively assess the maximum possible change to the hazard at the North Anna ESP site from the revision to this postulated source. The revisions to the Charleston source include a shorter recurrence interval (550 years) and different weights for the M_{max} (M_w 6.8 to 7.5). The ECFS-S seismic source was added to the source models for each of the six

EPRI teams for the final PSHA. Because the applicant reduced the recurrence interval, increased the weighting of higher M_{max} values, and also included the alternate source geometry of the ECFS-S into the final PSHA, the staff concludes that the applicant conservatively updated the characterization of the Charleston seismic source. This latter modification is conservative because it amounts to counting the Charleston seismic source twice. The result of these changes to the PSHA is that the low-frequency controlling earthquake for the ESP site has a magnitude of 7.2 at a distance of 308 km.

In RAI 2.5.2-2, the staff asked the applicant to provide additional details on the 2003 EPRI ground motion evaluation that it used for the ESP PSHA. To update the ground motion attenuation models in the CEUS, EPRI sponsored a Senior Seismic Hazard Advisory Committee (SSHAC) Level 3 analysis. NUREG/CR-6372 provides the guidelines for performing such an analysis. The EPRI ground motion study used 13 different ground motion attenuation relationships grouped into four clusters. In RAI 2.5.2-2(c), the staff asked the applicant to provide the weight assigned to each of the 13 ground-motion relationships within their respective cluster. For cluster 1, EPRI gave the highest weight (0.90) to the three attenuation relationships reported by Silva et al. The staff inferred from this higher weight that these relationships must have fit the data much better than other relationships. However, the applicant did not provide plots or tables of the residuals as a function of attenuation relation, magnitude, distance, and frequency. Therefore, the staff was unable to evaluate the weighting EPRI selected for cluster 1. Similarly, for clusters 2 and 3, the ground motion experts applied higher weights to different attenuation relationships within each cluster. Neither the EPRI 2003 ground motion report nor the applicant's response to RAI 2.5.2-2 provides the rationale for these weights.

In RAI 2.5.2-2(b), the staff asked the applicant to provide additional information on the Silva et al. cluster 1 attenuation relationships. In response, the applicant provided additional documentation on these attenuation relationships. The Silva et al. cluster 1 relationships use an expression for the seismic attenuation parameter, Q, that is frequency dependent. This frequency-dependent Q value was derived from an inversion of the data from the 1988 Saguenay earthquake. This inversion solves for Q, as well as the local site attenuation parameter kappa and the stress drop, which is the difference between the initial stress before and earthquake and the final stress. The staff was unable to determine how the recordings from a single earthquake can provide well-resolved values of both crustal Q and site kappa. In addition, the Q value of 317 at 1 Hz is much lower than values found in other studies of eastern North American earthquakes. In addition, other studies have found less frequency dependence of Q in the east than in the west, which is contrary to the findings of Silva et al.

In RAI 2.5.2-2(d), the staff asked the applicant to explain the weights given to each of the four clusters. In response to RAI 2.5.2-2, the applicant stated that the expert panel members, convened for the EPRI ground motion study, were asked to subjectively evaluate how well the alternative ground motion models relied on seismological principles. The staff considers the applicant's response to RAI 2.5.2-2(d) to be somewhat indirect. The applicant provided additional information, but the details still remain abstract in terms of specific "seismological principles." The response emphasizes the ranking of model clusters and the judgments involved in balancing data consistency and adherence to seismological principles. However, the applicant provided only abstract and very general references to these seismological principles. As a result, the staff was unable to evaluate the criteria or the weights applied to the four clusters.

In Open Item 2.5-1, the staff requested clarification and further information from the applicant regarding each of the three issues outlined above. With regard to the unequal weighting for the cluster 1 attenuation relationships (RAI 2.5.2-2(a)), the applicant provided the staff with tables of statistics that compare each of the ground motion relationships and the CEUS earthquake database. For each model and ground motion frequency, the applicant determined the deviation between the median model prediction and the actual recorded motion. Using the mean and variance of the deviations, the applicant determined the weight for each model in cluster 1. In addition to the tables of statistics, the applicant also provided plots of residuals for each of the cluster 1 ground motion models and plots comparing the final overall cluster 1 model to the actual CEUS earthquake data.

With regard to the staff's concerns, described above in RAI 2.5.2-2(b), concerning the Silva et al. cluster 1 attenuation relationships, the applicant stated the following:

> The model functional form, basis for parameter selection, and the results developed in Silva et al. (2002) and its predecessor, Silva et al. (1997), are the responsibility of the lead author. Of particular relevance is the interdependence between model parameters, how the parameters were determined, model sensitivity to its parameters, and reasonable ranges in parameter values, based on expert judgement and expert interpretation of the scientific literature. It is unclear if a summary justification for the results of the Silva et al. (1997 and 2002) studies would resolve the items identified that seem, ultimately, to represent differences in expert judgement.
>
> Differences in expert judgement are often difficult to reconcile. For this very reason, the SSHAC [Senior Seismic Hazard Advisory Committee] process was developed and accepted for use by the NRC. The EPRI 2003 ground motion model was developed by implementing a SSHAC Level 3 assessment process during which the EPRI Expert Panel identified the Silva et al. relationships as ones that should be included in the assessment and evaluated. The EPRI Expert Panel considered specific parameterizations of individual ground motion relationships in determining whether or not a relationship should be included in the SSHAC Level 3 assessment process. All ground motion relationships identified as viable by the Expert Panel were evaluated using the same criteria following the SSHAC Level 3 process.
>
> The SSHAC process does not guarantee that every scientist will agree with the assessments. It is rather intended to assure that the assessed results reflect the preponderance of current scientific views, which is the underpinning of safety decisionmaking.

Since the EPRI 2003 expert panel members gave the three Silva et al. attenuation relationships the highest overall weight (0.90) in cluster 1, the staff asked the applicant to explain whether this biased the final overall cluster 1 ground motion model towards the model functional form and parameters used by these three attenuation relationships. Specifically, the three Silva et al. attenuation relationships each have different earthquake source terms and parameters; however, these relationships have the same wave propagation travel path terms and parameters. As such, the staff asked the applicant to explain if this limited path variability biased the overall cluster 1 ground motion model. In response to the staff's concern, the applicant stated the following:

> The ground motion models in Cluster 1 considered a range of alternative stress drop models and alternative Q and path models. Collectively, these models represent alternative single-corner [shape] source spectrum models for the CEUS. In aggregate, these models provide a measure of the epistemic [modeling] uncertainty in the median ground motion based on the single-corner source spectrum models (e.g., intra-cluster variability).

The applicant also stated that, as part of the CEUS model development, EPRI evaluated whether an additional component of uncertainty for wave propagation travel path effects should be included for each of the model clusters. The individual models within each model cluster contribute to the overall cluster variability since they each use different source and path parameters. However, the EPRI (2004) report states that there may be additional variability in the modeling parameters that is not captured by the ground motion models that make up a cluster. As described above, the staff expressed concern that the path variability for cluster 1 may be too small since the three Silva et al. attenuation relationships, which have an overall weight of 0.90, each have the same travel path model terms and parameters. EPRI, as part of its ground motion assessment, compared the overall cluster 1 ground motion variability (both source and path) with the variability of different path model terms and parameters used by the different individual models. In other words, EPRI isolated the travel path variability by equally weighting each of the alternative travel path models and compared this variability to the overall variability for each of the ground motion clusters. Figure 4-6 of the EPRI (2004) ground motion report shows this travel path variability, and Figure 4-2 of the report depicts the cluster 1 variability. Comparing the variability shown in Figures 4-2 and 4-6, the applicant concluded that "these variabilities were similar, although the results in Figure 4-6 are higher, particularly at distances beyond 100 km." The applicant stated that most of the models in cluster 1 had already "considered the variability in path effects as aleatory [e.g., random scatter] variability and thus it is ultimately included in the overall probabilistic hazard analysis."

With regard to the staff's concerns, described above in RAI 2.5.2-2(d), the latest version of the EPRI ground motion report provides an expanded explanation of the seismological principles that the expert panel members used to determine the overall weight for each of the four clusters. The seismological principles considered by the expert panel members include (1) seismic source modeling, (2) crustal wave propagation, and (3) near-surface crustal effects. Based on the single criterion of seismological principles, the four ground motion clusters were weighted fairly equally (0.245, 0.221, 0.257, and 0.277). In addition to seismological principles, the expert panel members also relied on consistency with the CEUS earthquake database and the modeling of variability as criteria for determining the final overall cluster weights (0.275, 0.312, 0.196, and 0.217).

For its review of the applicant's response to Open Item 2.5-1, the staff examined the plots and tables of model residuals provided by the applicant for the cluster 1 ground motion models. The staff verified that, for the ground motion frequencies (1, 5, and 10 Hz), the three Silva et al. ground motion models do provide the smallest mean residual values (i.e., best fit to the earthquake data) compared to the other cluster 1 models. As a result, EPRI gave weights of 0.192, 0.148, and 0.560 to these three ground motion models.

To resolve the concern that these three models, which account for 90 percent of the overall cluster 1 model, do not adequately represent the variability in travel path, the staff compared Figures 4-2 and 4-6 in the EPRI (2004) ground motion report. As noted by the applicant, there is a slightly higher variability for distances beyond 100 km as shown in Figure 4-6. This result suggests that travel path variability for the overall cluster 1 model may be somewhat low. However, for source distances out to about 300 km, the differences in variability are negligible. This result implies that the overall cluster 1 model uncertainty contains a sufficient amount of travel path variability.

To resolve the concern regarding the use and application of seismological principles to assign final overall weights to each of the four cluster groups, the staff reviewed the new information provided in the latest version of the EPRI ground motion report. Based on the criterion of seismological principles, the EPRI expert panel members gave similar weights to each of the four ground motion clusters. This result implies that the EPRI expert panel members did not find significant differences among the four model clusters regarding the use of seismological principles. The staff also reviewed the seismological principles used by the expert panel members and determined that these principles are relevant and significant for ground motion estimation.

In conclusion, as described above, the applicant has adequately resolved each of the staff's concerns with regard to the development by EPRI of new ground motion models for the CEUS. Therefore, the staff concludes that the applicant's use of the EPRI (2004) ground motion attenuation models provides an adequate estimate of the ground motion for CEUS earthquakes and, as such, an adequate characterization of the seismic hazard for the ESP site.

The staff concludes that the applicant's PSHA adequately characterized the overall seismic hazard of the ESP site. As set forth above, the staff finds that the applicant's underlying assumptions and update of the previous EPRI PSHA adequately incorporate the most recent studies and evaluations of the seismic source zones surrounding the ESP site. The staff also concludes that the applicant's controlling earthquakes for the ESP site (magnitude of 5.4 at 20 km and magnitude of 7.2 at 308 km) are generally consistent with previous PSHA results for the region. In addition, the staff finds that the ground motions developed by the applicant from the controlling earthquakes are consistent with the most recent ground motion evaluations. Accordingly, the staff concludes that the applicant followed the guidance in RG 1.165 for evaluating the regional earthquake potential and determining the ground motion resulting from the controlling earthquakes. Based on the foregoing, the staff considers Open Item 2.5-1 to be resolved.

2.5.2.3.5 Seismic Wave Transmission Characteristics of the Site

The staff focused its review of SSAR Section 2.5.2.5 on the applicant's incorporation of the seismic wave transmission characteristics of the material overlying the base rock at the site into the determination of the SSE. SSAR Section 2.5.4.7 provides a description of the transmission characteristics of the site material.

In RAIs 2.5.2-1(c) and 2.5.2-8, the staff asked the applicant to explain how it factored the properties of the site-specific subsurface materials into the determination of the SSE. According to the applicant's responses, it calculated the SSE directly using the EPRI 2003 ground motion models, which assume generic hard rock conditions for all of the CEUS. The shear wave velocity assumed by the EPRI 2003 ground motion models for the generic hard rock conditions is 9200 ft/s. The applicant stated that, since the containment (reactor) building and primary supporting safety-related structures would be founded on sound bedrock, either Zone IV or Zone III-IV rock, the generic hard rock conditions assumed by the EPRI 2003 ground motion report are a “good approximation” for the ESP site. As such, the applicant did not factor in any of the local ESP site properties for its determination of the SSE.

As set forth in the DSER, the staff considered the applicant’s response above to be inadequate based on a comparison of the average bedrock Zone III-IV shear wave velocity (3300 ft/s) and the generic hard rock shear wave velocity (9200 ft/s) assumed by EPRI 2003. SSAR Figure 2.5-62 shows that the measured shear wave velocity values for the upper soil and rock layers beneath the ESP site are below that of the hard rock conditions assumed by EPRI 2003. Thus, the hard rock shear wave velocity of 9200 ft/s may not be reached at the ESP site until a considerable depth below the ground surface. In addition, 10 CFR 100.23(d)(1) states the following:

> The Safe Shutdown Earthquake Ground Motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface.

Therefore, as further set forth in the DSER, the staff determined that the applicant’s SSE did not represent the free-field ground motion at the free ground surface. Open Item 2.5-2 covered the necessity to include the local ESP site conditions into the determination of the SSE.

In response to Open Item 2.5-2, the applicant reran its analysis to determine the response of the ESP site at the free ground surface, as required by 10 CFR 100.23(d)(1). The applicant’s new analysis use a rock subsurface profile that extends from the top of Zone III-IV bedrock to a depth of 160 ft under the site where the shear wave velocity reaches about 9200 ft/s. The applicant defined the top of rock layer Zone III-IV to be its control point for consistency with the guidance in Section 3.7.1 of NUREG-0800, which states the following:

> For sites composed of one or more thin soil layers overlying a competent material or in case of insufficient recorded ground-motion data, the control point is specified on an outcrop or a hypothetical outcrop at a location on the top of the competent material.

The applicant used the ESP rock subsurface profile to estimate the ground motion amplification of the site and, therefore, to determine an SSE that incorporates the local site rock properties. To determine the control point SSE at the top of Zone III-IV rock, the applicant (1) developed a shear wave velocity profile for the ESP site, (2) generated alternative randomized rock columns to incorporate the variability in the rock properties, (3) selected seed earthquake time histories, and (4) performed the final ground response analysis. SER Section 2.5.2.1.6 describes each of these steps in detail.

The staff reviewed the applicant’s analysis to ensure that it accurately incorporates the local site properties and conditions as well as their uncertainties. The applicant developed 50 different randomized rock columns in order to model the uncertainties in the rock properties, such as shear wave velocities, densities, and damping values. The staff also verified that the response spectra from the two earthquake time histories used by the applicant for its convolution match the low- and high-frequency spectra from the two controlling earthquakes. As a result of the applicant's inclusion of the local site rock properties, some of the spectral acceleration values for the final SSE ground motion spectrum increased by as much as a factor of 1.67. As shown previously in Figure 2.5.2-6, these increases mainly occur at frequencies above 10 Hz. The staff concludes that the applicant's site response analysis accurately incorporates the local site properties as well as the variability in these properties. Based on the above, the staff considers Open Item 2.5-2 to be resolved.

2.5.2.3.6 Safe-Shutdown Earthquake Ground Motion

The staff focused its review of SSAR Section 2.5.2.5 on the applicant's procedure to determine the SSE. For SSAR Revision 3, issued in September 2004, the applicant used two different methods to determine the ground motion response spectra for the ESP site.

Originally, the applicant used a new method to determine the site SSE, referred to as a performance-based approach. In RAI 2.5.2-1, the staff asked the applicant to explain how the performance-based approach meets the requirements of 10 CFR 100.23, which provide the geologic and seismic siting criteria as well as a definition of the SSE. In response to RAI 2.5.2-1, the applicant explained how the performance-based approach conforms with the requirements of 10 CFR 100.23. In RAI 2.5.2-9, the staff asked the applicant for further details on the performance-based approach beyond those provided in SSAR Section 2.5.2.6. In response to RAI 2.5.2-9, the applicant provided further justification for the performance-based approach, including the derivation of some of the key relationships.

After reviewing the applicant's responses to RAIs 2.5.2-1 and 2.5.2-9 regarding its performance-based approach, the staff informed the applicant that it would need to devote additional time and resources to review this new method. In a letter dated August 19, 2004, the applicant informed the staff that it would revise SSAR Section 2.5.2 to base the selected SSE on the reference probability approach, in accordance with RG 1.165. The applicant also indicated that it would retain the performance-based approach in the SSAR as "alternate and further justification for the final SSE." Since the applicant has chosen to determine the final SSE in accordance with RG 1.165, the staff decided that it will not evaluate the performance-based approach for conformance with the requirements of 10 CFR 100.23 or review the overall acceptability of the approach. Therefore, the staff did not reach any conclusion with respect to the information in the SSAR regarding the performance-based approach or the applicant's responses to RAIs 2.5.2-1 and 2.5.2-9 that pertain to the performance-based approach.

In conjunction with its decision to base the final SSE on the reference probability approach in accordance with RG 1.165, the applicant also decided to use a higher reference probability ($5x10^{-5}$) than that recommended by RG 1.165 ($1x10^{-5}$). In addition, the applicant chose to use the mean PSHA curves rather than the median curves. Because the mean hazard curves are higher then the median curves, the applicant's use of the mean curves is conservative. In RAI 2.5.2-1(d), the staff asked the applicant to justify the proposed higher reference probability. In response to RAI 2.5.2-1(d), the applicant stated that it used a higher reference probability because of (1) higher ground motion estimates from the 2003 EPRI ground motion models, (2) shorter recurrence intervals for the New Madrid and Charleston seismic sources, and (3) the use of the mean hazard instead of the median hazard. Each of these factors (particularly the first two) increase the overall seismic hazard level for the CEUS and specifically, for the 29 nuclear power plant sites used to determine the original reference probability. Because the reference probability recommended in RG 1.165 ($1x10^{-5}$) is based on the LLNL and EPRI PSHAs from the late 1980s, the staff concurs with the applicant's conclusion that this value is likely to be out of date and overly conservative.

To evaluate the applicant's use of a higher reference probability ($5x10^{-5}$) and use of mean rather than median PSHA results, the staff performed an independent analysis to reevaluate the reference probabilities for the 29 nuclear power sites in the CEUS that were used to determine the original reference probability. For its independent analysis, the staff used the most recent 2002 USGS PSHA mean and median hazard curves to determine the probability of exceeding the SSEs for the 29 CEUS sites. The staff also applied the same 5 Hz and 10 Hz site correction factors that were used in the LLNL seismic hazard analysis, published in 1993. Although the staff has not officially endorsed the 2002 USGS PSHA results, the staff was able to verify that the reference probability proposed by the applicant ($5x10^{-5}$) is sufficiently conservative. This larger reference probability value ($5x10^{-5}$) implies a lower return period (20,000 yrs) for the design ground motion; however, the staff was able to verify through its analysis that this revised reference probability results in a final SSE of adequate severity that is representative of the seismic hazard for the ESP site.

Using the RG 1.165 approach, the applicant determined the ground motion response spectra for the ESP site controlling earthquakes (magnitude of 5.4 at 20 km and magnitude of 7.2 at 308 km). The applicant then enveloped these two response spectra with the performance-based spectrum to create the final SSE spectrum. The staff's acceptance of the use of the performance-based spectrum to envelope the two controlling earthquake response spectra does not imply that the staff has endorsed the performance-based approach. As described in Appendix F to RG 1.165, any smooth spectral shape that envelopes the two controlling earthquake response spectra is acceptable as the site SSE. However, as set forth in the DSER, the staff (see Open Item 2.5-2) determined that this final SSE did not meet the requirements specified in 10 CFR 100.23(d)(1), which states that "the Safe Shutdown Earthquake Ground Motion for the site is characterized by both horizontal and vertical free-field ground motion response spectra at the free ground surface." As discussed above, the applicant addressed the staff's concern by performing a detailed site response analysis that incorporates the local site properties as well as the variability in these properties. Therefore, the final ESP site SSE meets the requirements specified in 10 CFR 100.23 in that it incorporates the local site subsurface properties and represents the free-field ground motion.

In SSAR Sections 2.5.2.6.9 and 2.5.2.6.10, the applicant alluded to future modifications of the site SSE spectrum in order to obtain an engineering design spectrum (EDS) that represents "the proper input into the large nuclear power plant structures." The applicant stated that the ESP site SSE is not suitable for the design of the SSCs of nuclear power plants because of high spectral accelerations in the high-frequency range (about 15 to 30 Hz). According to the applicant, the EDS would take into account plant-specific structural characteristics and local site conditions, as well as the ESP SSE spectrum. However, the ESP application does not include the EDS because the applicant has not selected a specific reactor design. The applicant proposed to include the EDS as part of a COL application. Because the applicant did not provide any specific recommendations or procedures for developing the EDS, the staff cannot evaluate the merits of the proposed approach.

The staff considers the SSE developed for the ESP site to be consistent with Appendix S to 10 CFR Part 50, which defines the SSE as the "vibratory ground motion for which certain

structures, systems, and components must be designed to remain functional."
Section 2.5.2.3.5 of this SER addresses the applicant's compliance with 10 CFR 100.23(d) with regard to the SSE. Future modifications of the SSE spectrum, if any, in an application for a COL or CP must be compatible with 10 CFR Parts 50 and 100.

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 and the applicant's responses to the RAIs and open items, as described above, 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 that this PSHA follows the guidance provided in RG 1.165. 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 SSE was determined in accordance with RG 1.165 and Section 2.5.2 of NUREG-0800 and accurately includes the effects of the local ESP subsurface properties. The staff concludes that the proposed ESP site is acceptable from a geologic and seismologic standpoint and meets the requirements of 10 CFR 100.23.

2.5.3 Surface Faulting

SSAR Section 2.5.3 describes the potential for tectonic fault rupture at the ESP site. The applicant concluded that the site has no potential for tectonic fault rupture since no capable tectonic sources exist within a 5-mile radius of the ESP site. SSAR Section 2.5.3.1 describes the applicant's geological, seismological, and geophysical investigations to assess the potential for surface faulting within a 5-mile radius of the ESP site. SSAR Section 2.5.3.2 describes the geologic evidence, or absence of evidence, for surface deformation. SSAR Section 2.5.3.3 describes the correlation of earthquake epicenters with faults in the vicinity of the ESP site. SSAR Section 2.5.3.4 provides the ages of the most recent deformations in the site area. Finally, SSAR Sections 2.5.3.5 through 2.5.3.8 describe tectonic structures in the site area, the absence of capable sources and Quaternary deformation, and the potential for tectonic or nontectonic deformation at the site.

2.5.3.1 Technical Information in the Application

2.5.3.1.1 Surface Faulting Investigations

<u>Geological, Seismological, and Geophysical Investigations</u>

According to SSAR Section 2.5.3.1, the applicant performed the following investigations to assess the potential for surface faulting at and within a 5-mile radius of the ESP site:

- compilation and review of existing data
- interpretation of aerial photography
- field reconnaissance
- review of seismicity

- discussions with current researchers in the area

Based on previous site investigations performed for the existing NAPS Units 1 and 2, the applicant concluded that (1) no evidence of surface rupture, surface warping, or the offset of geomorphic features indicative of active faulting exists, (2) no historical seismic activity has occurred in the site area, as the closest epicenter location is 30 miles away, and (3) inspections of excavations during construction and examination of soil and rock samples from borings reveal no evidence of geologically recent faulting.

The applicant performed aerial and field reconnaissance investigations within a 25-mile radius of the ESP site, and it examined and interpreted aerial photographs of all known faults within 5 miles of the site. Through these studies, the applicant verified the existence of mapped bedrock faults in the site area and assessed the presence or absence of geomorphic features that indicate potential Quaternary fault activity.

In addition to its own investigations, the applicant used USGS maps of the area, as well as a USGS compilation of all Quaternary faults, liquefaction features, and possible tectonic features in the eastern United States, to assess the potential for surface faulting within a 5-mile radius of the ESP site.

<u>Geologic Evidence for Surface Deformation</u>

SSAR Section 2.5.3.2 lists the following bedrock faults that are within 5 miles of the ESP site:

- Chopawamsic fault
- Spotsylvania thrust fault
- unnamed faults “a,” “b,” and “c”
- Sturgeon Creek fault
- Long Branch thrust fault

All of these faults formed during the early Paleozoic Era as part of the regional Taconic orogeny and may have become reactivated during later Paleozoic orogenies (Acadian and Allegheny). The applicant stated that several of the faults may have been locally reactivated during the Triassic episode of continental rifting; however, none of these faults border Triassic basins, implying that Triassic reactivation, if any, was not significant. Figure 2.5.3-1, reproduced from SSAR Figure 2.5-56, shows these Paleozoic faults on an ESP site vicinity geologic map.

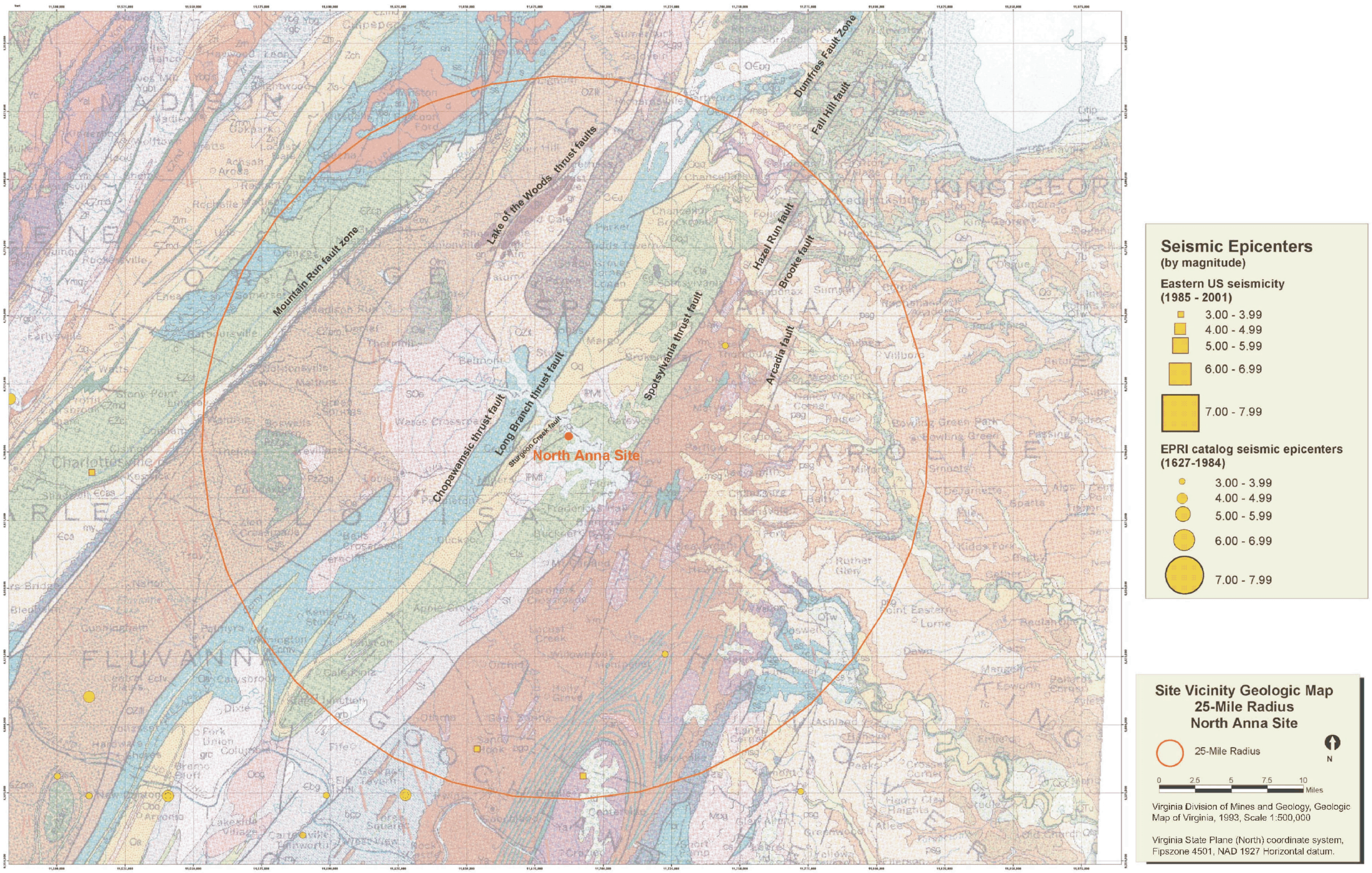

Figure 2.5.3-1 Site vicinity geologic map and seismicity (25-mile radius)

SSAR Section 2.5.3.2 states that the applicant identified no deformation or geomorphic features indicative of potential Quaternary activity in the literature or during aerial and field reconnaissance. In addition, the recent USGS compilation of all Quaternary faults, liquefaction features, and possible tectonic features in the eastern United States includes none of the faults listed above as potential Quaternary faults.

In RAI 2.5.3-1, the staff asked the applicant to provide additional detail on its field investigations and aerial reconnaissance of the site area. In response to RAI 2.5.3-1, the applicant stated that it performed aerial and field reconnaissance along faults within a 5-mile radius of the ESP site. The applicant's reconnaissance emphasized unnamed fault "a" and the Sturgeon Creek fault because of their proximity to the site. In addition, the applicant covered parts of the Spotsylvania, Chopawamsic, and Long Branch faults where these faults were mapped near local roads and/or where they potentially offset plutonic (igneous) margins or metamorphic contacts. Based on the absence of any geomorphic expression indicative of potential Quaternary deformation, the applicant concluded that none of the faults are capable. In addition, the applicant stated that all of the faults in the site area cross gently rolling topography, with relief on the order of 200 ft, and that this rolling topography formed through dissection and erosion of a once broad, continuous Miocene (5-24 ma) pediment that extended across the region. The applicant looked for potential elevation differences in the Miocene pediment gravels across each of the faults that would suggest post-Miocene vertical separation. Based on its field reconnaissance, the applicant did not observe any significant elevation differences. Therefore, the applicant concluded based on its detailed field observations and aerial reconnaissance that, for all seven faults within the site area, no evidence or criteria would suggest Quaternary activity on these structures.

Foundation excavations for the abandoned NAPS Units 3 and 4 exposed the unnamed bedrock fault "a" traversing the North Anna site. Detailed investigations of this fault show no evidence of Quaternary faulting. Therefore, the applicant concluded that this fault is not a capable tectonic source. In reviewing the applications for construction permits for abandoned NAPS Units 3 and 4, the applicant indicated that the Atomic Energy Commission (which subsequently became the NRC) accepted this position in its 1974 SER for Units 3 and 4.

In RAI 2.5.3-2, the staff asked the applicant to further support its conclusion that unnamed fault "a" does not extend beyond the ESP site, as mapped by Pavlides (Ref. 36, SSAR Section 2.5). In its response, the applicant stated that the NAPS licensee discovered fault "a" in 1973 during the foundation excavation for the abandoned NAPS Units 3 and 4 and subsequently mapped fault "a" for a distance of about 3000 ft. Virginia Power did not observe fault "a" in the foundation excavations for the existing Units 1 and 2. The applicant stated that Pavlides, who is deceased, did not provide an explanation for extending fault "a" for a total distance of about 7 miles. Subsequently, Mixon and others (Ref. 66, SSAR Section 2.5) adopted Pavlides' interpretation of the extent of fault "a." The applicant stated that Pavlides did not map any offset stratigraphic contacts in the Lake Anna area to support the mapped location of the fault. In addition, a close inspection of the original mapping by Pavlides compared to the compilation map by Mixon shows that the offsets that are apparently mapped in the stratigraphic contacts appear to be a compilation error. The applicant provided further evidence to support its original mapping of fault "a" in response to RAI 2.5.3-1.

Correlation of Earthquake with Capable Tectonic Sources

SSAR Section 2.5.3.3 states that no reported historical earthquake epicenters have been associated with bedrock faults within a 25-mile radius of the ESP site vicinity. The applicant established a seismic monitoring network for NAPS and recorded very small earthquakes (microearthquakes) over a 3.5-year period from 1974 to 1977. The applicant used this monitoring program to determine if seismic activity could be associated with faults in the site area or if Lake Anna was producing reservoir-induced seismicity. The applicant concluded that the microearthquakes detected in the site area could not be associated with either faults in the site area or with the impoundment of Lake Anna. Four of the original 17 seismic monitoring stations in the network were incorporated into the VT Central Virginia Monitoring Network for the specific purpose of monitoring any changes in seismicity in the region of the NAPS. To date, no changes in local earthquake activity have been observed that would alter the conclusions regarding the lack of association of microearthquakes with faults in the site area. Microearthquakes in the site area occur at a level no greater than the spatially varying background activity found in the CVSZ.

Ages of Most Recent Deformations

SSAR Section 2.5.3.4 states that none of the seven faults within 5 miles of the ESP site exhibit evidence of Quaternary activity. All of these faults formed during the Paleozoic Era as part of the Taconic orogeny and may have been reactivated during later Paleozoic orogenies or during the Triassic continental rifting. Based on a review of the available literature and field investigations, the applicant concluded that the seven bedrock faults within 5 miles of the site are old structures that formed during the Paleozoic-age orogenies or early Mesozoic-age rifting.

Relationship of Tectonic Structures in Site Area to Regional Tectonic Structures

SSAR Section 2.5.3.5 states that the seven faults in the site area are located within the Chopawamsic belt, which is interpreted to be an island-arc that was accreted to North America during the Taconic orogeny. Following the Taconic orogeny, rocks of the Chopawamsic belt were deformed and thrust westward during the Acadian and Allegheny orogenies that occurred later during the Paleozoic Era. Extensional tectonics may have also affected the rocks in the Chopawamsic belt during the Mesozoic rifting.

Characterization of Capable Tectonic Sources

SSAR Section 2.5.3.6 states that no capable tectonic sources exist within 5 miles of the ESP site.

Designation of Zones of Quaternary Deformation Requiring Detailed Fault Investigations

SSAR Section 2.5.3.7 states that no zones of Quaternary deformation warrant detailed investigations within the site area.

Potential for Tectonic or Nontectonic Deformation at the Site

SSAR Section 2.5.3.8 states that the ESP site has a negligible potential for tectonic deformation. Since the original studies in the early 1970s, no new information has been reported to suggest the existence of any Quaternary surface faults or capable tectonic sources within the site area. In addition, the site shows no evidence of nontectonic deformation, such as glacially induced faulting, collapse structures, growth faults, salt migration, or volcanic intrusion.

2.5.3.2 Regulatory Evaluation

SSAR Section 2.5.3 describes the applicant's evaluation of the potential for surface deformation that could affect the site. In SSAR Section 1.8, the applicant stated that the information presented in SSAR Section 2.5.3 conforms with the requirements of GDC 2 of Appendix A to 10 CFR Part 50, Appendix S to 10 CFR Part 50, and 10 CFR 100.23. The applicant also stated that it developed the geological, seismological, and geophysical information used to evaluate the potential for surface deformation in accordance with the guidance presented in NUREG-0800, Revision 3, Section 2.5.3, and RGs 1.70, 1.132, 1.165, and 4.7.

In its review of the application, the staff considered the regulatory requirements in 10 CFR 100.23(d)(2), which state that an applicant for an ESP must determine the potential for surface tectonic and nontectonic deformations. The staff notes that application of Appendix S in an ESP review, as referenced in 10 CFR 100.23(d), is limited to defining the minimum SSE for design. Section 2.5.3 of NUREG-0800 and RG 1.165 provide specific guidance concerning the evaluation of information characterizing the potential for surface deformation, including the geological, seismological, and geophysical data that the applicant must provide to establish the potential for surface deformation.

2.5.3.3 Technical Evaluation

This section of the SER provides the staff's evaluation of the seismological, geological, and geophysical investigations carried out by the applicant to address the potential for surface deformation that could affect the site. The technical information presented in SSAR Section 2.5.3 resulted from the applicant's surface and subsurface investigations performed in progressively greater detail as they moved closer to the ESP site. Through its review, the staff determined whether the applicant complied with the applicable regulations and conducted its investigations with an appropriate level of thoroughness.

In order to thoroughly evaluate the surface faulting investigations performed by the applicant, the staff sought the assistance of the USGS. The staff and its USGS advisors visited the ESP site and met with the applicant to assist in confirming the interpretations, assumptions, and conclusions presented by the applicant concerning potential surface deformation. Specific areas of review include the geological investigations (SSAR Section 2.5.3.1), evidence for surface deformation (SSAR Section 2.5.3.2), correlation of earthquake activity with capable seismic sources (SSAR Section 2.5.3.3), ages of most recent deformations (SSAR Section 2.5.3.4), site area and regional tectonic relationships (SSAR Section 2.5.3.5), characterization of capable tectonic sources (SSAR Section 2.5.3.6), Quaternary deformation in the site region (SSAR Section 2.5.3.7), and the potential for surface tectonic deformation at the site (SSAR Section 2.5.3.8).

2.5.3.3.1 Surface Faulting Investigations

The staff focused its review of SSAR Sections 2.5.3.1 through 2.5.3.8 on the adequacy of the applicant's investigations to ascertain the potential for surface deformation that could affect the site. The staff reviewed the applicant's summary of previous site investigations performed for the existing NAPS Units 1 and 2 and the abandoned NAPS Units 3 and 4, as well as recent investigations.

In RAI 2.5.3-1, the staff asked the applicant to provide additional detail on its field investigations and aerial reconnaissance of the site area. In its response, the applicant stated that it performed aerial and field reconnaissance along each of the faults within a 5-mile radius of the ESP site. The staff reviewed the evidence presented by the applicant's response to RAI 2.5.3-1, particularly the applicant's documentation of its field reconnaissance. Specifically, the staff reviewed the applicant's description of its search for evidence of Quaternary deformation for each of the faults, including the applicant's field observations across the Miocene pediment that extends across the region. The staff and its USGS consultants also visited the site area and viewed the continuous, gently inclined Miocene surface referred to in the applicant's response. The staff did not observe any significant vertical displacements that would indicate post-Miocene (5-24 ma) displacement or activity. In summary, the staff and its consultants did not observe evidence for Quaternary activity on any of these local faults and conclude that the applicant has adequately investigated the potential for surface deformation as required by 10 CFR 100.23.

In RAI 2.5.3-2, the staff asked the applicant to further support its conclusion that unnamed fault "a" does not extend beyond the ESP site as mapped by Pavlides (Reference 36, SSAR 2.5.2). In its response, the applicant stated that Virginia Power discovered fault "a" in 1973 during the foundation excavation for the abandoned NAPS Units 3 and 4 and subsequently mapped fault "a" for a distance of about 3000 ft. Virginia Power did not observe fault "a" in the foundation excavations for the existing NAPS Units 1 and 2. The applicant stated that Pavlides, who is deceased, did not provide an explanation for extending fault "a" for a total distance of about 7 miles. Subsequently, Mixon and others (Ref. 66, SSAR Section 2.5.2) adopted Pavlides' interpretation of the extent of fault "a." The applicant stated that Pavlides did not map any offset stratigraphic contacts in the Lake Anna area to support the mapped location of the fault. In addition, the applicant's inspection of the original mapping by Pavlides compared to the compilation map by Mixon showed that the offsets apparently mapped in the stratigraphic contacts appear to be a compilation error. During its field reconnaissance, the applicant found no scarps or lineaments along the extended trace of fault "a" as mapped by Pavlides. The staff notes that the NAPS licensee's trenching of the fault "a" shows that it is most likely a minor fault or bedrock shear within the Ta River metamorphic suite and that it is very unlikely that such a minor fault could be recognized or mapped over a significant distance without a significant number of exposures. The applicant provided further evidence, described above, to support its original mapping of fault "a" in response to RAI 2.5.3-1. Based on this evidence, the staff concludes that fault "a" is unlikely to extend much farther than originally mapped by the applicant.

In SSAR Table 1.9-1, the applicant identified the item "Capable Tectonic Structures or Sources" as an ESP site characteristic. This item specifies that no fault displacement potential exists within the investigative area. As described above, the staff reviewed the applicant's description of unnamed fault “a” in SSAR Section 2.5.3.2.2 and concludes that the ESP site has no fault displacement potential.

Based on its review of SSAR Sections 2.5.3.1 through 2.5.3.8 and the applicant’s responses to the RAIs, as set forth above, the staff concludes that the applicant adequately investigated the potential for surface faulting in the site area. The staff concludes that the applicant performed extensive field and aerial reconnaissance of the local faults and concurs with the applicant’s assertion that no capable faults exist within the site area. The staff and its USGS consultants also visited the site area and were able to view some of these local faults. Based on its site visit and its review of SSAR Section 2.5.3, as set forth above, the staff concurs with the applicant’s conclusion that there is no evidence of Quaternary folding or faulting that could be associated with these local faults.

2.5.3.4 Conclusions

In its review of the geologic and seismologic aspects of the ESP site, the staff considered the pertinent information gathered by the applicant during the regional and site-specific geological, seismological, and geophysical investigations. As a result of this review, described above, the staff concludes that the applicant performed its investigations in accordance with 10 CFR 100.23 and RG 1.165 and provided an adequate basis to establish that no capable tectonic sources exist in the site vicinity that would cause surface deformation in the site area. The staff concludes that the site is suitable from the perspective of tectonic surface deformation and meets the requirements of 10 CFR 100.23.

2.5.4 Stability of Subsurface Materials and Foundations

SSAR Section 2.5.4 presents information on the stability of subsurface materials and foundations at the ESP site. SSAR Section 2.5.4.2 describes the engineering properties of the subsurface materials, SSAR Section 2.5.4.3 summarizes both the previous subsurface investigations and ESP exploration program, SSAR Section 2.5.4.4 summarizes geophysical investigations performed at the site, SSAR Section 2.5.4.5 describes the extent of anticipated excavations, fills, and slopes, Section SSAR 2.5.4.6 describes the ground water conditions at the site, SSAR Section 2.5.4.7 provides the response of subsurface materials to dynamic loading, and SSAR Section 2.5.4.8 describes the liquefaction potential of the site. SSAR Sections 2.5.4.1, 2.5.4.9, and 2.5.4.11 refer to topics that the SSAR covers in greater detail elsewhere. Finally, SSAR Section 2.5.4.12 summarizes techniques that would be used to improve subsurface conditions.

2.5.4.1 Technical Information in the Application

2.5.4.1.1 Geologic Features

SSAR Section 2.5.4.1 refers to the description of regional and site geologic features in SSAR Sections 2.5.1.1 and 2.5.1.2. Section 2.5.1.3 of this SER contains the technical evaluation of this information.

2.5.4.1.2 Properties of Subsurface Materials

SSAR Section 2.5.4.2 describes the static and dynamic engineering properties of the ESP site subsurface materials. Section 2.5.4.2 also describes the subsurface materials, as well as laboratory test results and the engineering properties of the subsurface materials.

Description of Subsurface Materials

The applicant stated that it derived the properties of the subsurface materials encountered at the site from 140 subsurface borings made to date at both the NAPS and the ESP sites. The applicant divided the subsurface materials into five zones and described them as summarized below. Figures 2.5.4-1 and 2.5.4-2, reproduced from SSAR Figures 2.5-57 and 2.5-58, show two subsurface profiles (A-A' and B-B') that depict the layering of each of the soil and rock zones beneath the ESP site as well as the ESP borehole locations.

Zone IV Bedrock

Zone IV is composed of fresh to slightly weathered gneiss, which is a metamorphic rock that exhibits a banded texture (foliation) in which light and dark bands alternate. Gneiss is composed of feldspar, quartz, and one or more other minerals such as mica and hornblende. The top of the Zone IV (including Zone III-IV) bedrock at the ESP site ranges from an elevation of 188 to 298 ft.

Zone III Weathered Rock

The weathered rock has the same constituents as the parent rock. It is described as moderately to highly weathered rock, sometimes with unweathered seams and sometimes with a high fracture frequency. It is defined as having at least 50 percent core stone. The top of the Zone III bedrock at the ESP site ranges from an elevation of 205 to 298 ft.

Zone IIA and IIB Saprolites

Saprolites are a further stage of weathering beyond weathered rock. They have been produced by the disintegration and decomposition of the bedrock in place and have not been transported. Although classified as soils, saprolites contain the relict [remnant] structure of the parent rock, as well as some core stone of the parent rock. The ESP site saprolites in many instances maintain the foliation characteristics of the parent rock. They are classified primarily as silty sands, although there are also sands, clayey sands, sandy silts, clayey silts, and clays, depending on their degree of weathering. The fabric is anisotropic. The texture shows angular geometrically interlocking grains with a lack of void network, very unlike the well-pronounced voids found in marine or alluvial sands and silts.

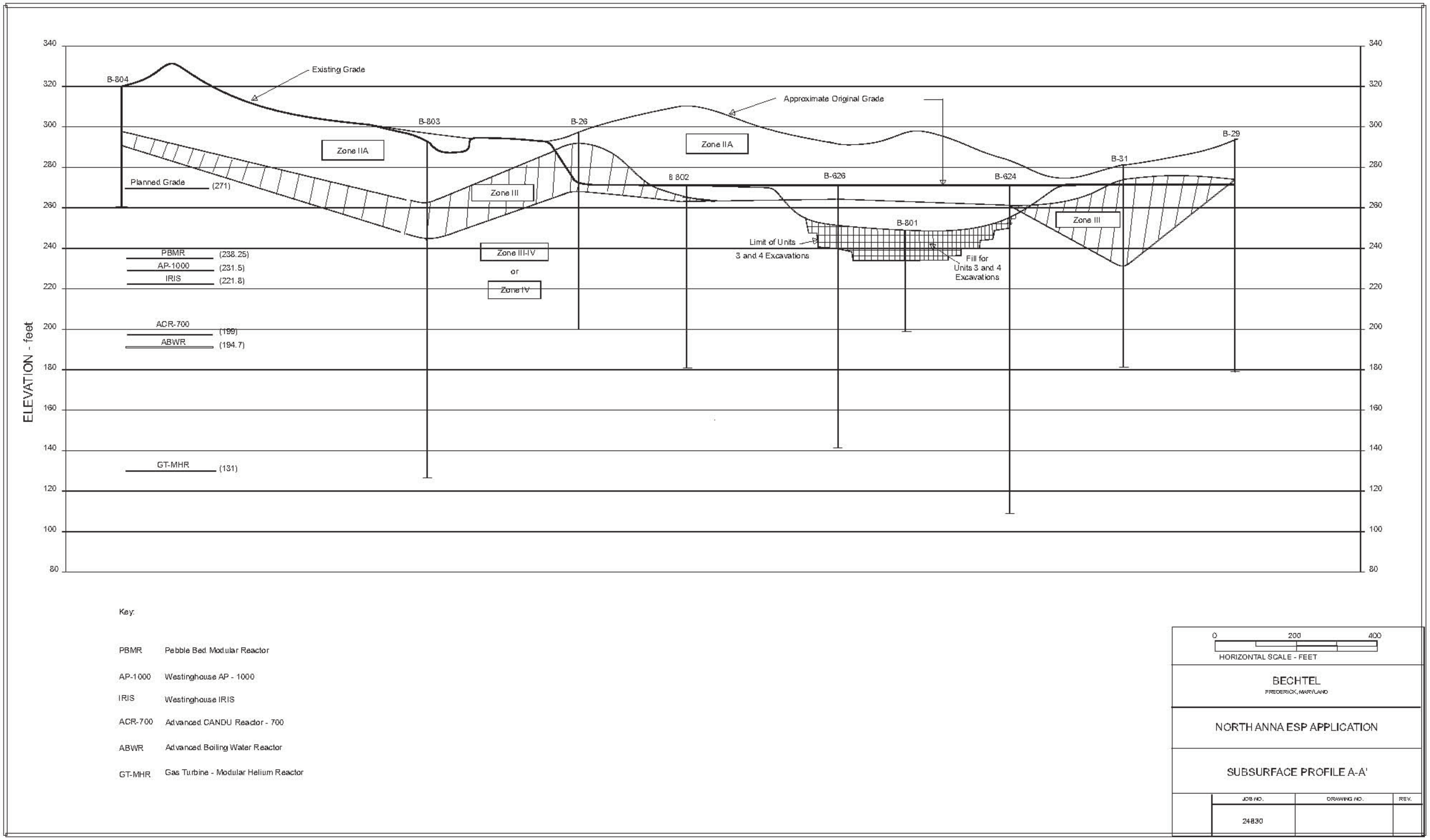

Figure 2.5.4-1 Subsurface Profile A-A'

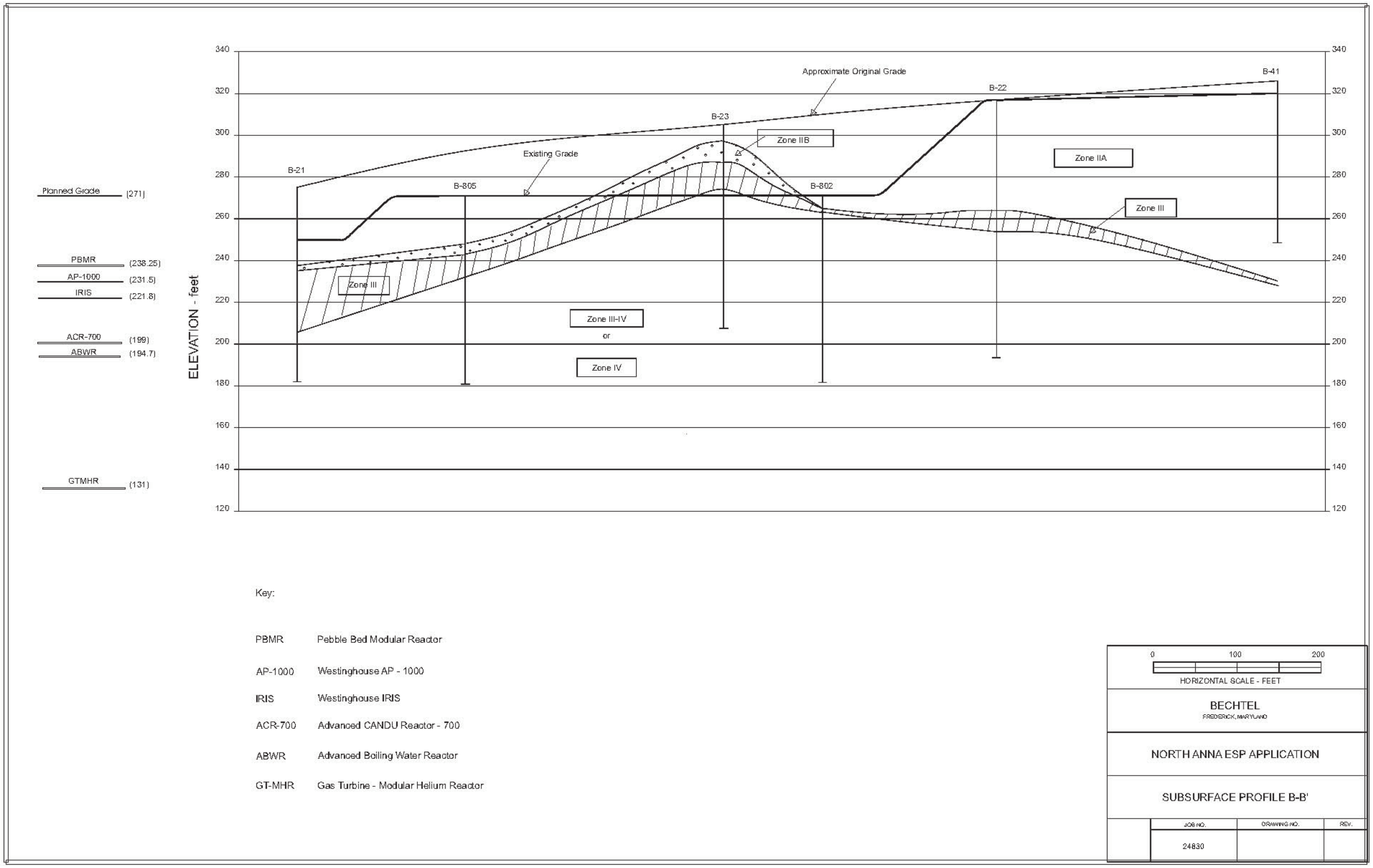

Figure 2.5.4-2 Subsurface Profile B-B'

The distribution of the Zone IIA and IIB saprolites varies throughout the site. On average, the Zone IIB saprolites represent about 20 percent of the saprolites on site and are typically very dense, silty sands with 10 to 50 percent core stone. The thickest Zone IIB deposit encountered in the borings is 37 ft. The overlying Zone IIA saprolites comprise, on average, about 80 percent of the saprolitic materials on site. About 75 percent of the Zone IIA saprolites are classified as coarse grained (sands, silty sands), while the remainder are fine grained (clayey sands, sandy and clayey silts, and clays). The saprolites typically become finer toward the ground surface. The thickest Zone IIA deposit encountered in the borings is 101 ft.

Zone I and Fill

Typically, very little Zone I residual soil exists onsite; on average, less than one percent of the soil is Zone I. The Zone I soils are either at the surface or are immediately below the fill placed during construction of the earlier units. This fill generally consists of Zone IIA soils.

Laboratory Testing

SSAR Section 2.5.4.2.4 describes the results of numerous laboratory tests of soil and rock samples performed previously, as well as the new tests performed for the ESP site investigation. The applicant performed the large majority of the tests on the Zone IIA saprolite soils for the various investigations for the SWR for the existing NAPS units; the following briefly summarizes these investigations.

Laboratory Tests for the SWR

The laboratory testing of the SWR soils focused on the strength, compressibility, and liquefaction potential of the Zone IIA saprolites. The tests include (1) cyclic triaxial tests to provide input for analysis of the liquefaction potential of the soils, (2) static triaxial shear tests including both consolidated-undrained as well as unconsolidated-undrained tests to determine shear strength parameters, (3) consolidation tests to determine the deformation behavior under various loadings, and (4) examinations of thin sections to determine the fabric, texture, and mineralogy of the saprolite. Appendix A to SSAR Section 2.5.4 presents the results of the laboratory testing of the SWR soils, which the applicant used to determine liquefaction potential, static stability, and the response of the soil to dynamic loading.

Laboratory Tests for ESP

The applicant performed laboratory testing for the ESP investigation to verify the large number of test results for previous investigations. The ESP tests focused on (1) verifying the basic properties of the Zone IIA saprolite, (2) obtaining chemical tests on the Zone IIA saprolites for corrosiveness toward buried steel and aggressiveness toward buried concrete, and (3) obtaining additional strength and elastic modulus data for the bedrock on which the main safety-related structures might be founded. Appendix B to SSAR Section 2.5.4 presents the results of the ESP laboratory tests, summarized for soil in SSAR Table 2.5-43 and for rock in SSAR Table 2.5-44. The results listed in these SSAR tables include (1) Atterberg limits (i.e., liquid, plastic, and plasticity), (2) sieve weight percentages using a #200 sieve (0.075 mm opening), and (3) soil chemistry (i.e., pH, chlorides, and sulfates). The applicant stated that the ESP laboratory test results are similar to those obtained from previous testing.

Engineering Properties

Table 2.5-45 of the SSAR presents the engineering properties of materials in subsurface Zones IIA, IIB, III, III-IV, and IV, which the applicant derived from the previous studies and from ESP field exploration and laboratory testing programs. These properties include standard geotechnical parameters such as natural moisture content, undrained shear strength, effective cohesion, effective friction angle, total unit weight, standard penetration test (SPT) blow count values, shear and compression wave velocities, elastic and shear moduli, consolidation characteristics, and static earth pressure coefficients. The following sections describe the sources and/or methods used to develop the selected properties shown in SSAR Table 2.5-45.

Rock Properties

The results given in SSAR Table 2.5-41 provide the basis for the recovery and rock quality designations (RQDs). The ESP rock strength results shown in SSAR Table 2.5-44 and the rock strengths from the investigations for the existing units form the basis for the unconfined compressive strength. The unit weight is based on the values measured in the ESP rock strength tests (SSAR Appendix 2.5.4B).

The elastic modulus values are based on the values shown in SSAR Table 2.5-44. These values agree well with those derived from the geophysical tests performed for the ESP exploration program, as described in SSAR Section 2.5.4.4.2. The shear modulus values are derived from the elastic modulus values using the Poisson's ratio values given in SSAR Table 2.5-45, which are based on the values provided in SSAR Table 2.5-44. Low- and high-strain modulus values are essentially the same for high-strength rock (i.e., for the Zone IV rock). Similarly, no strain softening is assumed for the Zone III-IV rock. The shear and compression wave velocities are based on the crosshole and downhole seismic tests performed as part of the ESP exploration program. These results, summarized in SSAR Section 2.5.4.4.2, agree with those of the geophysical tests performed for the existing units.

In RAI 2.5.4-2(a), the staff asked the applicant to describe the extent of severely weathered fracture zones in the Zone III-IV and IV rock that Virginia Power observed during the site investigation for abandoned Units 3 and 4. The applicant observed similarly fractured rock in four of the seven ESP borings. In response to RAI 2.5.4-2, the applicant provided a table that shows an RQD of less than 25 percent in nine of the borings for abandoned Units 3 and 4. The applicant noted that most of the rock thicknesses for the low RQD intervals (less than 10 percent) are only 1 to 2 ft thick. In RAI 2.5.4-2(b), the staff asked the applicant to describe the impact of these fractured rock zones on the suitability of the site to host safety-related structures. In response to RAI 2.5.4-2(b), the applicant stated the following:

> As noted in these SSAR sections, any weathered or fractured zones encountered at foundation level would be excavated and replaced with lean concrete. If such zones exist below sound rock beneath the foundation, they would have no impact on the stability of the foundation, since these zones are typically only 0.5 to 1-foot thick, and are confined within an unfractured rock mass with strengths of 4,000 to 12,000 psi (compared to the maximum foundation pressure of just over 100 psi). The foundation itself would consist of a large, thick, highly-reinforced concrete mat that is so stiff that it cannot logically yield.

> Multiple borings would be performed at each structure location once the building locations are chosen as part of detailed engineering. These borings would identify whether there are any thicker fracture zones beneath the foundation than those encountered in the ESP borings and in the abandoned Units 3 and 4 borings. If any thicker zones are found, analysis would be performed to identify their impact on foundation stability. If they are close enough to the foundation to potentially impact stability, they would be excavated and replace with lean concrete.

Soil Properties

Grain size curves from 13 sieve analyses of Zone IIA silty sand samples from the ESP laboratory testing program fit within the envelope of the 12 sieve analyses of Zone IIA silty sands sampled from borings near the SWR pump house. The natural moisture content of the fine-grained Zone IIA saprolite, determined from the moisture content tests performed on fine-grained Zone IIA saprolites for the past and the present (ESP) investigations, ranges from 14 to 56 percent.

The applicant estimated undrained shear strength of the fine-grained Zone IIA saprolite from SPT –values and cone penetrometer test (CPT) results, as well as from the results of 18 unconsolidated-undrained triaxial compression tests and 3 unconfined compression tests. The effective strength parameters for the fine-grained saprolite are based on the results of consolidated-undrained triaxial tests on fine-grained saprolite run for the previous ISFSI (Ref. 6, SSAR Section 2.5) and SWR investigations (Appendix A to SSAR Section 2.5.4).

The applicant stated that it would typically assume an effective angle of internal friction of the medium-dense coarse-grained saprolite (N=20 blows/ft) of about 35 degrees. However, the high silt content and the presence of low-plasticity clay minerals reduce this angle. Consolidated-undrained triaxial tests reported in Appendices 2C and 3E to the UFSAR for the existing units produced internal friction angles ranging from 23 to 33 degrees, with a median of 30.8 degrees. Thus, the applicant selected an angle of 30 degrees. The average effective cohesive component from the UFSAR Appendix 2C tests is 0.275 kps per square foot (ksf). The applicant selected a value of 0.25 ksf for the cohesive component.

Based on a large amount of testing performed after low unit weights were measured in the Zone IIA saprolites in the SWR area, the NAPS licensee concluded that there are isolated lower densities, but that these are not typical. Table 3.8-13 of the NAPS UFSAR identifies 125 pounds per cubic foot (pcf) as a design total unit weight. The 130 pcf shown in SSAR Table 2.5-45 for the Zone IIB saprolites reflects the high relative density of that material.

The applicant stated that the SPT design N-value of 20 blows/ft for the Zone IIA saprolite is conservatively based on the results reported in SSAR Table 2.5-40. Those results show median N-values for the ESP and ISFSI investigations of 21 blows/ft, with the median N-values for the existing units, abandoned Units 3 and 4, and SWR investigations ranging from 25 to 52 blows/ft.

The shear wave velocities measured in the ESP crosshole seismic tests in the Zone IIA sandy silt from a depth of 7.5 to 27 ft range from 650 to 1350 ft/s, with an average of 998 ft/s. The CPT seismic results are somewhat higher. The UFSAR has a value of 950 ft/s for the Zone IIA saprolite. The applicant selected a value of 950 ft/s for the Zone IIA saprolite, as shown in SSAR Table 2.5-45. For the Zone IIB saprolite, the shear wave velocity derived from the low strain value of shear modulus agrees well with the results from the CPT seismic tests, at around 1600 ft/s. Section 2.5.4.7 of the SSAR gives the profile of shear wave velocity versus depth for the saprolite.

The applicant derived the high-strain (i.e., in the range of 0.25 to 0.5 percent) elastic modulus values for the coarse-grained Zone IIA saprolite and the Zone IIB saprolite using the relationship with the SPT –value given in the literature (Ref. 151, SSAR Section 2.5). In addition, the applicant derived the high-strain elastic modulus for the fine-grained Zone IIA saprolite using the relationship with undrained shear strength (also given in SSAR Ref. 151). The applicant stated that it slightly adjusted the Zone IIA coarse- and fine-grained values to obtain a common value. The applicant obtained the shear modulus (G) values from the elastic modulus values using the relationship between elastic modulus (K), shear modulus, and Poisson's ratio (ν).

$$G = \frac{3}{2} K \frac{(1 - 2\nu)}{(1 + \nu)}$$

The applicant derived the low-strain (i.e., 10^{-4} percent) shear modulus for the Zone IIA saprolite from the shear wave velocity of 950 ft/s. Similarly, the applicant derived the low-strain shear modulus (G_{max}) of the Zone IIB saprolite from the shear wave velocity of 1600 ft/s. The applicant obtained the elastic modulus values for the Zone IIB saprolite from the shear modulus values using the relationship between elastic modulus, shear modulus, and Poisson's ratio (Ref. 150, SSAR Section 2.5).

The values derived from the settlement studies performed for the SWR pump house, as detailed in Appendix 3E to the UFSAR, include the recompression ratio (total amount of settlement) and the coefficient of secondary compression (after primary consolidation). The values of unit coefficient of subgrade reaction are based on values for medium-dense sand (Zone IIA saprolite) and very dense sand (Zone IIB saprolite) provided by Terzaghi (Ref. 152, SSAR Section 2.5). The earth pressure coefficients (ratio of lateral load to vertical load) are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall.

In RAI 2.5.4-4, the staff asked the applicant to explain how the total thickness of the soil layers sampled at the ESP site (105 ft) is sufficient to characterize the soil properties underlying the site. The applicant responded that the 138 borings previously performed by Virginia Power for Units 1 and 2 as well as the abandoned Units 3 and 4 characterize the soils at the North Anna site very well. The applicant stated that the soils in all of borings show the same general subsurface profile and that it used the ESP borings to show that the soil (and rock) profiles in each of the borings fit within the general subsurface profile.

Chemical Properties

The applicant performed chemical tests on selected Zone IIA samples. In addition to the tests performed for the ESP site investigation (see the results shown in SSAR Table 2.5-43), Virginia Power previously performed chemical tests on two samples from the subsurface investigation for the existing units. The six pH test results range from 5.7 to 6.9, in the mildly corrosive to neutral range. The six sulfate test results range from about 1 to 28 parts per million, indicating no aggressiveness toward concrete. Three of the chloride test results range from 100 to 170 milligrams per kilogram (mg/kg), indicating little corrosive potential toward buried steel. The fourth chloride test produced 920 mg/kg, indicating potential corrosiveness toward buried steel.

2.5.4.1.3 Exploration

SSAR Section 2.5.4.3 describes the previous subsurface investigations performed at the NAPS site as well as the ESP exploration program.

Previous Subsurface Investigation Programs

For the existing Units 1 and 2, the NAPS licensee performed 60 borings in 1968, with boring depths ranging from 20 to 150 ft. For the abandoned Units 3 and 4, Virginia Power performed 47 borings in 1971, with boring depths ranging from 40 to 175 ft. Virginia Power performed an additional 22 borings in the SWR area after 1976, as well as 9 borings in 1994 for the ISFSI. The borings used SPT sampling, Dames and Moore soil samplers, and NX-size double-tube core barrels for rock coring. SSAR Tables 2.5-30 through 2.5-37 summarize the boring locations, the elevations for each of the subsurface zones, and RQDs. Figure 2.5.4-3, reproduced from SSAR Figure 2.5-59, shows the locations of the previous borings.

In RAI 2.5.4-3, the staff asked the applicant to describe how it integrated the NAPS licensee's site investigations for the SWR and the ISFSI with its field investigations for the ESP site. The applicant responded that the SWR and ISFSI borings are as close to the ESP area as any other borings and disclosed the same subsurface profile displayed by the other borings at the North Anna site (see SER Figure 2.5.4-3). In addition, the applicant stated that it used some of the SWR and ISFSI borings, which are close to the southeast corner of the ESP footprint, noted in RAI 2.5.4-1, to help characterize the ESP area.

ESP Subsurface Investigation Program

The applicant stated that it performed the ESP subsurface investigation in 2002, covering the area proposed for the new units and the cooling towers for the new units. This investigation consisted of relatively few exploration points, compared to previous field explorations for the existing units, abandoned units, SWR, and ISFSI. According to the applicant, it designed the ESP field explorations primarily to confirm the results obtained from the previous extensive investigations. The applicant stated that it would perform additional structure-specific exploration and testing during detailed engineering, and a COL application would describe this testing. Figure 2.5.4-4, reproduced from SSAR Figure 2.5-60, shows the ESP exploration point locations.

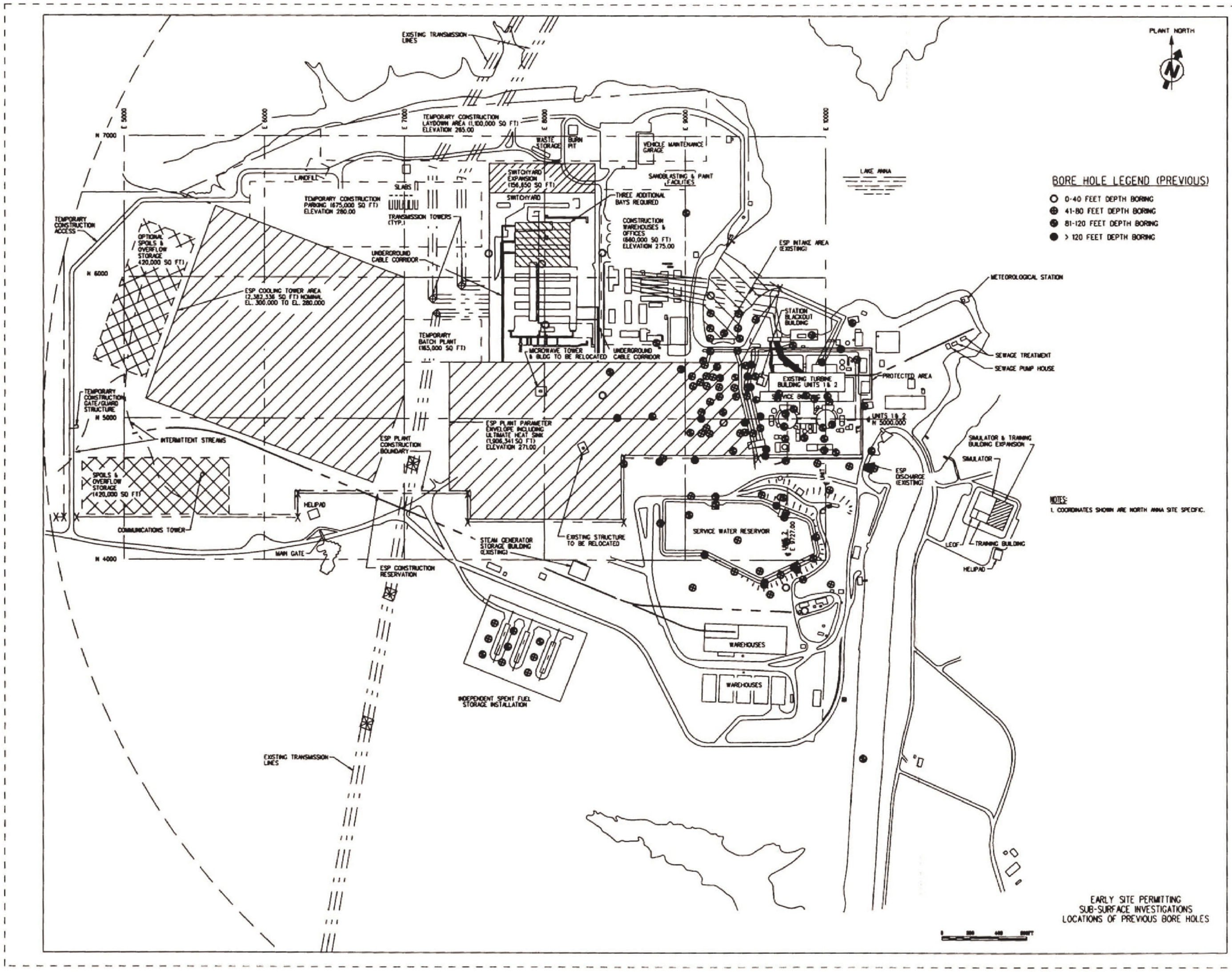

Figure 2.5.4-3 Locations of previous boreholes

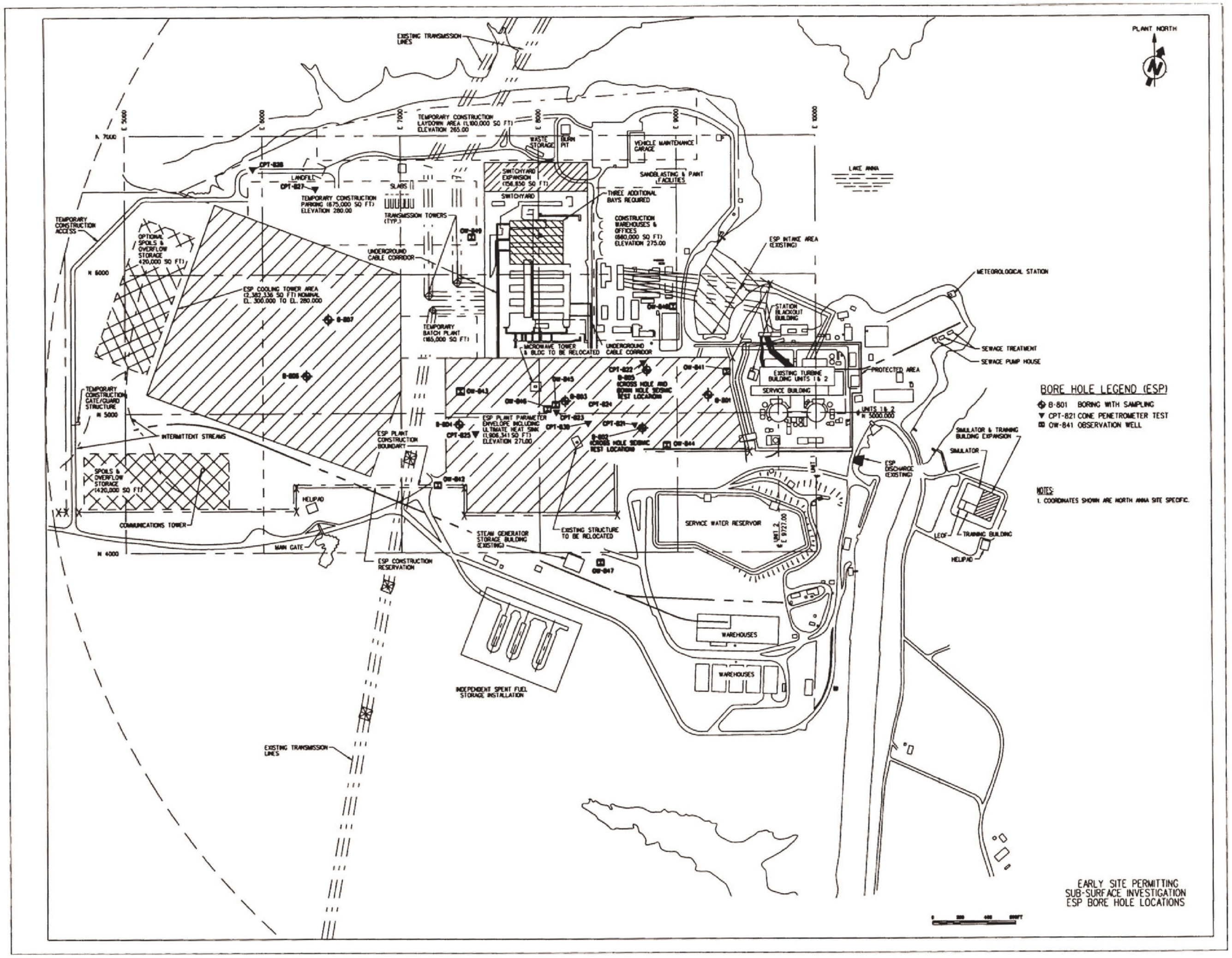

Figure 2.5.4-4 ESP borehole locations

The scope of work related to the ESP site investigation consisted of the following:

- seven exploratory borings
- nine observation wells
- eight CPTs
- two downhole seismic cone tests
- two pore pressure dissipation tests
- two sets of crosshole seismic tests
- one downhole seismic test
- a survey of all exploration points
- laboratory testing of borehole samples and cores

Appendix B to SSAR Section 2.5.4 provides details and results of the exploration program. The following summarizes the borings, observation wells (OWs), and CPTs.

Borings and Samples/Cores

According to the applicant, the seven borings drilled range from 50 to 170 ft in depth, averaging 85 ft. The 170-ft deep boring is 30 ft deeper than the deepest reactor design considered for the ESP. The applicant stated that it conducted the SPT in general accordance with American Society for Testing and Materials (ASTM) D1586 and performed rock coring in general accordance with ASTM D2113. The applicant stated that, after removal from the SPT split inner barrel, it carefully placed the recovered rock in wooden core boxes. The onsite geologist visually described the core, noting the presence of joints and fractures and distinguishing natural breaks from mechanical breaks. The geologist also computed the percentage recovery and the RQD. Appendix B to SSAR Section 2.5.4 provides the boring logs and the photographs of the rock cores. These boring logs describe in detail the soil and rock materials encountered at different depths of the borings and also contain a record of the ground water level, the SPT blow counts, and the elevation of the top of the rock surface. The applicant used these data for the liquefaction analyses, bearing capacity calculations, and settlement analyses. The applicant stated that the soil and rock materials encountered in the ESP borings are similar to those found in the previous sets of borings conducted at the NAPS site.

In RAI 2.5.4-1, the staff asked the applicant to provide its basis for concluding that the subsurface conditions in the southeast portion of the ESP footprint (an area of about 500 ft by 1000 ft, in which there are no borings) do not materially differ from conditions in adjacent areas where borings were made. In its response, the applicant stated that the North Anna site is underlain by a consistent geologic profile, which extends to a depth of several thousand feet. The 145 borings performed throughout the North Anna site (including 7 for the ESP) indicate a consistent overall subsurface profile, with expected variations in the thickness of the various strata. As such, the applicant concluded that the southeast portion of the ESP footprint (see SER Figure 2.5.4-3) should be similar to the rest of the site.

In RAI 2.5.4-6, the staff asked the applicant to explain why it did not provide laboratory test results from the borings of subsurface materials over various depth intervals. The applicant responded that the containment (reactor) buildings for the new units would be founded on the Zone III-IV and/or Zone IV metamorphic gneiss bedrock at the North Anna site. Rock coring and testing performed by Virginia Power for Units 1 and 2 gave unconfined compressive strengths for the Zone III-IV and IV rock ranging from 1,000 to 16,300 psi with a median strength of 6,800 psi. The applicant stated that these rock strengths are typical for this type of rock and more than sufficient to support the maximum containment (reactor) building loads of about 100 psi. The applicant added that, during logging of the rock cores in the field for the ESP investigation, it was apparent that the metamorphic rock is a strong material. The applicant stated that it performed sufficient tests on the ESP cores to verify that the rock strengths are similar to or higher than those cores tested for Units 1 and 2. The applicant determined that the median value of the unconfined compressive strengths of the Zone III-IV and IV rock from the ESP investigation is 18,400 psi.

Observation Wells

The applicant screened eight OWs with depths ranging from about 25 to 50 ft in soil and/or weathered rock. The applicant advanced boreholes for these wells with hollow stem augers. The applicant obtained samples at 5-ft intervals to provide information on an appropriate depth to set the slotted screen. The applicant screened the ninth well in rock. Each well was developed by pumping. The applicant considered the well developed when the pH and conductivity stabilized and the pumped water was reasonably free of suspended sediment. The applicant then performed permeability tests in each well in general accordance with ASTM D4044, Section 8, using the slug test method. Appendix B to SSAR Section 2.5.4 contains the details of the boring logs for the OWs, the well installation records, the well development records, and the well permeability test results. The boring logs of the OWs also describe the soil and rock seen in these borings. The applicant stated that it would use the ground water level data, as recorded in the OWs, in developing the dewatering program at the time of construction.

Cone Penetrometer Tests

The applicant stated that it advanced each of the CPTs to refusal (i.e., no further penetration), to depths ranging from 4 to 58 ft. The applicant stated that it performed the piezocone tests in general accordance with ASTM D5778. The pore pressure filter was located immediately behind the cone tip. The applicant performed pore pressure dissipation tests at a depth of 27 ft in CPT-823 and at a depth of 32.5 ft in CPT-827. Appendix B to SSAR Section 2.5.4 contains the CPT logs, shear wave arrival times, and pore pressure versus time plots, while SSAR Tables 2.5-38 and 2.5-39 summarize the CPT locations and depths.

2.5.4.1.4 Geophysical Surveys

<u>Previous Geophysical Survey Programs</u>

The NAPS licensee performed several geophysical studies for the investigation for the existing Units 1 and 2, including a seismic refraction survey in 1968. The seismic (compressional wave) velocities measured by Virginia Power in the relatively unweathered rock (Zone IV) range from 13,000 to 16,000 ft/s. Compressional wave velocities measured in weathered rock are around 5000 ft/s. Shear wave velocities in the Zone IV rock range from about 4000 to 8000 ft/s. The corresponding compressional wave velocities are about 8,000 to 16,000 ft/s. Unit weights range from about 140 to 170 pcf. Weston Geophysical performed seismic crosshole tests between the Unit 1 and 2 reactors and obtained shear wave velocities in the Zone IV rock between 5000 and 6000 ft/s. The UFSAR for the existing units provides a shear wave velocity for the saprolite (Zone IIA) of 950 ft/s.

Geophysical Surveys for ESP

For the ESP site geophysical investigation, the applicant performed two crosshole seismic tests, one downhole seismic test in a borehole, and two downhole seismic tests using a cone penetrometer.

Crosshole Seismic Tests

The applicant performed crosshole seismic tests immediately adjacent to borings B-802 and B-805. The applicant stated that it performed these tests in accordance with ASTM D 4428/D 4428M. The applicant used the B-802 location to obtain readings in rock, while it used the B-805 location to obtain readings in soil. The applicant performed tests in boring B-802 at 5-ft intervals in the rock at depths ranging from 27 to 90 ft; however, it only obtained shear wave velocity results at depths ranging from 27 to 45 ft. The applicant stated that severe high-frequency noise appears to have degraded the results in general, but particularly below a depth of 45 ft. The high-frequency noise obscured all of the compressional wave forms. The shear wave velocities in the rock at depths between 27 and 45 ft range from 4500 to 6000 ft/s. The applicant performed tests in borings B-805A, B, and C at 2.5- to 5-ft intervals in the soil from near the surface to a depth of 27 ft. The seismic waveforms were reasonably clear, except for the bottom interval, close to the rock interface. The shear wave velocities range from about 610 to 1380 ft/s, the compressional wave velocities range from about 1240 to 6550 ft/s, and the computed dynamic Poisson's ratios range from 0.27 to 0.49.

Downhole Seismic Tests

Since the crosshole tests in borings B-802A, B, and C yielded no compressional wave results and gave no shear wave velocity results below a depth of 45 ft, the applicant conducted downhole seismic testing in boring B-802B. Appendix B to SSAR Section 2.5.4 contains a detailed description of the results. The applicant stated that the shear wave was reasonably well defined to a depth of 45 ft, less defined from a depth of 45 to 65 ft, and not defined below a depth of 65 ft. Between 22.5 and 65 ft, shear wave velocities range from about 3400 ft/s to 6380 ft/s. Between 22.5 and 87 ft, compressional wave velocities range from about 10,000 ft/s to 16,600 ft/s. The computed dynamic Poisson's ratios range from 0.38 to 0.45.

Downhole Seismic Tests with Cone Penetrometer

The applicant performed downhole seismic tests at 5-ft intervals in CPT-822 and CPT-825. It recorded shear waves with a geophone attached near the bottom of the cone string. Appendix B to SSAR Section 2.5.4 plots shear wave arrival times versus depth. In CPT-822, the computed shear wave velocity between depths of 10 and 22 ft was about 1275 ft/s. In CPT-825, the computed shear wave velocity between depths of 6 and 30 ft was 1175 ft/s. For greater depths, between 30 and 45 ft, the computed shear wave velocity was about 1660 ft/s, and between 45 and 52 ft, it was about 2438 ft/s.

In RAI 2.5.4-5, the staff asked the applicant to explain why SSAR Table 2.5-45 does not give shear wave velocities for Zone IIB saprolite and Zone III and III-IV weathered rock. In its response, the applicant stated that SSAR Table 2.5-45 gives average shear wave velocities for Zones IIB, III, and III-IV but does not provide a range of values. In contrast, it provides both average values and a range of shear wave velocity values for Zones IIA and IV. According to the applicant, it originally provided only average values for Zones IIB, III, and III-IV because the ESP borings did not sample these zones as abundantly as Zones IIA and IV. In response to this RAI, the applicant provided its method for determining the average shear wave velocity values for Zones IIB (1600 ft/s), III (2000 ft/s), and III-IV (3300 ft/s). In addition, the applicant used its laboratory measurements of the soil/rock properties for Zones IIB, III, and III-IV to indirectly determine the shear wave velocities. Accordingly, the applicant updated SSAR Table 2.5-45 to include the range in shear wave velocity for these three soil/rock zones.

2.5.4.1.5 Excavation and Backfill

SSAR Section 2.5.4.5 describes the extent of anticipated safety-related excavations, fills, and slope; excavation methods and stability; backfill sources and quality control; and construction dewatering impacts. The applicant stated that the construction of the proposed new units would involve a substantial amount of excavation in both soil and rock. Filling would consist almost entirely of backfilling around structures back up to plant grade. The only new permanent slope that may be created would be to the west of the SWR to accommodate the buried UHSs, if warranted by the selected design for the proposed additional units. The applicant stated that the top of the slope would be at least 200 ft from the top of the SWR embankment and, therefore, would not impact the SWR. Next, the applicant described excavation methods that it would use in soil and rock (i.e., blasting techniques and alternatives to blasting), backfill sources, and quality control. The applicant stated that structural fill would be either lean concrete or a sound, well-graded granular material. In addition, it would establish an onsite soils testing laboratory to control the quality of the fill materials and the degree of compaction. To control soil erosion, the applicant stated that it would line any sumps and ditches constructed for dewatering and slope the tops of excavations back to prevent runoff down the excavated slopes during heavy rainfall.

2.5.4.1.6 Ground Water Conditions

In SSAR Section 2.5.4.6, the applicant briefly described the ground water conditions at the ESP site and general plans for construction dewatering. Section 2.4.12 of the SSAR describes the ground water conditions at the ESP site in detail. The following summarizes the applicant's description of the ESP site ground water conditions in SSAR Section 2.5.4.6.

Nine OWs installed at the site as part of the ESP subsurface investigation program have exhibited ground water levels ranging from MSL elevations of 241 to 311 ft between December 2002 and June 2003. Based on the results of the slug tests in the wells, hydraulic conductivity values for the saprolite in which eight of the wells were screened range from 0.2 to 3.4 ft/day. The applicant estimated the hydraulic conductivity of the shallow bedrock in which one of the wells was screened to be about 2 to 3 ft/day. Ground water movement at the site is generally to the north and east, toward Lake Anna.

The applicant stated that ground water is present in unconfined conditions in both the surficial sediments and underlying bedrock at the ESP site. The ground water generally occurs at depths ranging from about 6 to 58 ft below the present-day ground surface. The design ground water level for the new units would range from 265 to 270 ft MSL in elevation. Section 2.4.12 of the SSAR derives this level.

The applicant stated that it can achieve dewatering for all major excavations using gravity-type systems. For soils, because of their relatively impermeable nature, sump-pumping of ditches would be adequate to dewater the soil. For rock, the applicant would use sump-pumping to collect water from relief drains that would be installed in the major rock excavation walls to prevent hydrostatic pressure buildup behind the walls.

2.5.4.1.7 Response of Soil and Rock to Dynamic Loading

In SSAR Section 2.5.4.7, the applicant estimated the seismic ground motion amplification/attenuation using the shear wave velocity profiles for the different subsurface materials, the variation of shear modulus and damping with strain, and the site-specific acceleration time histories. The applicant stated that the reactor containment buildings for the proposed additional units would be founded on Zone III-IV or Zone IV bedrock. However, other safety-related structures may be founded on the Zone III weathered bedrock, the Zone IIB very dense saprolitic sand, and/or the Zone IIA saprolitic sand.

Shear Wave Velocity Profile

The applicant made various measurements, summarized in SSAR Section 2.5.4.4, at the ESP site to obtain estimates of the shear wave velocity in the soil and rock. The applicant considered the Zone IV bedrock to be the base rock at a depth of 70 ft in the amplification/attenuation analysis. Table 2.5-45 of the SSAR shows an average shear wave velocity of 6300 ft/s for Zone IV. While in some locations the top of Zone III-IV or Zone IV bedrock is found close to or even above the planned plant grade, sound bedrock is relatively deep in other locations. The applicant stated that, in the case of relatively deep bedrock, some safety-related structures (excluding the reactors) may be founded on the Zone III weathered rock, Zone IIB saprolite, or Zone IIA saprolite. SSAR Figure 2.5-62, Profile (a), focuses on this situation; it shows the shear wave velocity values measured in Zone IIA saprolite for the ESP subsurface exploration program using crosshole and CPT downhole seismic testing. SSAR Figure 2.5-62 (reproduced previously as SER Figure 2.5.2-5) also shows the shear wave velocity of 950 ft/s given in the UFSAR of the existing units for the saprolite. The applicant took this as the average design value for the Zone IIA saprolite for the ESP evaluation. The design shear wave velocity versus depth profile shown in SSAR Figure 2.5-62, Profile (a), is anchored about the design value of 950 ft/s for the Zone IIA saprolite but reflects the expected increasing values with depth demonstrated in the crosshole and downhole seismic tests.

The applicant stated, as noted in SSAR Section 2.5.4.10.2, that it would improve any Zone IIA saprolites supporting safety-related structures to reduce potential settlement. In RAI 2.5.4-7, the staff asked the applicant to reconcile two conflicting statements in SSAR Sections 2.5.4.7.1 and 2.5.1.2.6. The applicant stated in SSAR Section 2.5.1.2.6 that Zone III (weathered rock) is not a suitable material for safety-related plant structures. However, the applicant stated in SSAR Section 2.5.4.7.1 that some safety-related structures (excluding the reactor containment building) may be founded on the Zone III weathered rock, Zone IIB saprolite, or improved Zone IIA saprolite. In response to RAI 2.5.4-7, the applicant noted that the statement in SSAR Section 2.5.4.7.1 is correct, and therefore it will delete the statement in SSAR Section 2.5.1.2.6. The applicant emphasized that only improved Zone IIA saprolite is appropriate for certain safety-related structures (see RAI 2.5.4-11 below). To compute the response of the improved Zone IIA saprolite to dynamic loading, the applicant computed the shear wave velocity through the improved soil based on this increase in stiffness. Profile (b) of SSAR Figure 2.5-62 shows these computed shear wave velocities and the unimproved Zone IIA shear wave velocities. This profile also shows the shear wave velocity values interpreted in SSAR Appendix 2.5.4B from the CPT-825 downhole seismic tests at a depth of 52 ft during the ESP subsurface exploration program. The applicant interpreted the subsurface materials below a depth of 30 ft in the CPT log as a silty sand and sandy silt mix. These could be either Zone IIB saprolitic sands or Zone III weathered rock (or both). From depths between 30 and 40 ft, the design profile uses the shear wave velocity for the Zone IIB saprolite from SSAR Table 2.5-45 (1600 ft/s), which is very close to the 1650 ft/s measured in the CPT-825 downhole seismic test. From depths of 40 to 55 ft, the design profile uses the shear wave velocity for the Zone III weathered rock from SSAR Table 2.5-45 (2000 ft/s). This is close to the mean of the two CPT-825 downhole seismic velocities measured in this zone, as shown in SSAR Figure 2.5-62, Profile (b). The applicant assumed Zone III-IV to extend from depths of 55 to 70 ft. Shear wave velocity for this rock is 3300 ft/s, derived from several values measured in the downhole seismic test performed adjacent to boring B-802 and from elastic modulus values from unconfined compression tests (SSAR Section 2.5.4.2.5).

<u>Variation of Shear Modulus and Damping with Strain</u>

Figure 2.5.4-5, reproduced from SSAR Figure 2.5-63, shows normalized shear modulus reduction curves, which are taken from research reports referenced in SSAR Section 2.5.4.

Curve 1 in this figure represents the Zone IIA saprolite (both unimproved and improved). This modulus reduction curve is the average of (1) the 1970 Seed and Idriss (Ref. 167, SSAR Section 2.5) average curve for sand and (2) five curves (from a 1993 EPRI report (Ref. 170, SSAR Section 2.5)) that take into account several factors, including reference strain and effective vertical stress. One of the five EPRI curves is a low-plasticity clay curve to account for the cohesive component of the Zone IIA saprolite. Curve 2 in SSAR Figure 2.5-63 represents the Zone IIB saprolite and is the modulus reduction curve recommended by Seed, et al. (Ref. 168, SSAR Section 2.5) for gravels, based on tests of four different gravels and crushed stone samples. The Zone IIB saprolite contains the relict structure of the parent rock. Since this contains up to 50 percent of core rock remaining in the saprolite, the applicant stated that it would behave more like a gravel or crushed stone than a sand.

The applicant stated that solid rock does not exhibit the strain-softening characteristics of soil. Thus, the Zone III-IV rock has no modulus reduction curve. However, at some stage of weathering, rock becomes sufficiently decomposed to exhibit modulus reduction. The applicant considered Zone III moderately to severely weathered rock as falling into this sufficiently weathered state. Curve 3 in SSAR Figure 2.5-63 was developed for mudstone (a soft rock) with a shear wave velocity of 1500 ft/s (Ref. 169, SSAR Section 2.5). SSAR Section 2.5.4.7.1 shows that Zone III has a shear wave velocity of 2000 ft/s. The applicant stated that when mudstone Curve 3 is used for shear modulus input in the soil/rock column amplification/

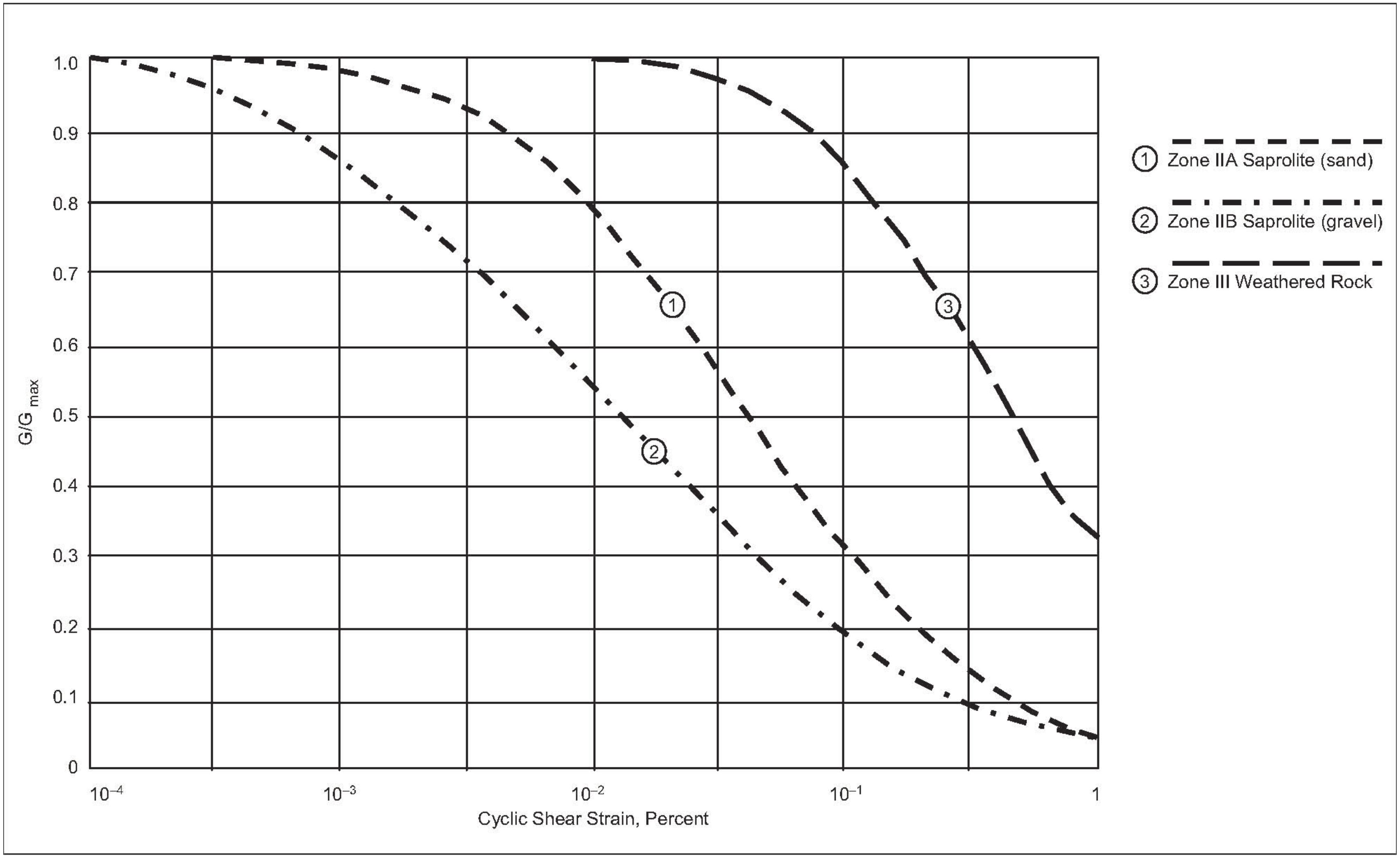

Figure 2.5.4-5 Variation of normalized shear modulus with cycle shear strain

attenuation analysis for the Zone III weathered rock, the shear modulus attenuation is significantly less than that exhibited by the sand and gravel curves.

In SSAR Section 2.5.4.7.1, the applicant stated the following:

> When the specific locations of safety-related structures are determined, if structures such as the diesel generator building and/or certain tanks are founded on saprolite or weathered rock, samples of foundation soils from those locations would be tested to determine location-specific shear modulus degradation relationships.

Figure 2.5.4-6, reproduced from SSAR Figure 2.5-64, plots the variation of the equivalent damping ratio of saprolite and weathered rock as a function of cyclic shear strain.

Curve I in SSAR Figure 2.5-64 represents the Zone IIA saprolite (both unimproved and improved). The applicant stated that this damping ratio versus cyclic shear strain curve is the average of (1) the Seed and Idriss (Ref. 167, SSAR Section 2.5) average curve for sand and (2) seven curves from Reference 170 that take into account several factors including reference strain and effective vertical stress. One of these seven curves is a low-plasticity clay curve to account for the cohesive component of the Zone IIA saprolite. Curve II in SSAR Figure 2.5-64 represents the Zone IIB saprolite. The applicant used the Seed et al. (Ref. 168, SSAR Section 2.5) curve for gravels. Curve III in SSAR Figure 2.5-64 represents the Zone III weathered rock. The applicant stated that it derived this curve by comparing Curve 3 in SSAR Figure 2.5-63 with Curves 1 and 2 in SSAR Figure 2.5-63 and applying the differences proportionally to SSAR Figure 2.5-64. The applicant stated that the damping ratio of the Zone III-IV rock does not vary with cyclic shear strain. However, since this rock has some intrinsic damping properties, the applicant used a damping ratio of 2 percent.

In RAI 2.5.4-8, the staff asked the applicant to provide its basis for the selected modulus reduction and damping ratio curves for Zones IIA, IIB, and III. In its response, the applicant stated that it used the 1993 EPRI report (Ref. 170, SSAR Section 2.5), where applicable, as the basis for the shear modulus reduction and damping ratio curves.

In RAI 2.5.4-8(c), the staff asked the applicant to explain its use of a damping ratio of 2 percent for the Zone III-IV rock. In its response, the applicant stated that the damping ratio for rock varies from site to site depending on the various factors, including the mineral composition of the rock, the integrity and fissuring of the rock mass, and the level of shear deformation in the rock formation. According to the applicant, damping ratios for rock are generally between 0.5 to 4.5 percent. The applicant selected 2 percent for the Zone III-IV rock based on engineering judgment and past experience. To determine the sensitivity of the selected damping ratio, the applicant reran its analysis using damping ratios of 0.5, 1.0, and 5.0 percent. The results show only a slight difference in the peak acceleration for the different damping ratios.

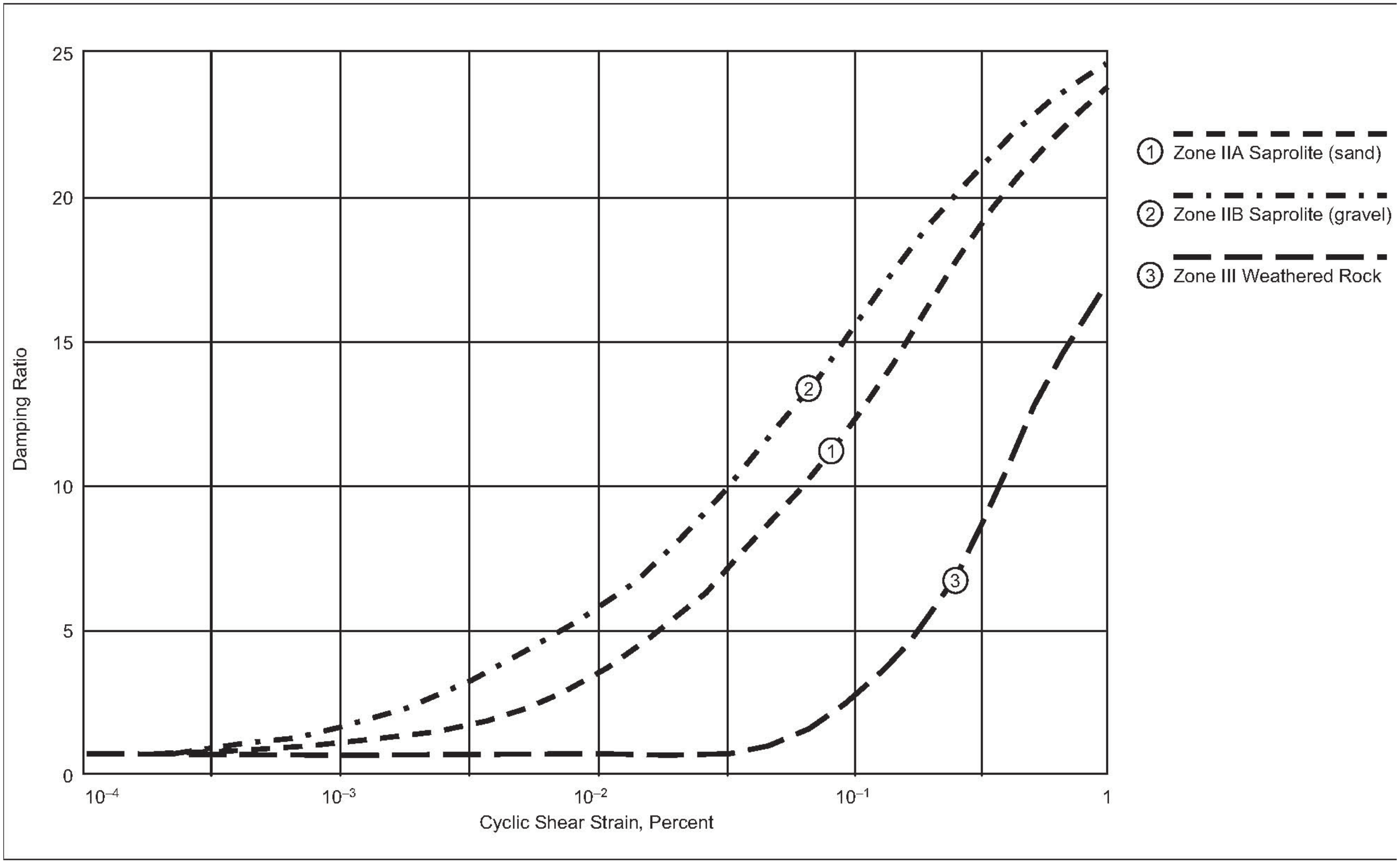

Figure 2.5.4-6 Variation of damping ratio with cyclic shear strain

Site-Specific Acceleration-Time Histories

The applicant developed two single horizontal-component acceleration time histories, which are compatible with the low- and high-frequency response spectra developed from the two controlling earthquakes and PSHA hazard curves. The applicant used these two acceleration time histories in the soil column amplification analysis described below.

In RAI 2.5.4-9(a), the staff asked the applicant to describe its method for developing the site-specific acceleration time histories. In its response, the applicant stated that it selected two horizontal-component acceleration time histories which it then matched to the low- and high-frequency response spectra from the two controlling earthquakes. The applicant then used these spectrum-compatible time histories for the site response analysis. In RAI 2.5.4-9(b), the staff asked the applicant to further describe the method it used for the development of the soil column amplification/attenuation analysis. In its response, the applicant stated that it used the SHAKE2000 computer program to compute the site dynamic responses for the four soil and rock profiles described in SSAR Section 2.5.4.7.1. The applicant provided the input soil parameters, the depth at which the hard rock ground motion was input (70 ft), and information on the number of iterations to compute the strain-compatible modulus and damping values for the SHAKE analysis. In RAIs 2.5.4-9(c) and (d), the staff asked the applicant to further describe the four soil profiles and how the analysis accounted for the variability of the soil properties. In response to RAIs 2.5.4-9(c) and (d), the applicant provided the soil properties for each of the four profiles and described the values that were varied in the analysis. The applicant stated that the shear wave velocity (V_s) and G_{max}, which is derived from V_s, have the most impact on the amplification/attenuation analysis. The applicant showed response spectra for different levels of G_{max} (67 to 150 percent). In RAI 2.5.4-9(e), the staff asked the applicant to justify its use of the mean 10^{-4} uniform hazard spectrum as the input rock motion. In response to RAI 2.5.4-9(e), the applicant stated that it initially used a time history matched to the mean 10^{-4} uniform hazard spectrum; however, it later revised this approach to use time histories that match the low- and high-frequency response spectra calculated from the two controlling earthquakes.

Soil Column Amplification/Attenuation Analysis

The applicant used the SHAKE2000 computer program to compute the site dynamic responses for the soil and rock profiles, described in SSAR Section 2.5.4.7.1. The analysis, performed in the frequency domain, used the two acceleration time histories briefly described in the previous section and in SSAR Section 2.5.2. The analysis used (1) the low-frequency controlling earthquake time history with a peak acceleration of 0.21g and (2) the high-frequency controlling earthquake time history with a peak acceleration of 0.43g.

Table 2.5-46 of the SSAR shows the zero period acceleration (ZPA) results for the SHAKE2000 analysis for the four soil profiles, given in SSAR Section 2.5.4.7.1. The ZPA results for soil Profile 1, with 30 ft of unimproved Zone IIA saprolite, are 0.91g for the high-frequency case and 0.46g for the low-frequency case. The applicant also determined the ZPA results for the four soil profiles using a G_{max} value that was 150 percent of the average G_{max} value. Using these higher G_{max} values, the applicant obtained ZPA values of 0.99g and 0.57g for the high-and low-frequency cases, respectively. As described in SSAR Section 2.5.4.8 and below, the applicant applied these amplified accelerations in the liquefaction evaluation of soils.

2.5.4.1.8 Liquefaction Potential

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength because of pore pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction can occur, leading to foundation bearing failures and excessive settlements, when (1) the ground acceleration is high, (2) soil is saturated (i.e., close to or below the water table), and (3) the site soils are sands or silty sands in a loose or medium-dense condition. The applicant stated that the ESP site meets the first criterion, and the second criterion applies in many areas of the NAPS site; however, the third criterion, involving the type and density of the soil, is much less clearly applicable. According to the applicant, the Zone IIB soils are extremely dense. The Zone III weathered rock has over 50 percent core stone and has typically been sampled by rock coring. As such, neither of these materials meets the loose or medium-dense criterion, and neither has liquefaction potential. The applicant stated that any needed structural fill would be a well-compacted, well-graded crushed stone that is not liquefiable. Reasoning that neither the Zone IIB soils nor the Zone III weathered rock are susceptible to liquefaction, the applicant only discussed the liquefaction potential of the Zone IIA saprolitic soil.

The applicant stated that there is no historical evidence that Zone IIA saprolitic soils have undergone liquefaction at the ESP site. Attachment 4 to Appendix 3E to the UFSAR indicates that examination of the structure and fabric of the material "leads to the conclusion that the saprolite is not susceptible to liquefaction." Despite its apparent low potential for liquefaction, the Zone IIA saprolite at the NAPS site has been the subject of several previous liquefaction analyses. SSAR Section 2.5.4.8.2 examines these analyses in view of the accelerations assumed for the ESP. In addition, the applicant performed a liquefaction analysis, summarized below, on potentially liquefiable samples obtained from the recent ESP exploration program.

<u>Effect of Soil Structure and Fabric on Liquefaction Potential</u>

SSAR Section 2.5.4.8 describes the soil structure and fabric of the saprolite. The applicant stated that the fabric of the saprolite is similar to that of its parent rock, a biotitic [mineral in mica group] quartz gneiss. According to the applicant, there is a strong foliation in the saprolite and the fabric is strongly anisotropic. The applicant contrasted the highly foliated and anisotropic fabric of the saprolite with that of an alluvial- or marine-deposited sand. The applicant stated that sand shows no foliation and no interlocking of grains. In addition, a thin section of sand shows a well-developed void network unlike that of saprolite. The applicant concluded by stating that the geometric interlocking of the grains and the lack of a void network indicates that the saprolite could not liquefy. Despite this conclusion, the applicant analyzed the potential of the saprolite to liquefy under both the high-frequency and low-frequency input bedrock motions.

<u>Acceptable Factor of Safety Against Liquefaction</u>

According to RG 1.198 (Ref. 172, SSAR Section 2.5), a factor of safety (FS) of 1.1 against liquefaction is considered low, FSs of 1.1 to 1.4 are considered moderate, and an FS of 1.4 is considered high. The Committee on Earthquake Engineering (Ref. 173, SSAR Section 2.5) states that there is no general agreement on the appropriate margin (factor) of safety. If the design earthquake ground motion is regarded as reasonable, an FS of 1.33 to 1.35 is suggested as adequate. However, when the design ground motion is excessively conservative, the Committee notes that engineers are content with an FS only slightly in excess of unity. The SSE at rock for the existing NAPS units has a maximum acceleration of 0.12g, amplified to 0.18g in the soil. The seismic margin maximum acceleration in soil (Ref. 174, SSAR Section 2.5) is 0.30g. The maximum ESP acceleration at hard bedrock rock is 0.39g, amplified at the unimproved soil surface to 0.99g (SSAR Table 2.5-46). Based on these results, which the applicant determined to be very conservative, the applicant considers an FS of 1.1 to be adequate for the Zone IIA soils at the ESP site.

Previous Liquefaction Analyses

Virginia Power performed a detailed liquefaction analysis at the NAPS site in December 1994 for a seismic margin assessment (Ref. 174, SSAR Section 2.5). For the analysis, Virginia Power used a maximum acceleration of 0.3g, a magnitude of 6.8, and three different approaches to assess the potential for soil liquefaction. For the first approach, Virginia Power used the Seed and Idriss simplified procedure (Ref. 175, SSAR Section 2.5), with some modifications to account for the age of the saprolites and for the overconsolidated nature of the saprolites. The resulting FSs range from 1.54 to 3.51. For the second approach, Virginia Power used a threshold strain analysis (Ref. 177, SSAR Section 2.5), with an average shear wave velocity in the saprolite of 950 ft/s, resulting in an FS just under 3.0. For the third approach, Virginia Power used the results of the 15 stress-controlled cyclic triaxial tests, described in SSAR Section 2.5.4.2.4. The FSs against liquefaction range from 1.51 to 1.99 for the SWR facilities (pump house, valve house, tie-in vault, and service water lines). Analysis of the SWR embankment gave FS values ranging from 0.91 to 3.61, with an average of more than 1.5. The applicant stated that the few values below 1 occurred in localized zones and concluded that overall FSs across the embankment are well within acceptable limits. A consistent pattern of low FSs across the foundation would indicate that significant movements of the embankments would occur.

Liquefaction Analyses Performed for ESP

Based on the deaggregation of the PSHA in SSAR Section 2.5.2, the applicant used two earthquakes in the liquefaction analysis. The low-frequency earthquake has a magnitude of 7.2 and a bedrock acceleration of 0.21g. The high-frequency earthquake has a magnitude of 5.4 and a bedrock acceleration of 0.43g. SSAR Table 2.5-46 shows the ZPA values for the four soil/rock profiles described in SSAR Section 2.5.4.7.1. Since the Zone IIB saprolite and the Zone III weathered rock are not liquefiable, the liquefaction analysis did not consider Profiles 2 and 3 in SSAR Table 2.5-46. In Profile 4, the Zone IIA saprolite is improved (i.e., this would be the profile for any safety-related structures founded on the Zone IIA saprolite). The applicant stated that the soil would be improved sufficiently to ensure that the improved soil has an FS greater than or equal to 1.1 using the SSE ground motion. In Profile 1, the Zone IIA saprolite (upper 30 ft) is not improved. Thus, the applicant considered only Profile 1 for the liquefaction analysis. As noted above, the applicant used PGA values of 0.57g and 0.99g for the liquefaction analyses, which are described below.

The applicant performed a liquefaction analysis of each sample of Zone IIA saprolite, obtained by SPT sampling during the ESP subsurface investigation, to determine the FS against liquefaction. The applicant also analyzed the CPT results following the method proposed by Youd, et al. (Ref. 178, SSAR Section 2.5). The applicant stated that, using PGA values of 0.57g and 0.99g, the analysis of the SPT results gave FS values against liquefaction of greater than 1.1, except in one case. The applicant's analysis of the CPT results shows 5-foot thick zones in two CPTs and a 22-foot thick zone in another CPT, where the FS values against liquefaction are less than 1.1, implying that these soil zones would liquefy.

The applicant also performed a liquefaction analysis using shear wave velocity criteria incorporating the design values of shear wave velocity shown in SSAR Figure 2.5-62 and tabulated in SSAR Table 2.5-46. To correct the shear wave velocity values for overburden pressure, the applicant used the method outlined in Youd, et al. (Ref. 178, SSAR Section 2.5). The resulting values all fell into the no-liquefaction zone in Figure 9 of Reference 178. However, when the applicant used the lower bound values of the shear wave velocity, shown in SSAR Table 2.5-45, in the liquefaction analysis, most of the top 20 ft of Profile 1 fell into the liquefaction zone as shown in Figure 9 of Reference 178.

The applicant also determined the liquefaction-induced dynamic settlement using the method outlined in Tokimatsu and Seed (Ref. 179, SSAR Section 2.5). The maximum estimated dynamic settlement of the Zone IIA saprolite caused by earthquake shaking is about 5 in.

The applicant concluded the following concerning the liquefaction potential of the soils at the ESP site:

- Only the Zone IIA saprolites fall into the gradation and relative density categories where liquefaction would be considered possible.

- The structure, fabric, and mineralogy of these saprolites lower the potential for liquefaction very substantially.

- For a conventional liquefaction analysis, an FS of 1.1 is adequate, based on the conservative estimate of the ESP design seismic acceleration.

- A liquefaction analysis of the ESP SPT samples using the low- and high-frequency ESP seismic parameters gave FS values greater than 1.1 for all except one SPT result.

- A liquefaction analysis of the ESP CPT measurements using the low- and high-frequency ESP seismic parameters indicated an approximately 22-ft-thick zone and two 5-ft-thick zones where the FS against liquefaction was less than 1.1.

- A liquefaction analysis of the shear wave velocity profile indicated no liquefaction when the average shear wave velocity values were used. Using lower shear wave velocity values resulted in liquefaction of most of the top 20 ft.

- Estimated dynamic settlements caused by earthquake shaking are about 5 in.

Based on the above analysis, the applicant concluded that some of the Zone IIA saprolitic soils have a potential for liquefaction based on the low- and high-frequency ESP seismic parameters. The applicant stated that the liquefaction analysis did not take into account the beneficial effects of the fabric of the saprolitic soil. The applicant concluded by stating that, if safety-related structures are founded on the Zone IIA saprolitic soils, these soils would be improved to reduce potential settlements and to ensure that the FS against liquefaction is equal to or greater than 1.1.

In RAI 2.5.4-10, the staff asked the applicant to describe how it varied the significant soil properties and seismic input values for each of the different liquefaction analyses. In addition, the staff asked the applicant to provide a sample liquefaction analysis. In its response, the applicant stated that it based its liquefaction analyses on the work of Youd et al. (Ref. 178, SSAR Section 2.5). For each of the three different analyses, the applicant varied G_{max}, the peak earthquake acceleration, and the earthquake magnitude.

2.5.4.1.9 Earthquake Design Basis

SSAR Section 2.5.4.9 refers to SSAR Section 2.5.2.6, which derives and discusses the SSE for the ESP site in detail. Section 2.5.2 of this SER contains the staff's review of that information.

2.5.4.1.10 Static Stability

SSAR Section 2.5.4.10 describes the allowable bearing capacities for each subsurface zone as well as the estimated settlement for each zone. The applicant stated that reactor containment buildings at the ESP site would be founded on Zone III-IV or Zone IV bedrock. Depending on the location of these buildings, the top of this bedrock could occur below the level of the shallower reactor designs. In such cases, the applicant stated that it would excavate to sound bedrock and pour lean concrete up to the bottom of the reactor foundation. In some locations, the top of Zone III-IV or Zone IV bedrock is found close to or even above the planned plant grade. In such cases, safety-related structures would be founded on bedrock or on a thin layer of lean concrete or compacted structural fill on the bedrock. In other locations, sound bedrock is relatively deep. In this case, the applicant stated that safety-related structures (excluding the reactors) may be founded on the Zone III weathered rock, Zone IIB saprolite, or Zone IIA saprolite. The following sections on bearing capacity and settlement focus on this latter situation. (As noted in SSAR Section 2.5.4.10.2, the applicant stated that it would improve any Zone IIA saprolites supporting safety-related structures to reduce potential settlement.)

<u>Bearing Capacity</u>

Table 2.5-47 in the SSAR gives the allowable bearing capacity values for each zone. The applicant stated that it based the Zone IIA allowable bearing capacity value of 4 ksf (4000 lb/ft^2) on Terzaghi's bearing capacity equations modified by Vesic (Ref. 180, SSAR Section 2.5). According to the applicant, the analysis considers the effective strength parameters for the coarse-grained material and both the undrained and effective strength parameters for the fine-grained material given in SSAR Table 2.5-45. As discussed in SSAR Section 2.5.4.10.2, settlement considerations usually dominate when this material is used for supporting foundations, and the actual allowable bearing capacity may be less than 4 ksf (especially for larger foundations) if the soils are not improved. The applicant stated that it based the Zone IIB allowable bearing capacity value of 8 ksf on Terzaghi's bearing capacity equations modified by Vesic (Ref. 180, SSAR Section 2.5), using the effective angle of friction given in SSAR Table 2.5-45. Since the Zone IIB soil is usually found beneath the ground water table, the applicant used the effective unit weight of the soil in computing the 8 ksf value. The Zone III allowable bearing capacity of 16 ksf is based on the value of 20 percent of the ultimate crushing strength given in several building codes. Table 2.5-45 in the SSAR gives the ultimate crushing strength as 86 ksf. The 16 ksf value is slightly lower than the 20 ksf given for weathered rock in Table 2.5-2 of the UFSAR. The applicant stated that although the 16 ksf allowable bearing capacity exceeds the maximum bearing pressures of many of the reactor designs considered in

the application, the containment (reactor) buildings would not be founded on the Zone III weathered rock. The bedrock in Zones III-IV and IV has an unconfined compressive strength of 4 ksi (576 ksf) and 12 ksi (1728 ksf), respectively (SSAR Table 2.5-45). The applicant stated that allowable bearing capacities of these materials are much higher than any applied structure bearing pressure. In addition, the applicant stated that, if excavation during construction reveals any weathered or fractured zones at the foundation level, it would excavate such zones and replace them with lean concrete. The allowable values of the bearing capacity of 80 and 160 ksf for Zone III-IV and IV rock, shown in SSAR Table 2-5.47, are presumptive values based on various building codes for moderately weathered to fresh foliated rock.

In RAI 2.5.4-11, the staff asked the applicant to provide further details concerning its calculation of the bearing capacities of the soil and rock underlying the ESP site. In its response, the applicant provided a sample calculation for the staff to review. In addition, the applicant stated that the maximum bearing pressure from the containment building foundation is 15 ksf, which is only a fraction of the allowable bearing capacity of the bedrock (Zone III-IV is 80 ksf and Zone IV is 160 ksf).

Settlement Analysis

Peck et al. (Ref. 182, SSAR Section 2.5) indicates that total settlement should be limited to 2 in., and differential settlement to 0.75 in., for the large mat foundations that support major power plant structures. According to Peck, for footings that support smaller plant components, the total settlement should be limited to 1 in. and the differential settlement to 0.5 in.

Settlement of Materials in Zones IIB, III, III-IV, and IV

The applicant stated that the settlement of the materials in Zones IIB, III, III-IV, and IV is essentially elastic. The applicant analyzed a large foundation with an assumed size of 150 ft by 300 ft (e.g., a turbine building foundation) for settlement assuming a soil profile of 20 ft for Zone IIB, underlain by 30 ft of Zone III, 50 ft of Zone III-IV, and 400 ft of Zone IV. The applicant used the high-strain elastic modulus values given in SSAR Table 2.5-45 as the stiffness values. The applicant found that the foundation has an average bearing pressure of 6 ksf. The computed total settlement of this structure is less than 0.5 in.

Settlement of Zone IIA

The applicant stated that Virginia Power recorded larger settlements than expected (i.e., 4.6 in.) beneath the SWR pump house of the existing units because of the weight of the pump house and the 30 ft of embankment fill that was built up around it. This settlement occurred over a 30-month period. The in-situ soil that settled beneath the pump house consists of about 65 ft of Zone IIA, mainly micaceous sandy silt. The applicant stated that the primary cause of this fairly large settlement appears to be the 5 to 20 percent mica content of these saprolites, along with a significant portion of low-plasticity clay minerals. The applicant concluded that, although the settlement of the SWR pump house is an extreme case and resulted from several factors, the potential for excessive settlement of the Zone IIA saprolite makes this material unsuitable to support any safety-related structure without ground improvement.

2.5.4.1.11 Design Criteria

SSAR Section 2.5.4.11 summarizes the geotechnical design criteria. In addition, various sections of the SSAR cover other applicable design criteria. SSAR Section 2.5.4.8 specifies that the acceptable FS against liquefaction of site soils should be 1.1. SSAR Section 2.5.4.10 presents bearing capacity and settlement criteria. SSAR Table 2.5-47 provides allowable bearing capacity values for the site subsurface materials. Generally acceptable total and differential settlements are limited to 2 in. and 0.75 in., respectively, for mat foundations and 1 in. and 0.5 in., respectively, for footings. 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 specifies that the minimum acceptable long-term seismic FS against slope stability failure is 1.1.

In RAI 2.5.4-12, the staff asked the applicant to explain why it did not provide design criteria pertaining to structural design. In its response, the applicant stated that structural criteria such as allowable wall rotation and FSs against structure sliding and overturning are not site specific and thus are not included in SSAR Section 2.5. The applicant stated that a COL application would describe these structural criteria.

2.5.4.1.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 outlines several ground improvement techniques that would be implemented before the Zone IIA saprolitic soils could be used to support safety-related foundations. As its primary choice for reducing the settlement potential of the Zone IIA saprolitic soils, the applicant considered vibro-stone columns. According to the applicant, vibro-stone columns have several advantages, including reduction of settlement, improvement of bearing capacity, and reduction of liquefaction potential. The vibro-stone column method involves the insertion of a vibratory probe (aided by water jets or compressed air) into the base of the stratum that needs improvement. Crushed stone is poured into the annulus and is densified by the vibrator. This process results in a series of highly compacted stone columns, typically about 3 ft in diameter, spaced on about 5- to 8-ft centers.

2.5.4.2 Regulatory Evaluation

SSAR Section 2.5.4 describes the applicant's evaluation of the stability of the subsurface materials and foundations at the ESP site. In SSAR Section 1.8, the applicant stated that it developed the geological, geophysical, and geotechnical information used to evaluate the stability of the subsurface materials in accordance with the requirements of 10 CFR 100.23. The applicant applied the guidance of RS-002, RG 1.70, DG-1105 (which has been superseded by RG 1.198 since the applicant submitted the SSAR), RG 1.132; and RG 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants."

In its review of SSAR Section 2.5.4, the staff considered the regulatory requirements in 10 CFR 100.23(c) and 10 CFR 100.23(d)(4). 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 soil and rock stability, liquefaction potential, and natural and artificial slope stability. Section 2.5.4 of RS-002 provides specific guidance concerning the evaluation of information characterizing the stability of subsurface materials, including the need for geotechnical field and laboratory tests as well as the geophysical investigations.

2.5.4.3 Technical Evaluation

This section provides the staff's evaluation of the geophysical and geotechnical investigations carried out by the applicant to determine the static and dynamic engineering properties of the materials that underlie the ESP site. The technical information presented in SSAR Section 2.5.4 resulted from the applicant's field and laboratory investigations performed for the ESP. The applicant intended its additional field and laboratory investigations to confirm the large volume of geotechnical data developed by Virginia Power for the existing units and the abandoned Units 3 and 4 within the ESP site area. The applicant used the subsurface material properties from its field and laboratory investigations to evaluate the liquefaction potential, bearing capacity, and potential for settlement.

Through its review of SSAR Section 2.5.4, the staff determined whether the applicant demonstrated the stability of the subsurface materials under both static and dynamic conditions. The staff also reviewed the applicant's field and laboratory investigations used to determine the geotechnical properties of the soil and rock underlying the ESP site. In addition, the staff observed some of the applicant's onsite borings and field explorations, performed in November and December 2002, to determine whether the applicant followed the guidance in RG 1.132.

2.5.4.3.1 Geologic Features

SSAR Section 2.5.4.1 references SSAR Sections 2.5.1.1 and 2.5.1.2 for a description of the regional and site geology. Section 2.5.1.3 of this SER presents the staff's evaluation of these two sections.

2.5.4.3.2 Properties of Subsurface Materials

The staff focused its review of SSAR Sections 2.5.4.2 and 2.5.4.3 on the applicant's description of (1) subsurface materials, (2) field investigations, (3) laboratory testing, and (4) static and dynamic engineering properties of the ESP site subsurface materials.

Normally, an applicant performs a complete field investigation and sampling program to evaluate the engineering properties and stability of the soil and rock underlying the site. However, since the applicant relied on Virginia Power's previous field and laboratory investigations for the existing and abandoned units, it used its ESP investigations to confirm previously established soil and rock properties. In RAI 2.5.4-1, the staff asked the applicant to provide its basis for concluding that the subsurface conditions in the southeast portion of the ESP footprint (an area of about 500 ft by 1000 ft, in which there are no borings) do not materially differ from conditions in adjacent areas, where borings were made. In response to RAI 2.5.4-1, the applicant stated that the North Anna site is underlain by a consistent geologic profile, which extends to a depth of several thousand feet. The applicant stated that the 145 borings performed throughout the North Anna site (including 7 borings for the ESP) indicate a consistent overall subsurface profile, with expected variations in the thickness of the various strata. As such, the applicant concluded that the southeast portion of the ESP footprint (see SER Figure 2.5.4-3) should be similar to the rest of the site. Because of the consistency of the soil and rock engineering properties across the NAPS and ESP sites, the staff has determined that Virginia Power's past investigations, combined with the ESP applicant's explorations, are adequate to characterize the subsurface conditions in the locations where data were collected. Further, based on its review of the NAPS and ESP borings, the staff has determined that a consistent geologic profile underlies the North Anna ESP site. The staff concludes, therefore, that the uncharacterized southeast portion of the site should have subsurface conditions similar to those found at the rest of the site. Accordingly, the staff concludes that the applicant has provided an adequate description of the subsurface profile. The applicant's commitment to perform additional borings to confirm its conclusions regarding engineering properties and the stability of soil and rock underlying future plant SSCs is **COL Action Item 2.5-1.**

In RAI 2.5.4-3, the staff asked the applicant to describe how it integrated Virginia Power's site investigations for the SWR and the ISFSI with its field investigations for the ESP site. In its response, the applicant stated that the SWR and ISFSI borings are as close to the ESP area as any other borings and disclose the same subsurface profile displayed by the other borings at the North Anna site (see SER Figure 2.5.4-3). In addition, the applicant stated that it used some of the SWR and ISFSI borings, which are close to the southeast corner of the ESP footprint, noted in RAI 2.5.4-1, to help characterize the ESP area. Because of the consistency of the soil and rock engineering properties across the NAPS and ESP sites, the staff has determined that Virginia Power's past investigations, combined with the ESP applicant's explorations, are adequate to characterize the subsurface conditions in the locations where data were collected.

In RAI 2.5.4-4, the staff asked the applicant to explain how the total thickness of the soil layers sampled at the ESP site (105 ft) is sufficient to characterize the soil properties underlying the site. In its response, the applicant stated that the 138 borings performed previously by Virginia Power for Units 1 and 2 as well as abandoned Units 3 and 4 characterize the soils at the North Anna site very well. The applicant stated that the soils in all the borings show the same general subsurface profile and that it used the ESP borings to show that the soil (and rock) profiles in each of the borings fit within the general subsurface profile. Based on the results of the NAPS and ESP borings, the staff has determined that a consistent geologic profile underlies the North Anna ESP site. The staff concludes, therefore, that the applicant adequately sampled the soil underlying the ESP site in order to confirm the results of borings previously performed by Virginia Power.

In RAI 2.5.4-2(a), the staff asked the applicant to describe the extent of severely weathered fracture zones in the Zone III-IV and IV rock that Virginia Power observed during the site investigation for abandoned Units 3 and 4. The applicant observed similarly fractured rock in four of the seven ESP borings. In its response, the applicant provided a table that shows an RQD of less than 25 percent in nine of the borings for abandoned Units 3 and 4. The applicant noted that most of the rock for the low RQD intervals (less than 10 percent) is only 1 to 2 ft thick. In RAI 2.5.4-2(b), the staff asked the applicant to describe the impact of these fractured rock zones on the suitability of the site to host safety-related structures. In its response, the applicant stated the following:

> As noted in these SSAR sections, any weathered or fractured zones encountered at foundation level would be excavated and replaced with lean concrete. If such zones exist below sound rock beneath the foundation, they

> would have no impact on the stability of the foundation, since these zones are typically only 0.5 to 1-foot thick, and are confined within an unfractured rock mass with strengths of 4,000 to 12,000 psi (compared to the maximum foundation pressure of just over 100 psi). The foundation itself would consist of a large, thick, highly-reinforced concrete mat that is so stiff that it cannot logically yield.
>
> Multiple borings would be performed at each structure location once the building locations are chosen as part of detailed engineering. These borings would identify whether there are any thicker fracture zones beneath the foundation than those encountered in the ESP borings and in the abandoned Units 3 and 4 borings. If any thicker zones are found, analysis would be performed to identify their impact on foundation stability. If they are close enough to the foundation to potentially impact stability, they would be excavated and replaced with lean concrete.

In its response to RAI 2.5.4-2, the applicant stated its commitment to excavate and replace with lean concrete any weathered or fractured zones found at the foundation level, and the staff proposes to include a condition in the ESP to require such activities (Permit Condition 2.5-1). The replacement of fractured rock with lean concrete is well understood and commonly done to enhance the strength and stability of the rock to support building loads. The excavation of weathered or fractured rock zones and their replacement with lean concrete will ensure the bearing capacity of such zones. The staff concludes that this is adequate to ensure the stability of structures that might be constructed on the proposed site.

In RAI 2.5.4-6, the staff asked the applicant to explain why it did not provide laboratory test results from the borings of subsurface materials over various depth intervals. In response to RAI 2.5.4-6, the applicant stated that the containment (reactor) buildings for the new units would be founded on the Zone III-IV and/or Zone IV metamorphic gneiss bedrock at the North Anna site. Rock coring and testing performed by Virginia Power for Units 1 and 2 gave unconfined compressive strengths for the Zone III-IV and IV rock ranging from 1,000 to 16,300 psi, with a median strength of 6,800 psi. The applicant stated that these rock strengths are typical for this type of rock and more than sufficient to support the maximum containment (reactor) building loads of about 100 psi. The applicant added that, during logging of the rock cores in the field for the ESP investigation, it was apparent that the metamorphic rock is a strong material. The applicant performed tests on the ESP cores sufficient to verify that the rock strengths are similar to or higher than those cores tested for Units 1 and 2. The applicant determined that the median value of the unconfined compressive strengths of the Zone III-IV and IV rock from the ESP investigation is 18,400 psi. Because the applicant verified through rock coring and testing during its ESP investigation that the unconfined compressive strength of the Zone III-IV and IV rock is similar to or higher that the cores tested for Units 1 and 2, the staff concludes that the applicant has adequately sampled the Zone III-IV and IV rock.

Furthermore, the staff concurs with the applicant's conclusion that the strength of the Zone III-IV and IV rock is sufficient to support the load of a containment building.

Based on its review of SSAR Sections 2.5.4.2 and 2.5.4.3 and the applicant's responses to its RAIs, as described above, the staff concludes that the applicant adequately determined the engineering properties of the soil and rock underlying the ESP site through its field and laboratory investigations. In addition, the applicant used the latest field and laboratory methods, in accordance with RGs 1.132 and 1.138, to determine these properties. Accordingly, the staff concludes that the applicant performed field investigation and laboratory testing sufficient to determine the overall subsurface profile as well as the material properties underlying the ESP site. The staff notes that the applicant committed to perform additional investigations once it has selected the building locations. The COL (or CP) applicant would describe these additional investigations in its COL (or CP) application.

2.5.4.3.3 Relationship of Foundations and Underlying Materials

Section 2.5.4.3 in RS-002 directs the staff to compare the applicant's plot plans and the profiles of all seismic Category I facilities with the subsurface profile and material properties. Based on this comparison, the staff can determine if (1) the applicant performed sufficient exploration of the subsurface and (2) the applicant's foundation design assumptions contain adequate margins of safety. The applicant decided to defer providing this information until a CP or COL application is submitted. Submission of a COL or CP applicant's plot plans and the profiles of all seismic Category I facilities for comparison with the subsurface profile and material properties is **COL Action Item 2.5-2.**

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 soil and rock dynamic properties. The applicant performed two crosshole seismic tests, one downhole seismic test, and two CPT seismic tests. The applicant compared the dynamic properties it obtained from these tests with the results from the previous geophysical surveys of the North Anna site performed by Virginia Power.

In RAI 2.5.4-5, the staff asked the applicant to explain why SSAR Table 2.5-45 does not provide shear wave velocities for Zone IIB saprolite and Zone III and III-IV weathered rock. In its response, the applicant stated that SSAR Table 2.5-45 gives average shear wave velocities for Zones IIB, III, and III-IV but does not provide a range of values. In contrast, it gives both average values and a range of shear wave velocity values for Zones IIA and IV. The applicant stated that it provided only average values for Zones IIB, III, and III-IV because the ESP borings did not sample these zones as abundantly as Zones IIA and IV. In response to this RAI, the applicant also provided its method for determining the average shear wave velocity values for Zones IIB (1600 ft/s), III (2000 ft/s), and III-IV (3300 ft/s). Because the applicant used both crosshole and downhole seismic tests, as well as direct and indirect methods, the staff concludes that the applicant has adequately measured the shear wave velocity for each of the soil and rock zones. For those zones (IIB, III, and III-IV) for which the applicant did not obtain so many samples from the ESP borings, the applicant used its laboratory measurements of the soil/rock properties to indirectly determine the shear wave velocities. Accordingly, the staff concludes that the applicant adequately sampled the soil and rock underlying the ESP site in order to determine the consistency of its dynamic properties with those previously obtained by Virginia Power in earlier explorations.

The staff has determined that the applicant used the latest geophysical and geotechnical measurement methods and equipment in accordance with the recommendations of RGs 1.132 and 1.138 to determine the dynamic properties of the soil and rock underlying the site. Based on its review of SSAR Section 2.5.4.4 and the applicant's response to the RAI, described above, the staff concludes that the applicant adequately determined the soil and rock dynamic properties through its geophysical survey of the ESP site.

2.5.4.3.5 Excavation and Backfill

In SSAR Section 2.5.4.5, the applicant provided a general description of (1) the extent (horizontally and vertically) of anticipated safety-related excavations, fills, and slopes, (2) excavation methods and stability, (3) backfill sources and quality control, and (4) control of ground water during excavation. The staff found this general description to be useful. However, the applicant has not selected a reactor design or location within the ESP site, and it did not provide detailed excavation and backfill plans or plot plans and profiles as outlined in Section 2.5.4 of RS-002. Therefore, the staff could not adequately evaluate the applicant's excavation and backfill plans and will await the future submittal of these plans by the ESP holder and/or as part of a COL or CP application. This is **COL Action Item 2.5-3.** The staff notes that, in SSAR Section 2.5.4.5, the applicant stated that it would (1) geologically map future excavations for safety-related structures and (2) evaluate any unforseen geologic features that are encountered. In addition, the applicant stated that it would notify the NRC "when any excavations for safety-related structures are open for their examination and evaluation." The staff proposes to include a condition in any ESP that might be issued requiring that the ESP holder and/or an applicant referencing such an ESP perform geologic mapping of future excavations for safety-related structures, evaluate any unforseen 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 examinations and evaluation. This is **Permit Condition 7.**

2.5.4.3.6 Ground Water Conditions

In SSAR Section 2.5.4.6, the applicant provided a general description of (1) ground water measurements and elevations and (2) construction dewatering plans. The staff found this general description to be useful. However, the applicant has not selected a reactor design or location within the ESP site and did not provide an evaluation of ground water conditions as they affect foundation stability or detailed dewatering plans as outlined in Section 2.5.4 of RS-002. Therefore, the staff could not evaluate the ground water conditions as they affect the loading and stability of foundation materials or the applicant's dewatering plans during construction, as well as ground water control throughout the life of the plant. As such, the staff will await the future submittal of these evaluations and plans as part of the COL or CP application. The need to evaluate ground water conditions as they affect foundation stability or detailed dewatering plans is **COL Action Item 2.5-4.**

2.5.4.3.7 Response of Soil and Rock to Dynamic Loading

In its review of SSAR Section 2.5.4.7, the staff focused on the applicant's shear wave velocity design profiles to determine the response of the soil and rock underlying the ESP site to dynamic loading. In addition, the staff reviewed the applicant's modeling of the variation of soil shear modulus and damping with cyclic shear strain. Finally, the staff reviewed the applicant's site dynamic response, which was based on a soil amplification/attenuation analysis using the four soil profiles.

In RAI 2.5.4-7, the staff asked the applicant to reconcile two conflicting statements in SSAR Sections 2.5.4.7.1 and 2.5.1.2.6. The applicant stated in SSAR Section 2.5.1.2.6 that Zone III (weathered rock) is not a suitable material for safety-related plant structures. However, the applicant stated in SSAR Section 2.5.4.7.1 that some safety-related structures (excluding the reactor containment building) may be founded on the Zone III weathered rock, Zone IIB saprolite, or improved Zone IIA saprolite. In response to RAI 2.5.4-7, the applicant stated that the statement in SSAR Section 2.5.4.7.1 is correct and that it will delete the statement in SSAR Section 2.5.1.2.6. The applicant emphasized that only improved Zone IIA saprolite is appropriate for certain safety-related structures only if it is improved (see RAI 2.5.4-11 below). Based on the applicant's clarification in its response to RAI 2.5.4-7, the staff concludes that it is appropriate to consider the construction of safety-related structures on improved Zone IIA, and Zone IIB, and Zone III materials.

In RAI 2.5.4-8, the staff asked the applicant to provide its basis for the selected modulus reduction and damping ratio curves for Zones IIA, IIB, and III materials. In response to RAI 2.5.4-8, the applicant stated that it used the 1993 EPRI report (Ref. 170, SSAR Section 2.5.2), where applicable, as the basis for the shear modulus reduction and damping ratio curves. The staff reviewed the curves that the applicant selected for each of the soil and rock zones to determine whether the applicant based its selection on appropriate criteria, such as grain size, cohesiveness, confining pressure, and shear wave velocity. The staff concludes that the shear modulus and damping curves selected by the applicant were based on appropriate criteria and are suitable for Zone IIA, IIB, and III soil and rock.

In RAI 2.5.4-8(c), the staff asked the applicant to explain its use of a damping ratio of 2 percent for the Zone III-IV rock. In response to RAI 2.5.4-8(c), the applicant stated that the damping ratio for rock varies from site to site depending on various factors, including the mineral composition of the rock, the integrity and fissuring of the rock mass, and the level of shear deformation in the rock formation. According to the applicant, damping ratios for rock are generally between 0.5 to 4.5 percent. The applicant selected 2 percent for the Zone III-IV rock based on engineering judgment and past experience. To determine the sensitivity of the selected damping ratio, the applicant reran its analysis using damping ratios of 0.5, 1.0, and 5.0 percent. The results reveal only a slight difference in the peak acceleration for the different damping ratios. Based on these results, the staff concludes that a damping ratio of 2 percent for the Zone III-IV rock is acceptable.

In RAI 2.5.4-9(a), the staff asked the applicant to describe the method that it used for the development of the site-specific acceleration time histories. In response to RAI 2.5.4-9(a), the applicant stated that it selected two horizontal-component acceleration time histories, which it then matched to the low- and high-frequency response spectra from the two controlling earthquakes. The applicant next used these spectrum-compatible time histories for the site response analysis. In RAI 2.5.4-9(b), the staff asked the applicant to further describe the method it used for the development of the soil column amplification/attenuation analysis. In response to RAI 2.5.4-9(b), the applicant stated that it used the SHAKE2000 computer program to compute the site dynamic responses for the four soil and rock profiles described in SSAR Section 2.5.4.7.1. The applicant provided the input soil parameters, the depth at which the hard rock ground was input (70 ft), and information on the number of iterations to compute the strain-compatible modulus and damping values for the SHAKE analysis. In RAIs 2.5.4-9(c) and (d), the staff asked the applicant to further describe the four soil profiles and how it accounted for the variability of the soil properties in the analysis. In response to RAIs 2.5.4-9(c) and (d), the applicant provided the soil properties for each of the four profiles and an analysis that demonstrated how it varied these properties. The applicant stated that V_s and G_{max}, which is derived from V_s, have the most impact on the amplification/attenuation analysis. The applicant showed response spectra for different levels of G_{max} (67 to 150 percent). In RAI 2.5.4-9(e), the staff asked the applicant to justify its use of the mean 10^{-4} uniform hazard spectrum as the input rock motion. In response to RAI 2.5.4-9(e), the applicant stated that it initially used a time history matched to the mean 10^{-4} uniform hazard spectrum; however, in Revision 3 to its SSAR, it revised this approach to use time histories that match the low- and high-frequency response spectra calculated from the two controlling earthquakes. Because the applicant used both the low-frequency and high-frequency time histories and four different rock/soil profiles and also accounted for the variability in the soil and rock properties, the staff concludes that the applicant accurately determined the dynamic response of the soil and rock underlying the ESP site to the input hard rock ground motion. As a result of RAI 2.5.4-9, the applicant revised portions of SSAR Sections 2.5.4.7 and 2.5.4.8.

Based on its review of SSAR Section 2.5.4.7 and the applicant's responses to the RAIs, as described above, the staff concludes that the applicant adequately determined the response of the soil and rock underlying the ESP site to dynamic loading. The staff notes the applicant's commitment in response to RAI 2.5.4-9 to perform further soil column amplification/attenuation analyses at the COL stage, once it selects specific locations for the nuclear power plant structures. This is **COL Action Item 2.5-5.** The applicant stated that this analysis would involve subsurface investigations to determine actual strata thicknesses and confirm the subsurface material properties at each location.

2.5.4.3.8 Liquefaction Potential

In its review of SSAR Section 2.5.4.8, the staff evaluated the applicant's liquefaction analyses. The staff's review focused on the applicant's conclusion that only the Zone IIA saprolite is susceptible to liquefaction, as well as the various liquefaction analyses and parameter inputs to these analyses. The applicant concluded that soil Profile 1, which has 30 ft of unimproved Zone IIA saprolite, is potentially susceptible to liquefaction in most of the upper portions. The applicant stated that, if safety-related structures are founded on the Zone IIA saprolitic soils, these soils would be improved to reduce any liquefaction potential.

In RAI 2.5.4-10, the staff asked the applicant to describe how it varied the significant soil properties and seismic input values for each of the different liquefaction analyses. In addition, the staff asked the applicant to provide a sample liquefaction analysis. In its response, the applicant stated that it based the liquefaction analyses on the work of Youd et al. (Ref. 178, SSAR Section 2.5). For each of the three different analyses, the applicant varied G_{max}, the peak earthquake acceleration, and the earthquake magnitude. Based on its review of the sample liquefaction analysis, the staff concludes that the applicant used the latest empirical method and adequately varied the significant soil and seismic input parameters in accordance with the guidance provided in RG 1.198, which recommends the Youd et al. method. Therefore, the applicant's liquefaction analyses are acceptable.

Based on its review of SSAR Section 2.5.4.8 and the applicant's response to RAI 2.5.4-10, described above, the staff concludes that the applicant has employed an acceptable methodology to determine the liquefaction potential of the soil underlying the ESP site.

Because portions of the Zone IIA saprolite are susceptible to liquefaction, the applicant stated that, if safety-related structures are founded on the Zone II saprolitic soils, these soils would be improved to reduce any liquefaction potential. Accordingly, the staff proposes to include a condition for any ESP that might be issued requiring that the ESP holder and/or an applicant referencing such an ESP improve Zone II saprolitic soils to reduce any liquefaction potential if safety-related structures are to be founded on them. This is **Permit Condition 8.** The applicant described techniques for improving the Zone IIA saprolitic soils in SSAR Section 2.5.4.12.

2.5.4.3.9 Earthquake Design Basis

SSAR Section 2.5.2.6 presents the applicant's derivation of the SSE. Section 2.5.2.3.6 of this SER summarizes the staff's evaluation of the SSE.

2.5.4.3.10 Static Stability

In its review of SSAR Section 2.5.4.10, the staff focused on the applicant's determination of the bearing capacities for each of the soil and rock zones, as well as the applicant's settlement analysis. The applicant presented bearing capacities for each of the soil and rock zones and described how it obtained these results. In addition, the applicant stated that the settlement of a large foundation with an assumed size of 150 ft by 300 ft, underlain by Zone IIB, would be less than 0.5 in.

In RAI 2.5.4-11, the staff asked the applicant to provide further details concerning its calculation of the bearing capacities of the soil and rock underlying the ESP site. In its response, the applicant provided a sample calculation for the staff to review. In addition, the applicant stated that the maximum bearing pressure from the containment building foundation is 15 ksf, which is only a fraction of the allowable bearing capacity of the bedrock (Zone III-IV is 80 ksf and Zone IV is 160 ksf). During its review of the sample bearing capacity calculation, the staff determined that the applicant used the widely accepted bearing capacity formulas developed by Terzaghi (D.P. Coduto, "Foundation Design," 2nd edition, issued 2001). Accordingly, the staff concludes that the applicant adequately determined bearing capacity values for each of the soil and rock zones. In addition, the staff concludes that the bearing capacities of Zones III-IV and IV rock are sufficient to handle the load of a containment building foundation.

Based on its review of SSAR Section 2.5.4.10 as described above, the staff concludes that the applicant provided an adequate preliminary assessment of the static stability of the ESP site. However, as described in RS-002, for the staff to perform a complete review of site static stability, the staff will need a COL or CP applicant to provide an analysis of the stability of all planned safety-related facilities when the locations of the plant structures are finally specified. This analysis should include bearing capacity, rebound, settlement, and differential settlements, as well as lateral loading conditions for all safety-related facilities. Therefore, the staff concludes that the applicant's description of the static stability is adequate to provide assurance of the stability of the ESP site, but the staff needs additional information to support any finding regarding detailed structure-specific stability. The need to provide an analysis of the stability of all planned safety-related facilities, including bearing capacity, rebound, settlement, and differential settlements under deadloads of fills and plant facilities, as well as lateral loading conditions, is **COL Action Item 2.5-6.**

2.5.4.3.11 Design Criteria

In SSAR Section 2.5.4.11, the applicant provided general geotechnical criteria, such as acceptable FSs against liquefaction, allowable bearing capacities, acceptable total and differential settlements, and acceptable FSs against slope stability failure. The applicant did not provide structural design criteria, such as wall rotation, sliding, and overturning.

In RAI 2.5.4-12, the staff asked the applicant to explain why it did not provide design criteria pertaining to structural design. In its response, the applicant stated that structural criteria, such as allowable wall rotation and FSs against structure sliding and overturning, are not site specific and thus are not included in SSAR Section 2.5. The applicant stated that a COL application would describe these structural criteria. Since 10 CFR Part 52, Subpart A, does not require the submission of such information, the staff concludes that the applicant's decision not to include structural design criteria in the ESP applicant is justified.

Based on its review of SSAR Section 2.5.4.11 and the applicant's response to the RAI, the staff concludes that the applicant adequately presented the necessary design criteria for the ESP site. The need to provide design-related criteria that pertain to structural design (such as wall rotation, sliding, and overturning) is **COL Action Item 2.5-7.**

2.5.4.3.12 Techniques to Improve Subsurface Conditions

In SSAR Section 2.5.4.12, the applicant presented a general description of the ground improvement techniques it may employ so that the Zone IIA saprolitic soils could be used to support safety-related foundations. Although this general description was useful to the staff, a COL or CP applicant should provide specific plans for each proposed ground improvement technique it plans to employ so that the staff may determine whether the chosen techniques will ensure that Zone IIA saprolitic soils will be able to support safety-related foundations. This is **COL Action Item 2.5-8.**

2.5.4.4 Conclusions

Based on its review of SSAR Section 2.5.4 and the applicant's responses to the associated RAIs, described above, the staff concludes that the applicant adequately determined the engineering properties of the soil and rock underlying the ESP site through its field and laboratory investigations. In addition, the applicant used the latest field and laboratory methods, in accordance with RGs 1.132, 1.138, and 1.198, to determine these properties. Accordingly, the staff concludes that the applicant performed sufficient field investigations and laboratory testing to determine the overall subsurface profile, as well as the properties of the soil and rock underlying the ESP site. Specifically, the staff concludes that the applicant adequately determined (1) the soil and rock properties through its field investigations and laboratory tests, (2) the response of the soil and rock to dynamic loading, and (3) the liquefaction potential of the Zone IIA saprolitic soils. The staff notes that the applicant committed to perform additional field investigations once it has selected the locations for safety-related structures at the COL stage.

In SSAR Sections 2.5.4.5 (excavation and backfill), 2.5.4.6 (ground water conditions), 2.5.4.10 (static stability), 2.5.4.11 (design criteria), and 2.5.4.12 (techniques to improve subsurface conditions), the applicant did not provide information sufficient for the staff to perform a complete evaluation. In addition, the applicant did not provide any information on the relationship of the foundation and underlying materials (Section 2.5.4.3 in RS-002). Each of the these topics depends on specific information related to building location and design and will be submitted as part of any COL or CP application.

In SSAR Table 1.9-1, the applicant identified three subsurface material properties as ESP site characteristics. The first site characteristic specifies that there is no potential for liquefaction at the ESP site. The applicant demonstrated, in SSAR Section 2.5.4.1.8, that any liquefaction at the ESP site would be limited to the Zone IIA saprolites, and if any safety-related structures are founded on the Zone IIA saprolites, these soils would be improved to reduce potential settlements and to ensure an FS for liquefaction greater than or equal to 1.1. The second site characteristic specifies a minimum bearing capacity value of 15 ksf. The bearing capacities for rock of Zones III and above underlying the ESP site are greater than 15 ksf (see SSAR Table 2.5-45). Finally, the third site characteristic specifies a minimum shear wave velocity of 3500 ft/s for the material underlying the foundation. The applicant stated that the reactor containment would be founded on Zone III-IV or IV bedrock. Because the average shear wave velocity (V_s) of the Zone III-IV bedrock is slightly less (3300 ft/sec) than this postulated PPE value (3500 ft/sec), the COL or CP applicant should determine the V_s of the actual material underlying the foundation for the reactor containment to ensure that V_s equals or exceeds that of the chosen design. This is **COL Action Item 2.5-9.** The staff has reviewed the applicant's suggested site characteristics and plant design parameters related to SSAR Section 2.5.4 for inclusion in an ESP, should the NRC issue one to the applicant. For the reasons set forth above, the staff agrees with the applicant's site characteristics and values.

2.5.5 Stability of Slopes

SSAR Section 2.5.5 presents information on the stability of permanent slopes at the NAPS site. The applicant used previous geological, geophysical, and geotechnical investigations as a basis for determining the stability of the slopes at the site. SSAR Section 2.5.5.1 describes the existing slope characteristics, SSAR Section 2.5.5.2 describes the design criteria and analyses of slope stability, SSAR Section 2.5.5.3 presents information from two sample borings on or close to the slope, SSAR Section 2.5.5.4 states that the slope does not contain compacted fill, and SSAR Section 2.5.5.5 describes a potential new slope that may be excavated at the site.

2.5.5.1 Technical Information in the Application

2.5.5.1.1 Slope Stability Analysis and Design Criteria

<u>Existing Slope Characteristics</u>

SSAR Section 2.5.5.1 describes an existing 2-horizontal to 1-vertical (2h:1v), 55-ft-high slope that descends from north of the SWR down to the south of the existing excavation made for the abandoned NAPS Units 3 and 4. The slope was excavated during construction of NAPS Units 1 and 2 and is made almost entirely of cut material. Since the top of this slope is 200 ft from the top of the SWR embankment, the applicant concluded that any potential instability of the slope would have no impact on the stability of the SWR embankment. However, sloughing or collapse of the slope could impact the new units, depending on their final location.

The NAPS licensee took two slope borings, conducted for the Unit 1 and 2 investigation, close to the area of the slope. As shown in the boring profiles, the soils in the slope consist almost entirely of Zone IIA saprolites. Saprolites are a further stage of weathering beyond weathered rock. Although saprolites are classified as soils, they still contain the relict structure of the parent rock and some core stone of the parent rock. About 75 percent of the Zone IIA saprolites are classified as coarse grained (sands, silty sands), while the remainder are fine grained (clayey sands, sandy and clayey silts, and clays). The majority of the saprolites obtained from the borings in the slope area are dense silty sands.

Design Criteria and Analyses

SSAR Section 2.5.5.2 presents the design criteria for the slope, as well as an analysis of the static and dynamic (seismic) stability analysis. The design criteria used for the slope include the following minimum FSs:

- end of construction—FS=1.4
- long-term static (nonseismic)—FS=1.5
- long-term seismic—FS=1.1

The applicant inspected the slope during the ESP site investigation and found no signs of distress. In addition, a comparison of recent and old photographs of the site shows that the condition of the slope is unchanged.

For the static and dynamic analyses of the slope, the applicant used the computer program SLOPE/W, which is a commercial software product that employs limit equilibrium theory to compute the FS of earth and rock slopes. For the static analysis, the SLOPE/W program used the Bishop method of slices. The applicant assumed that the saprolite is predominantly coarse grained, with a unit weight of 125 pcf, an angle of internal friction (ϕ') of 30 degrees, and an effective cohesion (c') of 0.25 ksf. The resulting FS for the static analysis is 1.75, which is above the minimum FS of 1.5 for long-term static stability.

For the seismic slope stability analysis, the applicant used the pseudostatic approach, which assumes that the horizontal and vertical seismic forces act on the slope in a static manner as a constant force. Since an actual seismic event would last only seconds, with the peak motions occurring for a small portion of the total duration, the applicant concluded that the pseudostatic approach is a conservative approach. For the high-frequency earthquake, the applicant used a peak horizontal acceleration of 0.65g, which is the average peak acceleration in the top 55 ft of unimproved soil (see SSAR Table 2.5-46). Similarly, the applicant used a vertical peak acceleration of 0.32g. The applicant stated that the resulting FS is significantly less than 1.1, which is the minimum FS required for seismic slope stability. For the low-frequency earthquake, the applicant used a peak horizontal acceleration of 0.26g, which is the average peak acceleration in the top 55 ft, and a vertical acceleration of 0.13g. The computed FS for this case is slightly greater than 1.1.

As an alternative to applying the peak acceleration values for the pseudostatic analysis, the applicant chose to use horizontal accelerations of 0.15g and 0.10g and a vertical acceleration of zero. The applicant provided the following argument to support these acceleration values:

Seed (Reference 186), in the 19th Rankine Lecture, addressed the over-conservatism intrinsic in the pseudo-static analysis. He looked at the more rational approach proposed by Newmark (Reference 187), where the effective acceleration time-history is integrated to determine velocities and displacements of the slope. He also examined dams in California that had been subjected to seismic forces, including several dams that survived the 1906 San Francisco earthquake. Based on his studies, he concluded that for embankments that consist of materials that do not tend to build up large pore pressures or lose significant percentages of their shear strength during seismic shaking, seismic coefficients of only 0.15g are adequate to ensure acceptable embankment performance for earthquakes up to Magnitude M=8.25 with peak ground accelerations of 0.75g. For earthquakes in the range of M=6.5, Seed recommends a horizontal seismic coefficient of only 0.1g with a vertical seismic coefficient of zero.

Since the fabric and interlocking angular grain structure of the Zone IIA saprolite have a low susceptibility to pore pressure buildup and liquefaction, the applicant concluded that it would not lose a significant portion of its shear strength during shaking. In addition, since the controlling earthquake magnitudes for the ESP site are 5.4 and 7.2, the applicant concluded that using the acceleration values recommended by Seed was justified. Using horizontal accelerations of 0.10g and 0.15g with a vertical acceleration of zero, the computed FSs are greater than 1.1, which is higher than the minimum FS for seismic slope stability. In summary, the applicant stated, “the Seed reductions are considered reasonable and valid, and the slope is considered to have an adequate factor of safety against failure during the ESP seismic event.”

In RAI 2.5.5-1, the staff asked the applicant to provide its basis for concluding that the existing slope has a low susceptibility to liquefaction and, therefore, concluding that a horizontal acceleration of 0.1g is suitable for the slope stability analysis. In its response, the applicant stated that it revised its previous liquefaction analysis because it is now basing the SSE on the RG 1.165 approach. The applicant’s revised liquefaction analysis (see SSAR Section 2.5.4.8) shows more widespread liquefaction within the Zone IIA saprolitic soils. However, since this analysis does not take into account the age, fabric, structure, and mineralogy of the saprolite, the applicant maintained that any liquefaction would not be widespread. The applicant also defended its use of 0.10g and 0.15g as the peak accelerations for the pseudostatic slope stability analysis. The applicant cited the research of Seed (Ref. 186, SSAR Section 2.5), who concluded that, if embankments do not liquefy or lose a significant amount of strength during a seismic event, they would displace at the crest but typically not fail in the conventional sense. The applicant stated that the design high-frequency earthquake has relatively low energy (magnitude 5.4), and therefore an acceleration of 0.10g is adequate. For the low-frequency earthquake, the applicant used a value of 0.15g for the peak acceleration. The pseudostatic slope stability analyses run with 0.1g and 0.15g both give FS values greater than 1.1.

The applicant also used the pseudostatic approach recommended by Kramer (Ref. 188, SSAR Section 2.5), which uses half of the peak acceleration value rather than a set peak value based on magnitude. Using Kramer’s method, for the high-frequency earthquake, the applicant used a horizontal peak acceleration value of 0.325g and a vertical peak acceleration of 0.1625g. For the low-frequency earthquake, the applicant used a horizontal peak acceleration of 0.13g and a vertical peak acceleration of 0.065g. With these peak acceleration values, the applicant found that the FS is just below 1.0 for the high-frequency ground motion and greater than 1.1 for the low-frequency ground motion. Since the FS is below 1.0 using Kramer's method, the applicant stated that it could not rule of the possibility of some liquefaction in the slope area.

Boring Logs

The applicant drilled two sample borings on or close to the existing 2h:1v slope to the north of the SWR. Figures 2.5-71 and 2.5-72 in the SSAR reproduce the logs of the two borings.

Compacted Fill

SSAR Section 2.5.5.4 states that the existing 2h:1v slope is a cut slope and does not contain fill materials in any significant quantity.

Proposed New Slope

SSAR Section 2.5.5.5 states that a new slope may be excavated to the west of the SWR to accommodate UHSs for the new units. The new slope would be approximately the same height and would have the same 2h:1v slope as the existing slope. In addition, this proposed new slope would comprise similar materials as the existing slope. Therefore, the applicant concluded that the analytical conclusions for the existing slope would apply to the new slope; the new slope would be stable under seismic and long-term static conditions.

Conclusions

In SSAR Section 2.5.5.6, the applicant stated that, based on the possibility of some liquefaction in the slope area (existing slope), as well as the marginal results that it obtained using Kramer's method (Ref. 188, SSAR Section 2.5), it would take measures to ensure the safety of the slope and the structures that may be located close to the bottom of the slope. The applicant stated that these measures could include reducing slope steepness, removing and replacing materials that could lose significant strength during the design earthquake, and ground improvement measures such as soil nailing, moving structures further from the toe of the slope, and/or providing walls/barriers to protect those structures.

2.5.5.2 Regulatory Evaluation

SSAR Section 2.5.5 presents information on the stability of permanent slopes at the ESP site. The applicant stated in SSAR Section 1.8 that it developed the information regarding slope stability in accordance with the guidance presented in Section 2.5.5 of RS-002 and RG 1.70 and that the information is intended to satisfy the requirements of 10 CFR 100.23.

In its review of the application, the staff considered the regulatory requirements in 10 CFR 100.23, which states that the applicant for an ESP must describe the geologic and seismic conditions of the proposed site necessary to determine site suitability. Section 2.5.5 of RS-002 provides specific guidance concerning the evaluation of information characterizing the stability of slopes under SSE conditions.

2.5.5.3 Technical Evaluation

The staff's review of SSAR Section 2.5.5 focused on the applicant's analysis of the stability of an existing slope adjacent to the ESP site, the failure of which might impact future structures located close to the slope. The staff reviewed the applicant's description of the existing slope characteristics, design criteria and analyses, and proposed new slope and design modifications.

2.5.5.3.1 Slope Stability Analysis and Design Criteria

The staff focused its review of SSAR Sections 2.5.5.1 through 2.5.5.6 on the adequacy of the applicant's slope stability analysis of an existing slope adjacent to the ESP site. In addition, the staff reviewed the applicant's summary of the slope subsurface conditions, as well as its proposed new slope and potential design modifications to ensure the safety of the slope and of the structures located close to the bottom of the slope.

To perform the slope stability analysis, the applicant used three different pseudostatic approaches. For the first approach, the applicant used average peak vertical and horizontal acceleration values (0.32g and 0.65g), which resulted in FS less than 1.1. For the second approach, the applicant used the approach recommended by Seed (Ref. 186, SSAR Section 2.5), which recommends peak acceleration values based on the magnitude of the earthquake. Using the Seed approach, the applicant originally used peak vertical and horizontal acceleration values of 0.10g, in accordance with the magnitudes for the controlling earthquakes. With these lower peak accelerations, the resulting FS were greater than 1.1, which is the minimum FS acceptable for seismic slope stability. In RAI 2.5.5-1, the staff asked the applicant to provide its basis for concluding that a horizontal acceleration of 0.1g is suitable for the slope stability analysis. In response to RAI 2.5.5-1, the applicant stated that it revised the peak horizontal acceleration value to 0.15g, since the controlling earthquake using the RG 1.165 approach has a magnitude of 7.2. The pseudostatic slope stability analyses run with 0.10g and 0.15g both give FS values greater than 1.1. For the third pseudostatic approach, the applicant used the peak acceleration values recommended by Kramer (Ref. 188, SSAR Section 2.5), which are half of the average peak acceleration values (0.16g and 0.33g). Using these values the FS is below 1.0 for the high frequency controlling earthquake, implying the possibility of some liquefaction in the slope area.

The applicant concluded its response to RAI 2.5.5-1 by stating, "in recognition of the high near-surface accelerations and the results of the liquefaction analysis, the SSAR will be revised to indicate measures that would be taken to ensure the safety of the slope and of the structures that may be located close to the bottom of the slope." The staff concurs with this decision, since two of the three pseudostatic liquefaction analysis approaches result in FS less than 1.1. The staff concludes that, for the purposes of the ESP application, the pseudostatic analyses used by the applicant are adequate to analyze the stability of the existing slope. However, because the Zone IIA saprolites are susceptible to liquefaction, and because the existing slope could change, depending on final plant design and layout, the staff concludes that the COL or CP applicant should conduct a more detailed dynamic analysis of the stability of the existing slope and any new slopes resulting from plant construction using the SSE ground motion. This is **COL Action Item 2.5-10**.

2.5.5.4 Conclusions

Based on its review of SSAR Section 2.5.5 and the applicant's response to RAI 2.5.5-1, described above, the staff concludes that the applicant sufficiently analyzed the stability of the existing slope for the purposes of the ESP application. Because of the susceptibility of the Zone IIA saprolites to liquefaction, the staff concludes that the COL or CP applicant should conduct a more detailed dynamic analysis of the stability of the existing slope and any new slopes using the SSE ground motion. This is **COL Action Item 2.5-10.** A more extensive dynamic analysis would be appropriate at the COL or CP stage, since the applicant will have determined the locations of safety-related structures relative to the existing or new slopes. In addition, the COL or CP applicant should provide plot plans and cross-sections/profiles of all of the safety-related slopes and should specify the measures that it will take to ensure the safety of the slopes and any structures located adjacent to the slopes. This is **COL Action Item 2.5-11.**

2.5.6 Embankments and Dams

2.5.6.1 Technical Information in the Application

In SSAR Section 2.5.6, the applicant stated that, since Lake Anna would only be used for normal (i.e., non-safety-related) plant cooling of the new units, it did not reanalyze the North Anna Dam as part of the ESP application. According to the applicant, the North Anna Dam was designed and constructed to meet the requirements for a seismic Category I structure in support of the existing NAPS units.

2.5.6.2 Regulatory Evaluation

SSAR Section 2.5.6 states that the applicant did not reanalyze the North Anna Dam since Lake Anna would only be used for normal plant cooling of the new units. As such, the applicant did not list any regulatory guidance or cite any regulations as applicable to SSAR Section 2.5.6.

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. Sections 2.4.4 and 2.5.5 in RS-002 provide similar information and guidance.

2.5.6.3 Technical Evaluation

Section 2.4.4 of this SER provides the staff's evaluation of potential dam failures; Section 2.5.5 of this SER provides its evaluation of slope stability.

2.5.6.4 Conclusions

Sections 2.4.4 and 2.5.5 of this SER present the staff's conclusions regarding dam failures and slope stability, respectively.

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