2.5.4 Stability of Subsurface Materials and Foundations

SSAR Section 2.5.4 presents the applicant's evaluation of the stability of subsurface materials that underlie the VEGP ESP site. There are a total of 12 subsections within SSAR Section 2.5.4. Among them, Section 2.5.4.1 does not contain any specific details, but refers to SSAR Sections 2.5.1.1 and 2.5.1.2 regarding regional (within a 320-kilometer (km) (200-mile (mi)) radius) and site area (within an 8-km (5-mi) radius) geology and tectonics for the ESP site. Section 2.5.4.9 cross-references SSAR Sections 2.5.2.6, 2.5.2.7, and 2.5.2.8 for determination of the SSE and operating-basis earthquake (OBE). Section 2.5.4.2 describes the engineering properties of the subsurface materials. Section 2.5.4.3 describes subsurface explorations performed at the ESP site, and Section 2.5.4.4 describes geophysical surveys performed by the applicant to determine shear and compressional wave velocities of the soil and rocks beneath the ESP site. Section 2.5.4.5 describes excavation and backfill work for the ESP site. Section 2.5.4.6 describes local ground water conditions. Section 2.5.4.7 describes site-specific shear wave velocity profile, shear modulus, and damping. Section 2.5.4.8 describes the applicant's evaluation of liquefaction potential, and Section 2.5.4.10 describes the bearing capacity and settlement analysis for the ESP site. Section 2.5.4.11 describes limitations in design values, such as settlements and factors of safety, and Section 2.5.4.12 describes briefly the methods to be used to improve the subsurface conditions at the ESP site.

This section describes the applicant's geotechnical investigation at the ESP site. In summary, the applicant conducted 14 exploratory borings, performed 10 CPTs, including 3 down-hole seismic CPTs, and suspension P-S velocity logging in 5 boreholes. The applicant also performed laboratory testing on grain size (61 tests), unit weight (31), natural moisture content (75), and Atterberg limits (27), and conducted 15 unconsolidated undrained triaxial tests. The applicant installed 15 ground water observation wells.

The applicant described the main load-bearing layers and their geotechnical properties at the ESP site. From top to bottom, they are the Upper Sand Stratum (Barnwell Group), Blue Bluff Marl (Lisbon Formation), and Lower Sand Stratum (Still Branch Formation), overlying the Dunbarton Triassic basin rock/Paleozoic rock.

- the Upper Sand Stratum is predominately sands, silty sands with occasional clayey seams.
- the Blue Bluff Marl is slightly sandy, cemented calcareous clay.
- the Lower Sand Stratum comprises fine-to-course sand with interbedded silty clay and clayey silt.

The applicant stated that it will replace the Upper Sand Stratum with engineering compacted fills. The applicant established the site-specific shear wave velocity profile and soil degradation and damping curves that feed into the site response study to obtain the site-specific GRMS. The applicant also proposed the settlement guidelines and derived key engineering parameters using the data from previous investigations for the existing VEGP Units 1 and 2. In addition, the applicant concluded that the Blue Bluff Marl is not liquefiable irrespective of ground motion level.

2.5.4.1 Technical Information in the Application

2.5.4.1.1 Description of Site Geologic Features

SSAR Section 2.5.4.1 refers to SSAR Sections 2.5.1.1 and 2.5.1.2 for the description of geologic and tectonic features within the ESP site region (320-km (200-mi) radius) and within the immediate ESP site area (8-km (5-mi) radius).

2.5.4.1.2 Properties of Subsurface Materials

SSAR Sections 2.5.4.2.2 and 2.5.4.2.3 describe in detail the engineering properties of the subsurface materials at the ESP site. The applicant stated that soils encountered during the ESP subsurface investigation constitute alluvial and Coastal Plain deposits, which include the Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum, from top to bottom. Figure 2.5.4-1 of this SER illustrates the relative position and thickness of these subsurface strata. Dunbarton Triassic (206 to 24 million years ago (mya)) basin rock and Paleozoic (543 to 248 mya) crystalline rock underlie these soil layers at the ESP site.

The applicant performed laboratory testing for the ESP investigation in accordance with RG 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants," Revision 2, issued December 2003. The applicant stated that these ESP tests focused primarily on verifying basic properties of the Upper Sand Stratum and the Blue Bluff Marl, as well as the upper portion of the Lower Sand Stratum. The applicant also performed laboratory testing on grain size, unit weight, natural moisture content, and Atterberg limits and conducted unconsolidated undrained triaxial tests for the ESP site. The applicant concluded that the engineering properties obtained from the ESP subsurface investigation and laboratory testing program were similar to those obtained from the previous field and laboratory testing programs for VEGP Units 1 and 2. SER Table 2.5.4-1 summarizes the geotechnical features of the five major geologic units and their corresponding engineering properties obtained through the ESP investigations.

The applicant also listed engineering properties and design values from the previous VEGP Units 1 and 2 investigation for the Upper Sand Stratum, compacted fills, Blue Bluff Marl, and Lower Sand Stratum as summarized in SER Table 2.5.4-2. The applicant stated that the engineering properties obtained from the ESP investigations were similar to those obtained from previous investigations. However, the applicant also stated that it will perform additional confirmatory tests during the COL application phase to support, for example, the use of a 478.9 kilopascal (kPa) (10,000 pounds per square foot (psf)) design shear strength value because it differs significantly from the values obtained during the ESP investigation. The applicant indicated in SSAR Section 2.5.4.2.3 that it referred to previous borings, but did not include them in the SSAR. Finally, the applicant stated that it did not include chemical tests in the ESP laboratory testing program, but that it would conduct such chemical tests as part of the COL investigation because of the backfill materials placed in the proximity of planned concrete foundations and buried metal pipes.

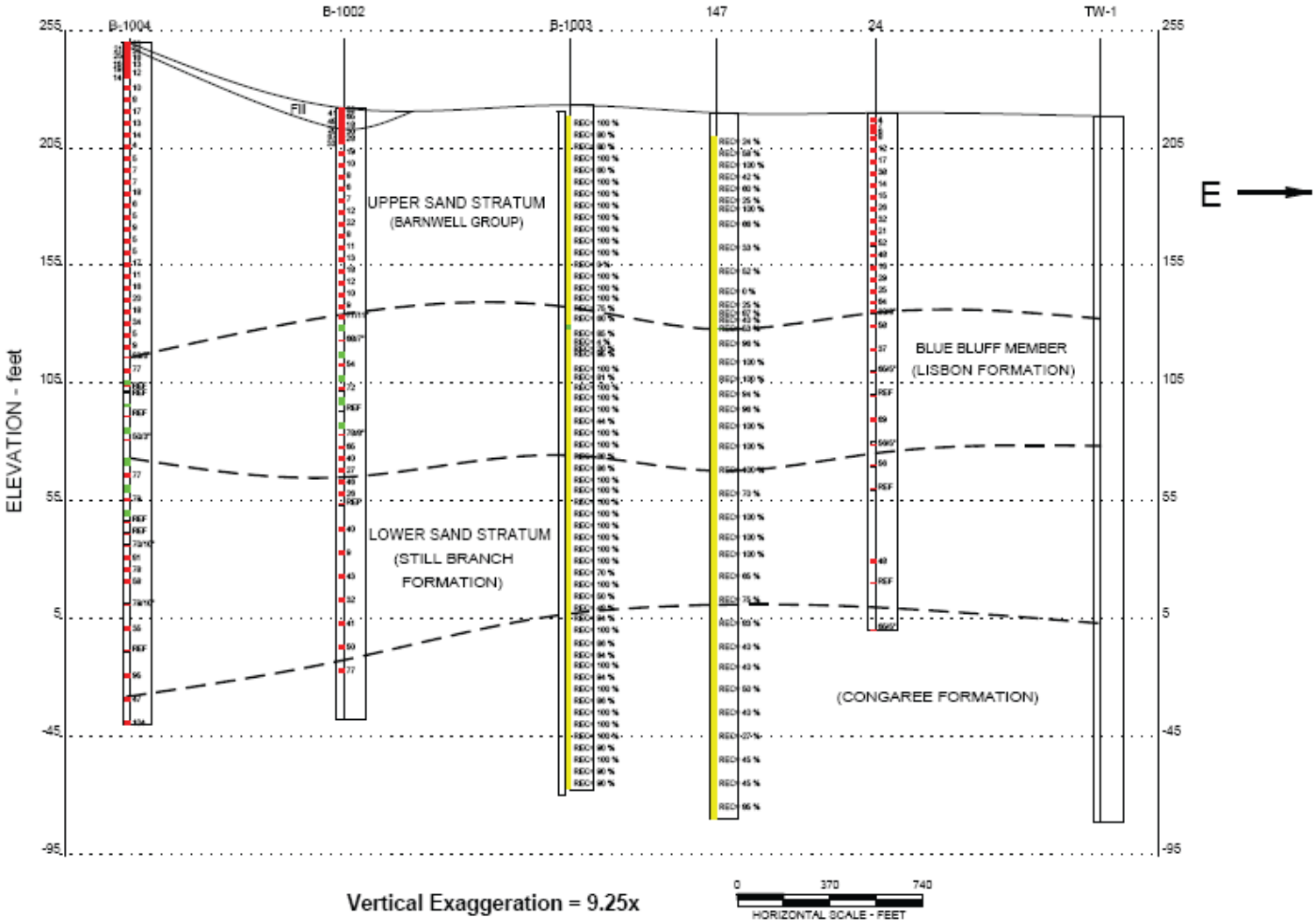


Figure 2.5.4-1 - Subsurface Profile (Reproduced from SSAR Figure 2.5.4-3, position of the profile shown in Figure 2.5.4-2 from A to A')

Table 2.5.4-1 - Engineering Properties of Subsurface Materials Based on ESP Site Investigation

Properties	Upper Sand Stratum (Barnwell Group)	Blue Bluff Marl (Lisbon Formation)	Lower Sand Stratum	Dunbarton Triassic Basin Rock	Paleozoic Crystalline Rock
General Description	Predominantly sands, silty sands, and clayey sands with occasional clay seams. A shelly limestone (Utley Limestone) layer was encountered at the interface of the Upper Sand Stratum and the Marl Bearing Stratum. The limestone contains solution channels, cracks, and discontinuities and was the cause of severe fluid loss observed.	Slightly sandy, cemented, calcareous clay and the upper 4.6 m (15 ft) is likely weathered.	Fine-to-coarse sand with interbedded silty clay and clayey silt.	Red sandstone, breccia, and mudstone, weathered through the upper 36.6 m (120 ft).	Competent rock with high shear wave velocities that underlies the noncapable Pen Branch Fault.
USCS symbol	SP/SM/SC/ML	SP/SM/SC	SP/SM/ML	N/A	N/A
Layer thickness	23.8 to 47.9 m (78 to 157 ft)	19.2 to 29.0 m (63 to 95 ft)	274.3 to 304.8 m (900 to 1000 ft)	N/A	N/A
Percent fines	8 to 78% Avg. 35%	24 to 77% Avg. 40%	3 to 79% Avg. 21%	N/A	N/A
Moisture content	20 to 90% Avg. 63%	14 to 67 % Avg. 35%	21 to 41% Avg. 30%	N/A	N/A
Unit weight	No measurement.	1659 to 2246 kg/m^2 (103.6 to 140.2 pcf) Avg. 1922 kg/m^2 (120 pcf)	1913, 1949, and 2055 kg/m^2 (119.4, 121.7, and 128.3 pcf) Avg. 1972 kg/m^2 (123.1 pcf)	N/A	N/A

Properties	Upper Sand Stratum (Barnwell Group)	Blue Bluff Marl (Lisbon Formation)	Lower Sand Stratum	Dunbarton Triassic Basin Rock	Paleozoic Crystalline Rock
Plastic Index (PI)	21 to 67% Avg. 37%	NP to 58 % Avg. 22%	NP to 19% Avg. 17%	N/A	N/A
Measured SPT N-Value (median energy transfer efficiency 0.75)	From weight of rod to 50 blows for 0-in. penetration (50/0”) Avg. 21 bpf	From 26 bpf to 50 blows for 1-in. penetration (50/1”) Avg. 83 bpf	From 9 bpf to 50 blows for 4-in. penetration (50/4”) Avg. 59 bpf	N/A	N/A
Shear wave velocity	173.7 to 1008.9 m/s (570 to 3310 fps)	323.1 to 1298.4 m/s (1060 to 4260 fps)	283.5 to 1423.4 m/s (930 to 4670 fps)	707.1 to 2849.9 m/s (2320 to 9350 fps)	N/A
Compressional wave velocity	396.2 to 2426.2 m/s (1300 to 7960 fps)	1414.3 to 2996.2 m/s (4640 to 9830 fps)	1521.0 to 2752.3 m/s (4990 to 9030 fps)	2225.0 to 5596.1 m/s (7,300 to 18,360 fps)	N/A
Possion’s ratio (calculated)	0.09 to 0.49	0.33 to 0.48	0.32 to 0.49	0.10 to 0.46	N/A
Strength Properties					
Internal friction angle	34 degree (determined based on N-values)	0	41 degree (determined based on N-values)	N/A	N/A
Undrained shear strength	No measurement	7.18 to 205.94 kPa (150 to 4300 psf)	No measurement	N/A	N/A
Elastic modulus	137.9 to 1378.9 MPa (0.2×10^5 to 2.0×10^5 psi)	1585.8 Mpa (2.3×10^5 psi)	No measurement	N/A	N/A
Shear modulus	55.2 to 468.8 MPa (0.8×10^4 to 6.8×10^4 psi)	551.6 Mpa (8.0×10^4 psi)	No measurement	N/A	N/A

Note: The parameters listed in this table are based on ESP site investigation and they are different from the design values listed in SSAR Table 2.5.4-1, which are mostly based on data from the previous investigation.

Table 2.5.4-2 - (SSAR Table 2.5.4-1) Static Engineering Properties of Subsurface Materials

Parameter[1]	Stratum			
	Upper Sand	Compacted Structural Fill	Blue Bluff Marl	Lower Sand
Depth range below El. 220 ft, feet	79 to 124	79 to 124	63 to 95	900
Average thickness, feet	92	92	76	900
USCS symbol	SP/SM/SC/ML	SP/SM/SC	CL/ML	SP/SM/ML
Natural moisture content (ω), %	N/A	N/A	35	N/A
Unit weight (pcf)	115	123 (moist) 133 (saturated)	115	115
Atterberg limits				
Liquid limit (LL), %	N/A[2]	N/A	51	N/A
Plastic limit (PL), %	N/A	N/A	26	N/A
Plasticity index (PI), %	N/A	N/A	25	N/A
Measured SPT N-value, bpf	20	N/A	80	50
Adjusted SPT N_{60}-value, bpf	25	N/A	100	62
Strength properties				
Undrained shear strength (c_u), ksf	-	0	10	0
Internal friction angle (Ø'), degrees	34	34	0	34
Elastic modulus (high strain) (E_s), ksf	900	1,500	10,000	10,800[3] 13,500[4]
Shear modulus (high strain) (G_s), ksf	350	600	3,500	4,200[3] 5,200[4]
Shear modulus (low strain) (G_{max}), ksf	3088	3820	20,475	20,538
Coefficient of Subgrade Reaction (k_1), tcf	N/A	300	N/A	N/A
Earth Pressure Coefficients				
Active (K_a)	N/A	0.3	N/A	N/A
Passive (K_p)	N/A	3.5	N/A	N/A
At Rest (K_o)	N/A	0.5	N/A	N/A
Coefficient of Sliding	N/A	0.45	N/A	N/A
Poison's Ratio	0.09-0.49		0.33-0.48	0.32-0.49

Notes.
[1]The values tabulated above are for use as a design guideline only. Reference should be made to specific boring and CPT logs and laboratory test results for appropriate modifications at specific design locations.
[2]N/A indicates that the properties were not measured or are not applicable.
[3]This value applies between depth of 0 to 100 ft below the bottom of the Blue Bluff Marl.
[4]This value applies between depth of 100 to 300 ft below the bottom of the Blue Bluff Marl.
Engineering properties for the Dunbarton Triassic Basin are not included because the rock is too deep to be of interest for foundation design.
Dynamic properties, including those for the Dunbarton Triassic Basin, can be derived from the shear wave velocity profile shown on Table 2.5.4-10.

2.5.4.1.3 Site Exploration

In SSAR Section 2.5.4.3, the applicant summarized subsurface investigation programs for both the existing VEGP Units 1 and 2 and for the ESP site. The applicant stated that the field investigations for VEGP Units 1 and 2 started in 1971. At the time, the field investigations included a total of 585 borings and 18,288 meters (60,000 ft) of drilling, as well as 8,656 meters (28,400 ft) of shallow refraction lines and 1,524 meters (5,000 ft) of deep refraction lines. The applicant deployed various investigation methods, including electric logging, natural gamma, density, neutron, caliper, three-dimensional velocity logs, and cross-hole velocities of the subsurface (to a depth of 88.4 meters (290 ft)). The applicant emphasized that it used the results from previous investigations to supplement subsurface data obtained during the ESP investigation.

The ESP investigation started in September 2005. The applicant performed the subsurface investigation over the area that would house the prospective reactors and switchyard, as well as the cooling towers. The applicant stated that this investigation consisted of exploration points that were selected primarily to confirm the results obtained from previous extensive investigations implemented for VEGP Units 1 and 2. In addition, the applicant stated that it will perform additional structure-specific explorations at the COL phase. The applicant also emphasized that it followed the guidelines in RG 1.132, Revision 2, issued October 2003. SER Figure 2.5.4-2 shows the locations of the borings for the ESP subsurface investigation.

In summary, the applicant conducted 14 exploratory borings with depths ranging from 27.4 meters (90 ft) to 407.8 meters (1338 ft), but only sampled 12 of them. The applicant also performed 10 CPTs, including 3 down-hole seismic CPTs, and suspension P-S velocity logging in 5 of the boreholes. In addition, the applicant implemented seismic reflection and refraction surveys in order to collect data to delineate the Pen Branch fault, discussed in detail in SSAR Sections 2.5.2.4 and 2.5.3.

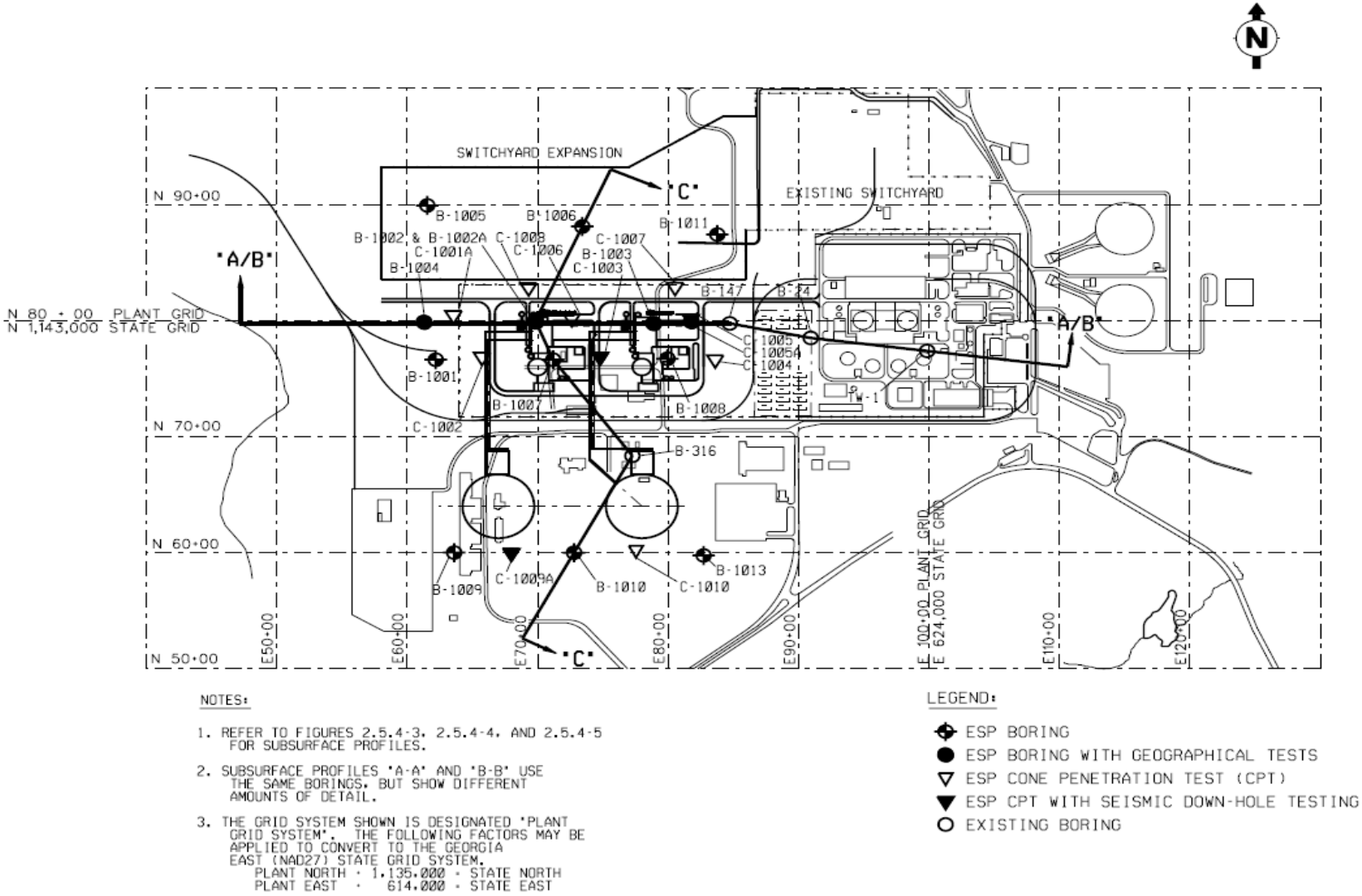
Boring and Sampling

The applicant stated that it sampled 13 borings (excluding boring B-1003) using a standard penetration test (SPT) sampler at continuous intervals to a 4.6-meter (m) (15-foot (ft)) depth and then at 1.5- to 3.0-m (5- or 10-ft) intervals below that depth, and that it operated the SPT in accordance with American Society for Testing and Materials (ASTM) D 1586 (1999), "Standard Test Method for Penetration Resistance and Split Barrel Sampling of Soil." The applicant recovered soil samples in accordance with ASTM D 2488 (2000), "Standard Practice for Description and Identification of Soils." The applicant performed a continuous core boring for boring B-1003 with an average core recovery rate of 77 percent.

CPT

The applicant described that it performed CPT testing at the ESP site, in accordance with ASTM D 5578 (2000), "Standard Test Method of Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils." The applicant also stated that it advanced each of the CPTs to refusals at depths ranging from 1.8 to 35.6 meters (6 to 116.7 ft). Most CPT locations met refusals at or near the top of the Blue Bluff Marl.

The applicant performed pore pressure dissipation tests at a depth of 20.7 and 24.1 meters (68 ft and 79 ft) in C-1003; 20.0 m (66-ft) depth in C-1004; 17.1 m (56 ft), 22.3 m (73 ft), and 25.0 m (82 ft) depths in C-1005; and 18.3 m (60 ft), 23.5 m (77 ft), 27.4 m (90 ft), and 30.2 m (99 ft) depths in C-1009A.



NOTES:

1. REFER TO FIGURES 2.5.4-3, 2.5.4-4, AND 2.5.4-5 FOR SUBSURFACE PROFILES.
2. SUBSURFACE PROFILES "A-A" AND "B-B" USE THE SAME BORINGS, BUT SHOW DIFFERENT AMOUNTS OF DETAIL.
3. THE GRID SYSTEM SHOWN IS DESIGNATED "PLANT GRID SYSTEM". THE FOLLOWING FACTORS MAY BE APPLIED TO CONVERT TO THE GEORGIA EAST (NAD27) STATE GRID SYSTEM.
   PLANT NORTH - 1,135,000 = STATE NORTH
   PLANT EAST - 614,000 = STATE EAST

LEGEND:

- ESP BORING
- ESP BORING WITH GEOGRAPHICAL TESTS
- ESP CONE PENETRATION TEST (CPT)
- ESP CPT WITH SEISMIC DOWN-HOLE TESTING
- EXISTING BORING

Figure 2.5.4-2 - ESP Boring Location (Reproduced from SSAR Figure 2.5.4-1)

In Situ Hydraulic Conductivity Testing

The applicant stated that it installed 15 observation wells during May and June 2005 and added another replacement well in October 2005. The applicant developed the wells by pumping and then performed permeability tests at each well in accordance with Section 8 of ASTM D 4044 (2002), "Standard Test (Field Procedure) for Instantaneous Change in Head (slug) Tests for Determining Hydraulic Properties of Aquifer."

2.5.4.1.4 Geophysical Surveys

SSAR Section 2.5.4.4 summarizes subsurface geophysical surveys performed for the existing VEGP Units 1 and 2 and for the ESP units at the VEGP site. SSAR Section 2.5.4.4.1 describes the field investigations that began in 1971 for the existing Units 1 and 2, including seismic refraction and cross-hole surveys. SSAR Section 2.5.4.4.2 describes three down-hole seismic CPT tests and five suspension P-S velocity tests performed by the applicant for the proposed VEGP Units 3 and 4. The following SER sections provide additional information relating to each of these surveys.

Previous Geophysical Investigations

The applicant described previous geophysical investigations performed for the existing VEGP Units 1 and 2. The geophysical methods used in previous investigations were seismic refraction and cross-hole surveys. The seismic refraction survey used compressive wave velocities to determine seismic discontinuities. The cross-hole seismic survey provided shear and compressional wave velocities for the subsurface materials to a depth of 88.4 meters (290 ft), 25.0 meters (82 ft) above mean sea level (msl).

ESP Site Geophysical Investigations

During the ESP investigation, the applicant conducted three down-hole seismic CPT tests and five suspension P-S velocity tests. The applicant also stated that it performed seismic reflection and refraction surveys to image the subsurface and to characterize the basement rocks across the ESP site.

The applicant obtained the shear wave and compressional wave velocity to the maximum explored depth of 407.8 meters (1338 ft). The Upper Sand Stratum shear wave velocities range from 179.8 to 1005.8 meters per second (m/s) (590 to 3300 ft per second (ft/s)), however, SSAR Section 2.5.4.2.5.2 gives different values from 173.7 to 1008.9 m/s (570 to 3310 ft/s), with an average value of 331.9 m/s (1089 ft/s). The Blue Bluff Marl shear wave velocities range from 323.1 to 1298.4 m/s (1060 to 4260 ft/s), with an average value of 772.3 m/s (2534 ft/s), and Lower Sand Stratum shear wave velocities range from 283.5 to 1423.4 m/s (930 to 4670 ft/s), with an average value of 696.0 m/s (2282 ft/s). SER Figure 2.5.4-3 shows the shear wave velocity values measured in the subsurface strata at the ESP site using suspension P-S velocity and CPT down-hole seismic testing. Shear wave velocities for the Dunbarton Triassic (206 to 24 mya) basin rock ranged from 707.2 to 2850 m/s (2320 to 9350 ft/s). In the upper part of the Dunbarton Triassic basin, the applicant detected a weathered zone about 36.6 meters (120 ft) thick, where shear wave velocity increased rapidly

with depth. The applicant did not directly detect the rock with a shear wave velocity of 2.8 km/s (9200 ft/s), but extrapolated linearly to the corresponding rock horizon.

The applicant stated that the shear wave velocity was measured through down-hole seismic tests, but that CPT measurements are limited to the Upper Sand Stratum because all CPTs reached refusal at the top of the Blue Bluff Marl. The shear wave velocity values obtained from CPT measurements are lower than those obtained from suspension P-S velocity tests, which may reflect site variability.

The applicant indicated that it used shear wave velocities measured from both CPTs and suspension logging to develop the design values for the Upper Sand Stratum, but only used shear wave velocities from suspension logging for the Blue Bluff Marl and Lower Sand Stratum. The applicant also stated that it will use shear wave velocities obtained for the compacted fill for the existing VEGP Units 1 and 2.

2.5.4.1.5 Excavation and Backfill

In SSAR Section 2.5.4.5, the applicant discussed future excavation, fills and slopes (for stability), excavation methods and stability issues, back fill sources and quality control, and the impact of dewatering. The applicant stated in SSAR Section 2.5.4.5.1 that a substantial amount of excavation to completely remove the Upper Sand Stratum and filling is warranted for construction purposes at the proposed VEGP Units 3 and 4. Additional planning for excavation and filling will take place during the COL phase. SSAR Section 2.5.4.5.2 describes the applicant's excavation methods that include excavating the Upper Sand Stratum using conventional excavation equipment and dewatering once the excavation reaches depths beneath the ground water table. SSAR Section 2.5.4.5.3 describes backfill methods that the applicant plans to follow at the proposed VEGP Units 3 and 4. These methods are similar to the guidelines used during construction of the existing Units 1 and 2. The applicant also stated that it will establish an onsite soils testing laboratory and will use a contractor with an approved quality control program. SSAR Section 2.5.4.5.4 refers to SSAR Section 2.5.4.6.2 for a discussion on the control of ground water and dewatering during the excavation phase.

Extent of Excavation

The applicant indicated that the plant grade will be at 67.1 meters (220 ft) above msl, and the base for the containment and auxiliary building foundations will be at 54.9 meters (180 ft) above msl, which corresponds to a depth of 12.2 meters (40 ft) below the final grade or approximately 15.2–18.3 meters (50–60 ft) above the Blue Bluff Marl. Other foundations in the power block will be at nominal depths near final grade.

The applicant stated that it will completely remove the Upper Sand Stratum during excavation. The total depth of excavation to the Blue Bluff Marl will range from approximately 24.4 to 27.4 meters (80 to 90 ft) below existing grade, based on the borings completed during the ESP subsurface investigation. The applicant also indicated that it may need to remove shelly and porous material near the top surface of the Blue Bluff Marl using deeper, localized excavations and that it will develop a detailed excavation plan during the COL stage.

Excavation Methods and Stability

The applicant indicated that it will use conventional excavating equipment to remove the Upper Sand Stratum and will follow the Occupational Safety and Health Administration's regulations during the excavation. The excavation will be open-cut with slopes no steeper than 2:1 (horizontal-to-vertical ratio). The applicant stated that it will seal and protect each temporary slope cut into the Upper Sand Stratum and will use sheet piles or soldier piles and lagging walls where there is insufficient space for open-cut excavation. The applicant will implement a dewatering plan once the excavation progresses beneath the ground water table, approximately 50.3 meters (165 ft) above msl. The applicant also committed that it will remove possible soft zones in the upper portion of the Blue Bluff Marl with conventional equipment, maintain proper slopes for the compacted fill, and clean up the excavation area thoroughly before the placement of structural fill.

Backfill Sources and Quality Control

The applicant indicated that compacted fill, to be used for the proposed Units 3 and 4, will be composed of sandy or silty sand material that contains no more than 25 percent fines (related to particle size) that are smaller than a No. 200 sieve. These guidelines proposed by the applicant follow those used during the backfill phase for the existing Units 1 and 2. The applicant stated that it will evaluate old borrows from previous construction, as well as new borrows if warranted, for potential backfill source material. In addition, the applicant stated that it will compact the fills, using the VEGP criteria (defined in ASTM D 1557, "Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56000 ft-lbf/ft^2 (2700 kN-m/m^3)") to within 3 percent of its optimum moisture content. In addition, the applicant will develop a test fill program similar to the one used for VEGP Units 1 and 2. The applicant will also establish an onsite testing laboratory to control the quality of the fill materials and the degree of compaction, as well as to ensure that the fill conforms to the requirement of the earthwork specification. The applicant will deploy a soil testing firm (other than the earthwork contracting firm) that has an established quality control program. Finally, the applicant stated that it will implement sufficient laboratory compaction and grain size distribution tests to ensure that variations in the fill material are considered, and it will perform sufficient density tests to ensure a minimum test per lift of one per 929.0 square meters (10,000 square ft) of fill placed.

2.5.4.1.6 Ground Water Conditions

SSAR Section 2.5.4.6 describes the applicant's ground water measurements and construction dewatering plan. SSAR Section 2.5.4.6.1 presents a summary of the ground water conditions at the ESP site and references SSAR Section 2.4.12 for a detailed discussion of those conditions. SSAR Section 2.5.4.6.2 discusses the construction dewatering methods used for the existing VEGP Units 1 and 2. The applicant plans to implement similar procedures during excavation for the proposed Units 3 and 4.

Ground Water Measurements and Elevations

Ground water is in unconfined conditions in the Upper Sand Stratum and in confined conditions in the Lower Sand Stratum at the ESP site. The Blue Bluff Marl is an aquaclude that separates

the unconfined and confined aquifers. The ground water table occurs at a depth of about 18.3 meters (60 ft) below the existing ground surface.

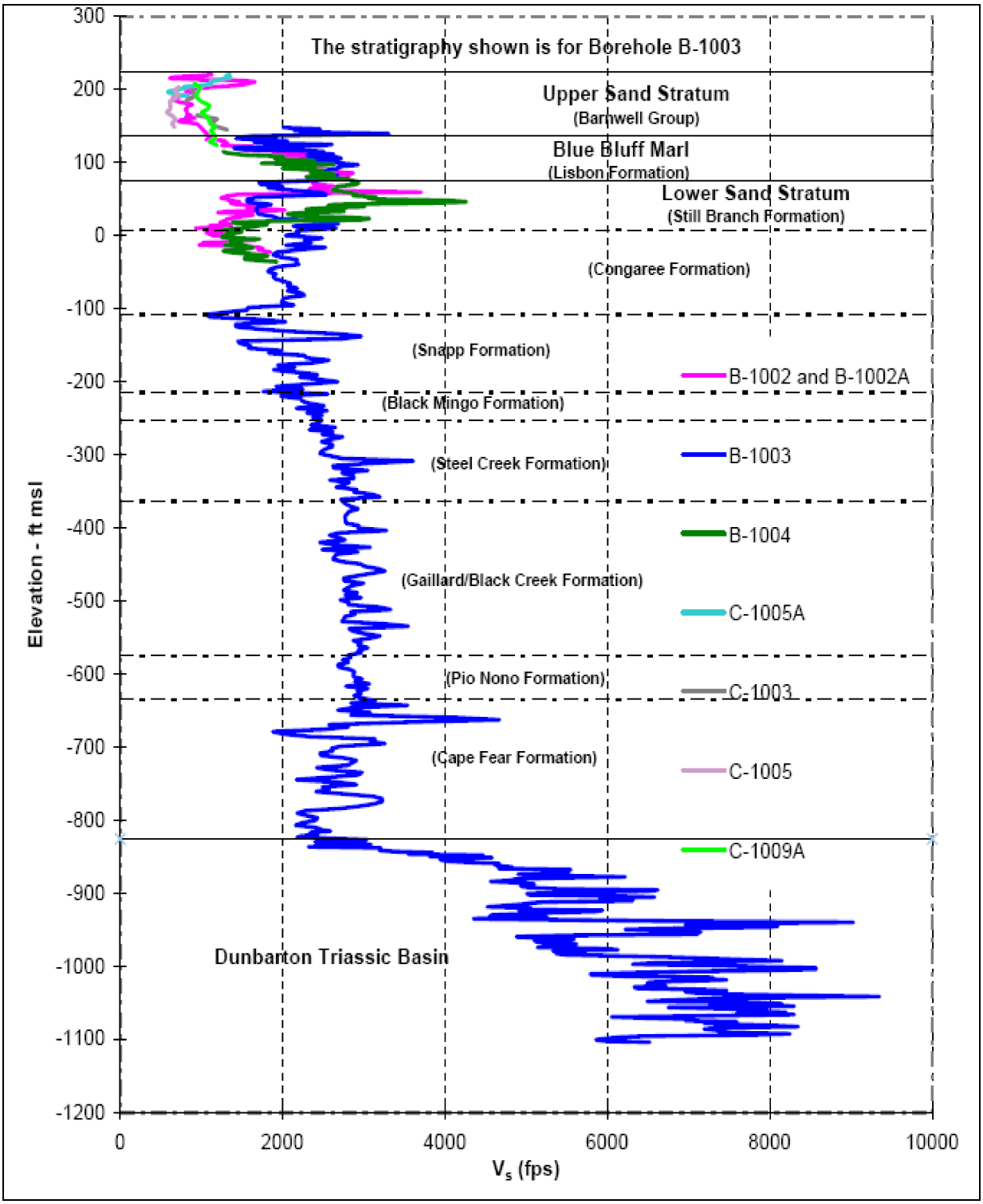


Figure 2.5.4-3 - ESP Site Shear Wave Velocity Measurements

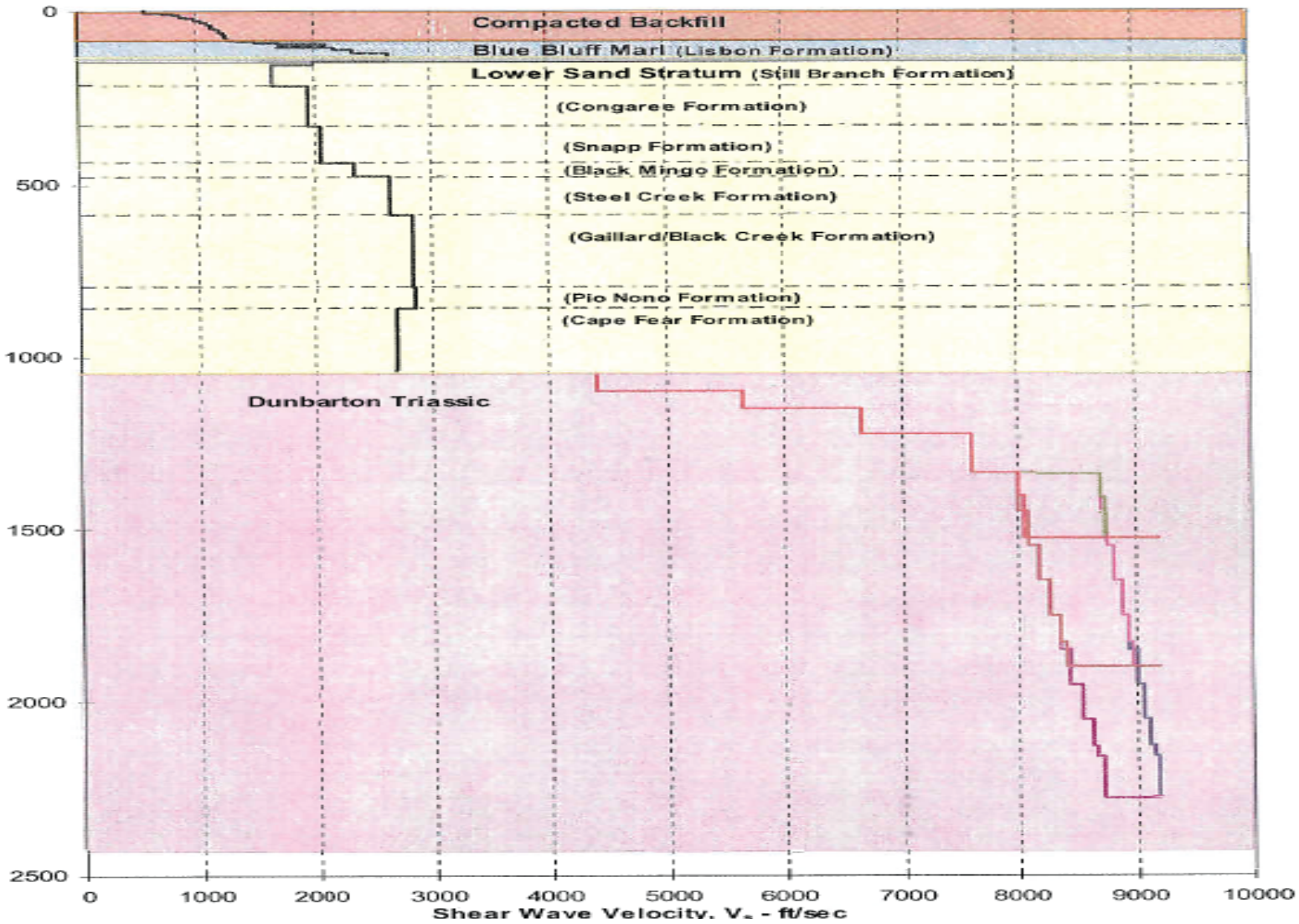


Figure 2.5.4-4 - ESP Site-Specific Base Shear Wave Velocity Profile

The applicant stated, in SSAR Section 2.5.4.6.1, that it installed 15 observation wells for its ESP subsurface investigation program, 10 wells in the unconfined aquifer and 5 wells in the confined aquifer. In addition to these wells, 22 existing wells (13 in unconfined aquifers and 9 in confined aquifers) were used as part of the ground water monitoring program for this ESP study. The applicant indicated that the wells exhibited ground water levels ranging from about 39.3 to 51.0 meters (128.2 to 167.2 ft) above msl during their installation The applicant also noted that it replaced one of the observation wells, but did not mention its reasoning for doing so. The applicant stated that the hydraulic conductivity (k-value) for the unconfined aquifer ranges from 1.13×10^{-5} to 9.34×10^{-4} centimeters per second (cm/s) and that the k-value for the confined aquifer ranges from 1.25×10^{-5} to 7.49×10^{-4} cm/s. The applicant referred to SSAR Section 2.4.12 for a more detailed description of ground water conditions at the ESP site.

Construction Dewatering Plan

The applicant stated that the design ground water level for the ESP site will be 50.3 meters (165 ft) above msl, based on the 10 years of ground water observations made prior to the ESP investigation and on the ground water monitoring program observations made for this ESP investigation. This ground water level (50.3 meters (165 ft) above msl) corresponds to the design ground water level for the existing VEGP Units 1 and 2. The applicant stated that it will dewater all of the major excavations using gravity systems. The applicant believed that, due to the relatively impermeable nature of the Upper Sand Stratum, sump-pumping of ditches will be adequate to dewater the soils. The applicant stated that it will use the same kind of ditch layout and pump numbers as used in the Unit 1 and 2 construction to remove ground water inflows. In addition, the applicant emphasized that it will dewater during new construction in a manner that minimizes the draw-down effects on the surrounding environment and on Units 1 and 2.

2.5.4.1.7 Response of Soil and Rock to Dynamic Loading

SSAR Section 2.5.4.7 describes the development of the site-specific base shear wave velocity profile, variations of the shear modulus and soil damping values with shear strain, and soil/rock column amplification analysis. SSAR Section 2.5.4.7.1.1 describes various measurements that were made at the ESP site to obtain estimates of the shear wave velocity for the soil, while SSAR Section 2.5.4.7.1.2 describes the shear wave velocities measured for the Dunbarton Triassic basin and Paleozoic crystalline rocks. SSAR Section 2.5.4.7.2.1 describes the variation of soil shear modulus with shear strain, and SSAR Section 2.5.4.7.2.2 addresses the variation of soil damping with cyclic shear strain. SSAR Section 2.5.4.7.3 discusses the site dynamic responses for the soil/rock profiles described by the applicant in SSAR Section 2.5.4.7.1. The applicant computed these responses in the frequency domain by using a complex response method and the SHAKE2000 computer program. The applicant presented the following information related to the response of soil and rock to dynamic loading.

Shear Wave Velocity Profile

The applicant stated that it used previous shear wave velocity determined for the existing units for the compacted fill at the ESP site and that it will do additional evaluation during the COL stage to confirm its shear wave velocities. Figure 2.5.4-3 shows the site-specific shear wave velocity profile based on the ESP investigation. The applicant indicated that Figure 2.5.4-4 (SSAR Figure 2.5.4-7) shows the profile used in the site seismic amplification analysis.

The applicant characterized the shear wave velocity beneath the soil (Upper Sand Stratum) with the following descriptions:

1. A weathered zone (about 200 ft in thickness) occurs at the top of the Dunbarton Triassic basin rocks and is characterized by a steep shear wave velocity gradient.

2. The shear wave velocity increases with a gentle gradient within the unweathered rocks.

3. Based on SRS data, the shear wave velocity profile of the deep boring B-1003 can extend deeper with a range of gentle gradients and a range of shear wave velocities.

4. Shear wave velocity for the Dunbarton Triassic basin rocks will likely not reach 9200 ft/s.

5. Shear wave velocity of the Paleozoic rock is at least 9200 ft/s.

The applicant indicated that the Pen Branch fault separates the Dunbarton Triassic basin from the Paleozoic crystalline rocks. In order to represent the variability in depth where the Paleozoic rocks are encountered, the uncertainty of the shear wave velocity gradient, and the velocity at the top of the unweathered Dunbarton Triassic basin rocks, the applicant considered six profiles, instead of one, to comprise the base case used in the site response analysis, as shown in SER Figure 2.5.4-4. To further account for potential variation due to the geometry of the fault, the applicant also performed sensitivity tests to verify that other variations in the site response are insignificant with respect to the six profiles considered.

Variation of Shear Modulus and Damping with Shear Strain

The applicant stated that it determined the shear modulus for various soil strata based on the unit weights and shear wave velocity of those strata, using Equation (1) below:

$$G_{max} = r(V_s)^2 = g(V_s)^2/g \qquad \text{Equation (1)}$$

where G_{max} is the maximum shear modulus value, r is the soil density, V_s is the shear wave velocity, g is the unit weight, and g is acceleration due to gravity. The applicant listed low-strain shear wave velocity and shear modulus in SER Table 2.5.4-3.

The applicant stated that it followed the EPRI procedures (EPRI TR-102293) to derive the dynamic shear modulus reduction in terms of depth for the granular soils (Upper and Lower Sand Strata) and the plasticity index (PI) for the cohesive soils (Blue Bluff Marl) (see SER Figure 2.5.4-5). The applicant also indicated that it used SRS shear modulus reduction factors, but did not explain how these were used (see SER Figure 2.5.4-6).

In order to develop complete shear modulus reduction curves, the applicant extended the EPRI curves beyond the 1 percent strain value reported in EPRI (TR-102293) to 3.3 percent using data from Silva (2006). The applicant stated that shear modulus degradation ratios are constant beyond a 2 percent strain level.

Damping

To determine the damping ratio, x, as a function of the cyclic shear strain, the applicant followed EPRI procedures (EPRI TR-102293). It also derived the damping ratio in terms of depth for granular soils (Upper and Lower Sand Strata) and PI for cohesive soils (Blue Bluff Marl), shown in SER Figure 2.5.4-7. The applicant stated that it also used the SRS relationships (Lee 1996) shown in SER Figure 2.5.4-8, but the applicant did not explain in SSAR Section 2.5.4.7.2.2 how the SRS relationships were used. Similarly, the applicant extended the damping curves beyond the 1 percent strain level, as described above, for the shear modulus reduction curves. The applicant then used these damping curves to determine the site response to dynamic loading, which is described in SSAR Section 2.5.2.5.1.5.

Soil/Rock Column Amplification

The applicant described that it used the SHAKE2000 (Bechtel 2000) computer program to compute dynamic response at the ESP site. SHAKE2000 uses an equivalent-linear procedure to account for the nonlinearity of soils by employing an iterative process used to obtain values for shear modulus and damping that are compatible with the equivalent strain induced in each sublayer.

2.5.4.1.8 Liquefaction Potential

In Section 2.5.4.8 of the SSAR, the applicant explained that soil liquefaction can occur when all of the following three criteria are met:

1. Design ground acceleration is high.
2. Soil is saturated (i.e., close to or below the water table).
3. Site soils are sands or silty sands in a loose or medium-dense condition.

The applicant stated that the Upper Sand Stratum at the ESP site meets these criteria. These soils consist of sands with a varying fines (particle size) content. Approximately 9.1 meters (30 ft) of the Upper Sand Stratum occurs beneath the ground water table at a depth of 18.3 meters (60 ft) beneath the ground surface. The average corrected SPT N-value within the Upper Sand Stratum was 25 blows per foot (bpf), indicating a medium-dense condition. However, the applicant indicated that the underlying Blue Bluff Marl is significantly cohesive, and the Lower Sand Stratum is sufficiently dense and deep; therefore, the applicant concluded that liquefaction is not a concern for these strata.

Previous Liquefaction Analysis

Previous investigations for the existing VEGP Units 1 and 2 evaluated the liquefaction potential for the Upper Sand Stratum using SPT blow counts and the simplified procedure of Seed and Idriss (1970). The evaluation indicated that, below the ground water table, the Upper Sand Stratum was susceptible to liquefaction when subjected to the maximum SSE acceleration (0.2 g) developed for the existing VEGP units. The applicant also used the analysis performed on borrow sources to conclude that there is an adequate factor of safety against liquefaction for backfill materials compacted to 97 percent of the maximum density obtained by ASTM D 1557.

Table 2.5.4-3 - Shear Wave Velocity and Dynamic Shear Modulus (Based on SSAR Tables 2.5.4-2 and 2.5.4-10)

Geologic Formation	Depth m (ft)	Elevation m (ft)	Shear Velocity m/s (f/s)	Shear Modulus MPa (psf)
Compacted Backfill	0 to 1.8 (0 to 6)	68.0 to 66.1 (223 to 217)	174.7 (573)	60.09 (1,255)
	1.8 to 3.0 (6 to 10)	66.1 to 64.9 (217 to 213)	223.1 (732)	98.11 (2,049)
	3.0 to 4.3 (10 to 14)	64.9 to 63.7 (213 to 209)	247.2 (811)	120.18 (2,510)
	4.3 to 5.5 (14 to 18)	63.7 to 62.5 (209 to 205)	265.5 (871)	138.76 (2,898)
	5.5 to 7.0 (18 to 23)	62.5 to 61.0 (205 to 200)	282.5 (927)	157.05 (3,280)
	7.0 to 8.8 (23 to 29)	61.0 to 59.1 (200 to 194)	299.6 (983)	176.87 (3,694)
	8.8 to 11.0 (29 to 36)	59.1 to 57.0 (194 to 187)	317.0 (1,040)	197.75 (4,130)
	11.0 to 13.1 (36 to 43)	57.0 to 54.9 (187 to 180)	332.8 (1,092)	218.00 (4,553)
	13.1 to 15.2 (43 to 50)	54.9 to 52.7 (180 to 173)	346.6 (1,137)	236.53 (4,940)
	15.2 to 17.1 (50 to 56)	52.7 to 50.9 (173 to 167)	538.1 (1,175)	252.52 (5,274)
	17.1 to 19.2 (56 to 63)	50.9 to 48.8 (167 to 160)	368.5 (1,209)	267.55 (5,588)
	19.2 to 21.6 (63 to 71)	48.8 to 46.3 (160 to 152)	375.5 (1,232)	277.51 (5,796)
	21.6 to 24.1 (71 to 79)	46.3 to 43.9 (152 to 144)	381.9 (1,253)	287.33 (6,001)
	24.1 to 26.2 (79 to 86)	43.9 to 41.8 (144 to 137)	388.0 (1,273)	296.19 (6,186)
Blue Bluff Marl (Lisbon Formation)	26.2 to 28.0 (86 to 92)	41.8 to 39.9 (137 to 131)	426.7 (1,400)	334.11 (6,978)

Geologic Formation	Depth m (ft)	Elevation m (ft)	Shear Velocity m/s (f/s)	Shear Modulus MPa (psf)
	28.0 to 29.6 (92 to 97)	39.9 to 38.4 (131 to 126)	518.2 (1,700)	494.17 (10,321)
	29.6 to 31.1 (97 to 102)	38.4 to 36.9 (126 to 121)	640.1 (2,100)	754.11 (15,750)
	31.1 to 32.0 (102 to 105)	36.9 to 36.0 (121 to 118)	518.2 (1,700)	494.17 (10,321)
	32.0 to 33.8 (105 to 111)	36.0 to 33.8 (118 to 112)	670.6 (2,200)	827.66 (17,286)
	33.8 to 37.5 (111 to 123)	33.8 to 30.5 (112 to 100)	716.3 (2,350)	944.34 (19,723)
	37.5 to 45.4 (123 to 149)	30.5 to 22.6 (100 to 74)	807.7 (2,650)	1,200.83 (25,080)
Lower Sand Stratum (Still Branch)	45.4 to 47.5 (149 to 156)	22.6 to 20.4 (74 to 67)	609.6 (2,000)	684.02 (14,286)
	47.5 to 65.8 (156 to 216)	20.4 to 2.1 (67 to 7)	502.9 (1,650)	465.54 (9,723)
(Congaree)	65.8 to 100.9 (216 to 331)	2.1 to -32.9 (7 to -108)	594.4 (1,950)	650.21 (13,580)
(Snapp)	100.9 to 133.5 (331 to 438)	-32.9 to -65.5 (-108 to -215)	624.8 (2,050)	718.63 (15,009)
(Black Mingo)	133.5 to 145.4 (438 to 477)	-65.5 to -77.4 (-215 to -254)	716.3 (2,350)	944.34 (19,723)
(Steel Creek)	145.4 to 178.9 (477 to 587)	-77.4 to -110.9 (-254 to -364)	807.7 (2,650)	1,200.83 (25,080)
(Gaillard/Black Creek)	178.9 to 243.2 (587 to 798)	-110.9 to -175.3 (-364 to -575)	868.7 (2,850)	1,388.96 (29,009)
(Pio Nono)	243.2 to 261.5 (798 to 858)	-175.3 to -193.5 (-575 to -635)	874.8 (2,870)	1,408.54 (29,418)
(Cape Fear)	261.5 to 319.7 (858 to 1,049)	-193.5 to -251.8 (-635 to -826)	826.0 (2,710)	1,255.85 (26,229)

Geologic Formation	Depth m (ft)	Elevation m (ft)	Shear Velocity m/s (f/s)	Shear Modulus MPa (psf)
Dunbarton Triassic Basin & Paleozoic Crystalline Rock	319.7 (1,049)	-251.8 (-826)	826.0 (2,710)	N/A
	333.1 (1,093)	-265.2 (-870)	1,615.4 (5,300)	N/A
	403.3 (1,323)	-335.3 (-1,100)	2,377.4 (7,800)	N/A

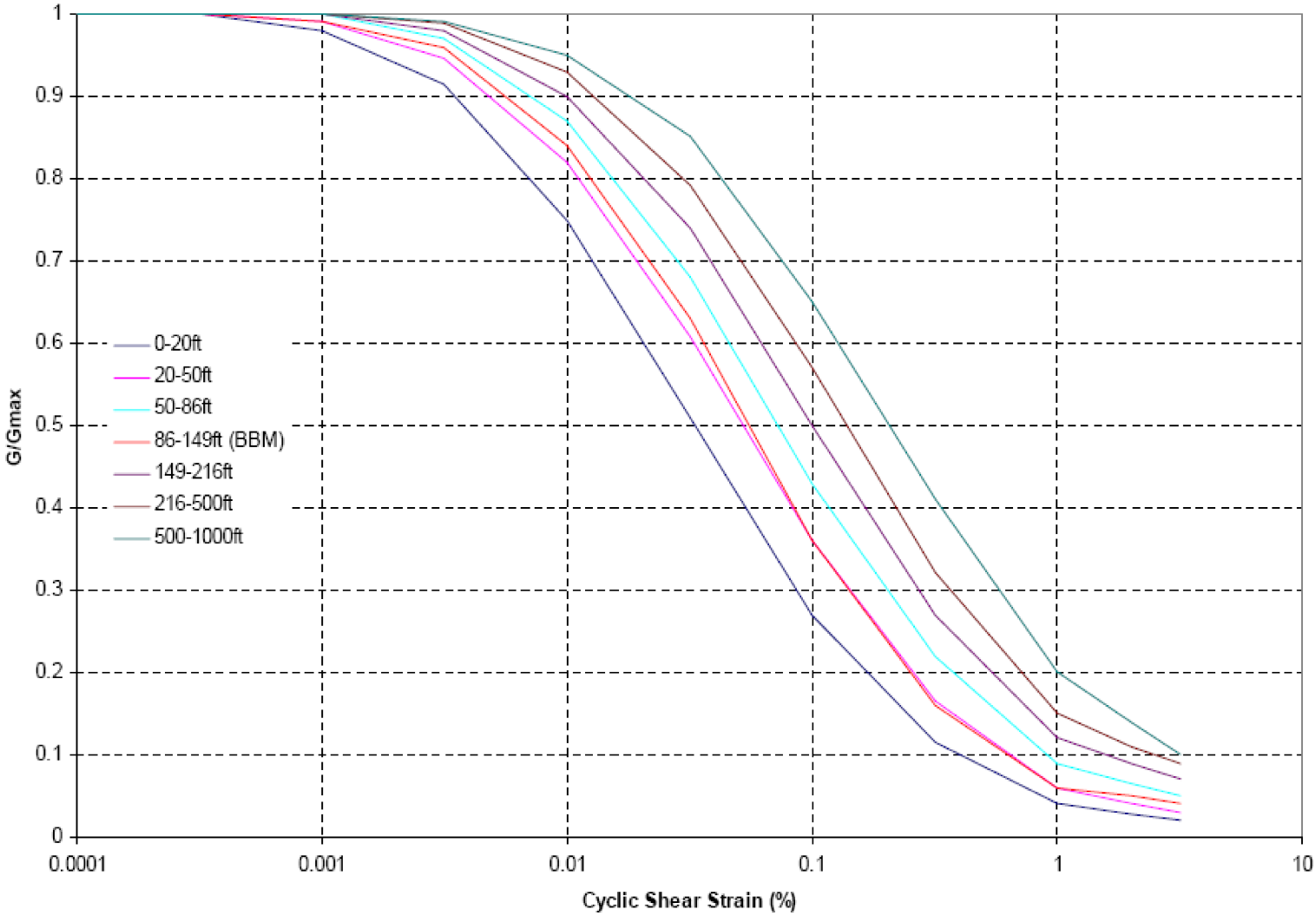


Figure 2.5.4-5 - Shear Modulus Reduction Curve For SHAKE2000 Analysis—EPRI Curve (Reproduced from SSAR Figure 2.5.4-9)

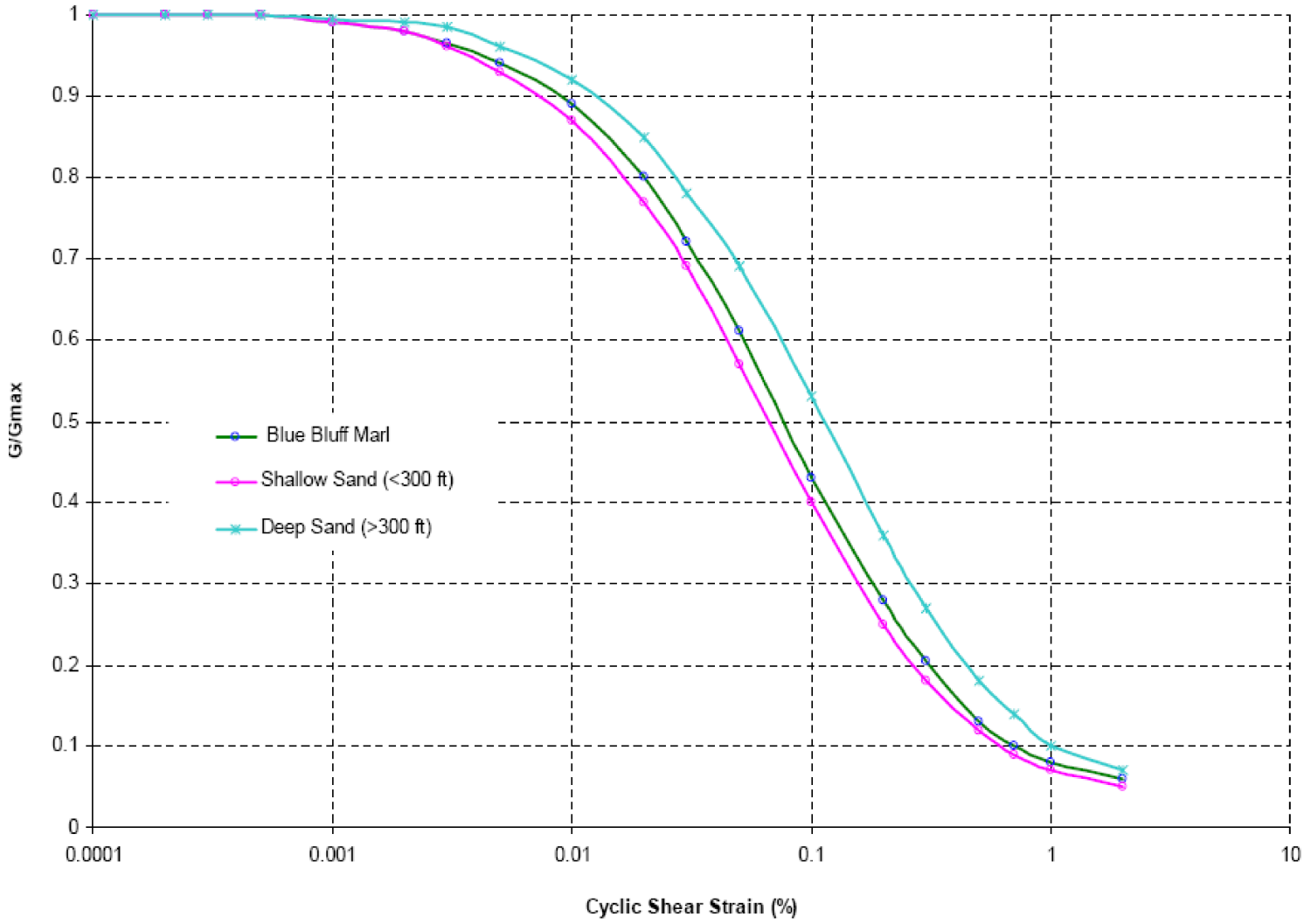


Figure 2.5.4-6 - Shear Modulus Reduction Curve for SHAKE2000 Analysis—SRS Curve

(Reproduced from SSAR Figure 2.5.4-10)

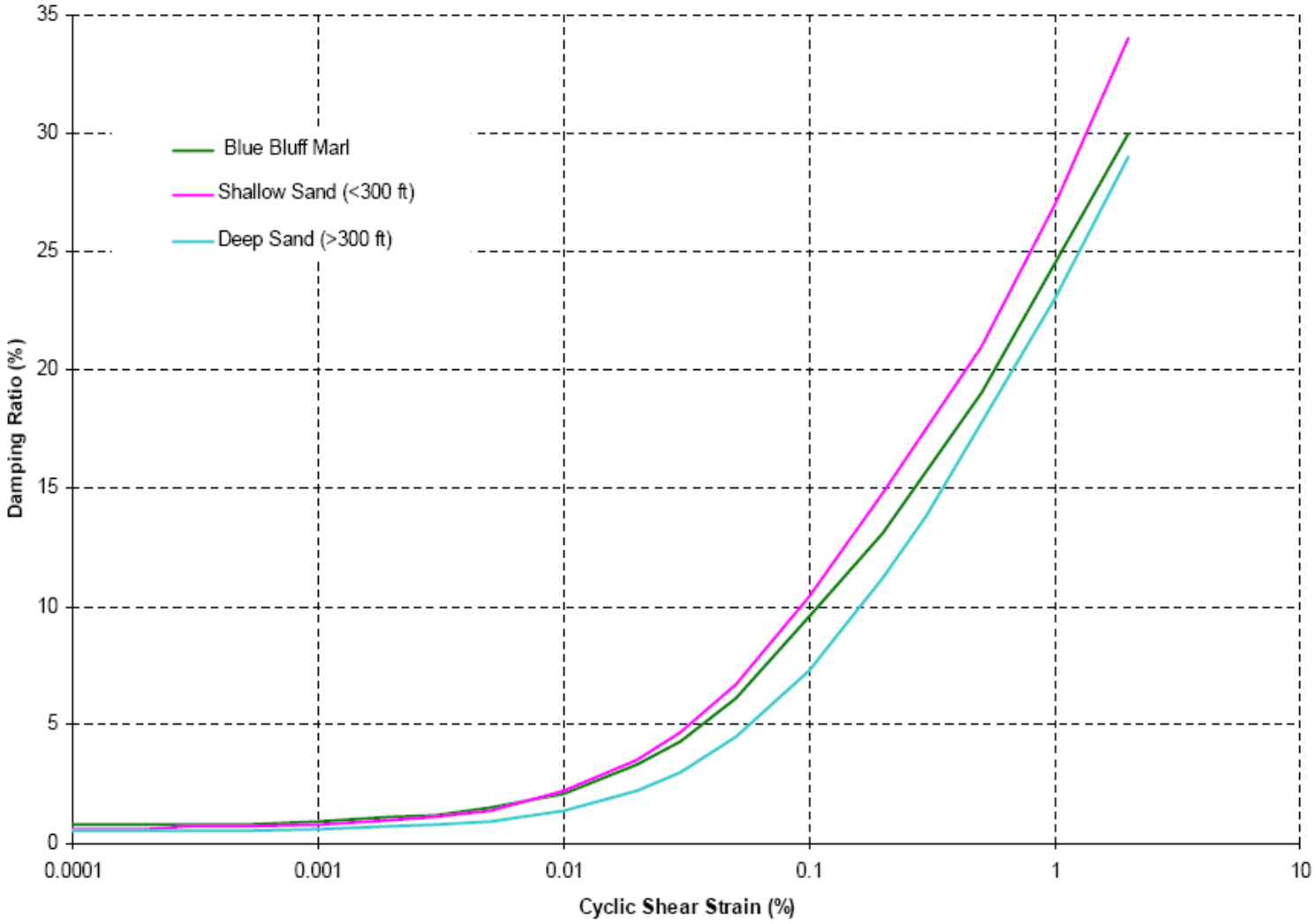


Figure 2.5.4 - Damping Curve For SHAKE2000 Analysis—EPRI Curve
(Reproduced from SSAR Figure 2.5.4-11)

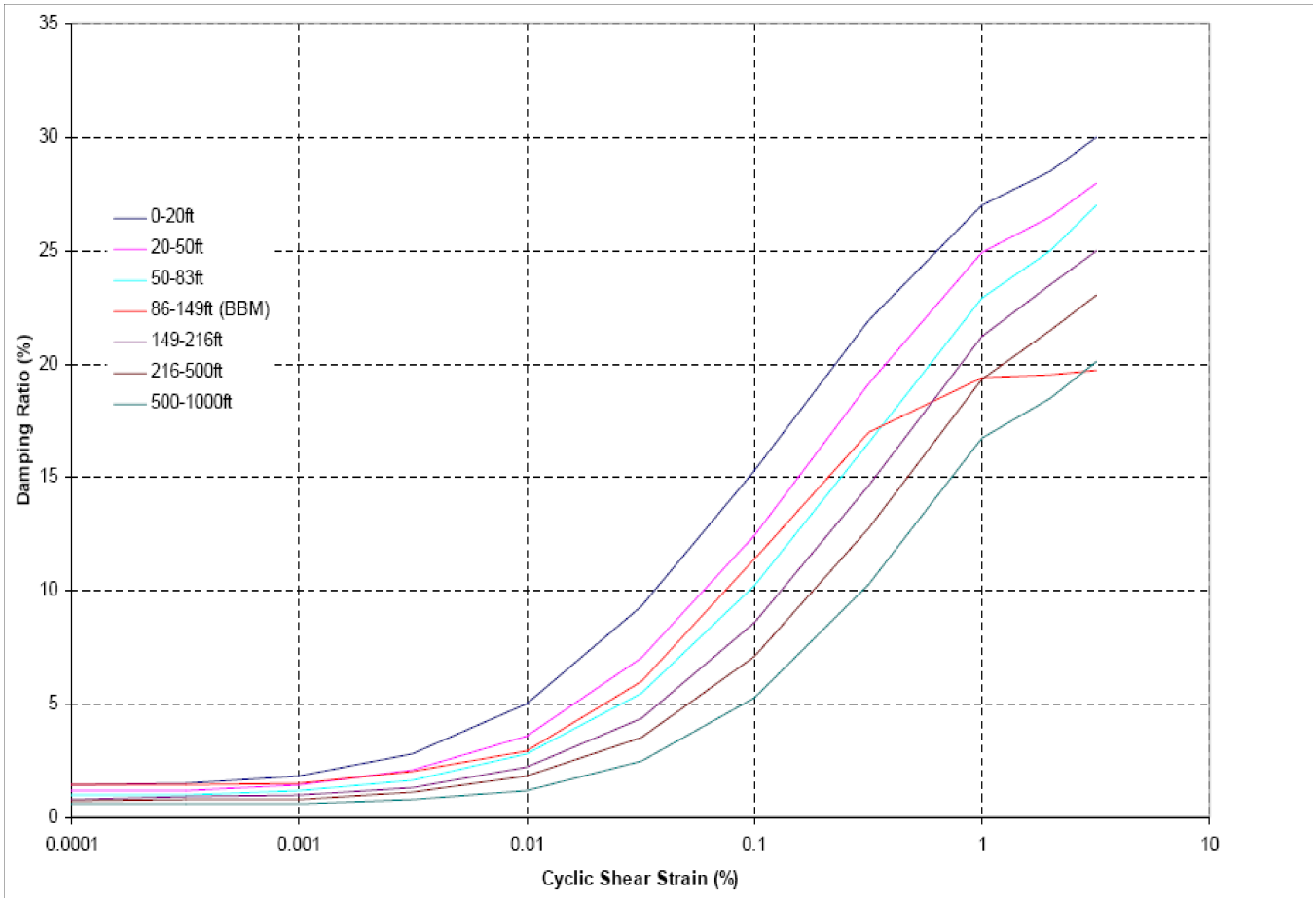


Figure 2.5.4-8 - Damping Curve For SHAKE2000 Analysis—SRS Curve (Reproduced from SSAR Figure 2.5.4-12)

ESP Site Liquefaction Analysis

The applicant stated that, based on previous investigations, it will remove the Upper Sand Stratum and replace it with compacted, nonliquefiable fills to meet the necessary plant grade within the footprint of the planned power block. The applicant also stated that, because it will remove the Upper Sand Stratum in the power block area, it did not perform a liquefaction study; however, confirmatory liquefaction analysis will proceed once the applicant determines, during the COL phase of the project, what the backfill material will be.

2.5.4.1.9 Earthquake Design Basis

SSAR Section 2.5.4.9 states that SSAR Sections 2.5.2.6 and 2.5.2.7 discuss and explain the SSE and that SSAR Section 2.5.2.8 discusses the OBE.

2.5.4.1.10 Static Stability

SSAR Sections 2.5.4.10.1 and 2.5.4.10.2 describe the allowable bearing capacity for foundations and the settlement potential for compacted fills at the ESP site.

Bearing Capacity

The applicant stated that it calculated allowable bearing capacity values for foundations placed at a depth of 1.2 meters (4 ft) below finish grade, based on Terzaghi's bearing capacity equations (1955) that were modified by Vesic (1975) (see SSAR Section 2.5.4) using the effective friction angle provided for compacted fills beneath the existing Units 1 and 2. The applicant's values are shown in SER Figure 2.5.4-5. The applicant modeled the containment building mat as a circle with a diameter of about 43.3 meters (142 ft), placed at a depth of 12.0 meters (39.5 ft) below finish grade, and it calculated the allowable bearing capacity for the foundation to be 1.5 kpa (30700 psf) under static loading conditions and 2.2 kpa (46000 psf) under dynamic loading conditions.

Settlement Analysis

The applicant stated that, according to Peck et al. (1974), for a large mat foundation that supports major power plant structures, the total settlement should not exceed 50.8 millimeters (mm) (2 inches (in.)) and differential settlement should not exceed 19.05 mm (0.75 in.) for footings that support smaller plant components. The total settlement should not exceed 25.4 mm (1 in.), and the differential settlement should not exceed 12.7 mm (0.5 in.). The applicant stated that it will observe Peck's (1974) guidelines when designing the foundation for VEGP Units 3 and 4. However, the applicant also indicated that exceeding those guidelines will not necessarily cause detrimental effects to the structure and foundation because VEGP Units 1 and 2 observed settlements exceeding the guidelines significantly (from 4.0 to 4.3 inches for the containment building, 3.2 to 3.4 inches for the control building, 3.4 to 3.7 inches for the auxiliary building, and 4.5 to 4.8 inches for the cooling towers).

Settlement of Soils Beneath the Site

The applicant stated that any settlement of the compacted fill is essentially elastic and would occur during the construction period. The applicant also stated that settlement for the Blue Bluff Marl and Lower Sand Stratum should be minimal because of the approximately 27.4-m (90-ft) overburden and the dense Lower Sand Stratum. The applicant also analyzed typical foundations for settlement, assuming a profile consisting of 24.1 meters (79 ft) of fills underlain by the Blue Bluff Marl and Lower Sand Stratum. SER Figure 2.5.4-10 shows computed total settlement of these typical foundations.

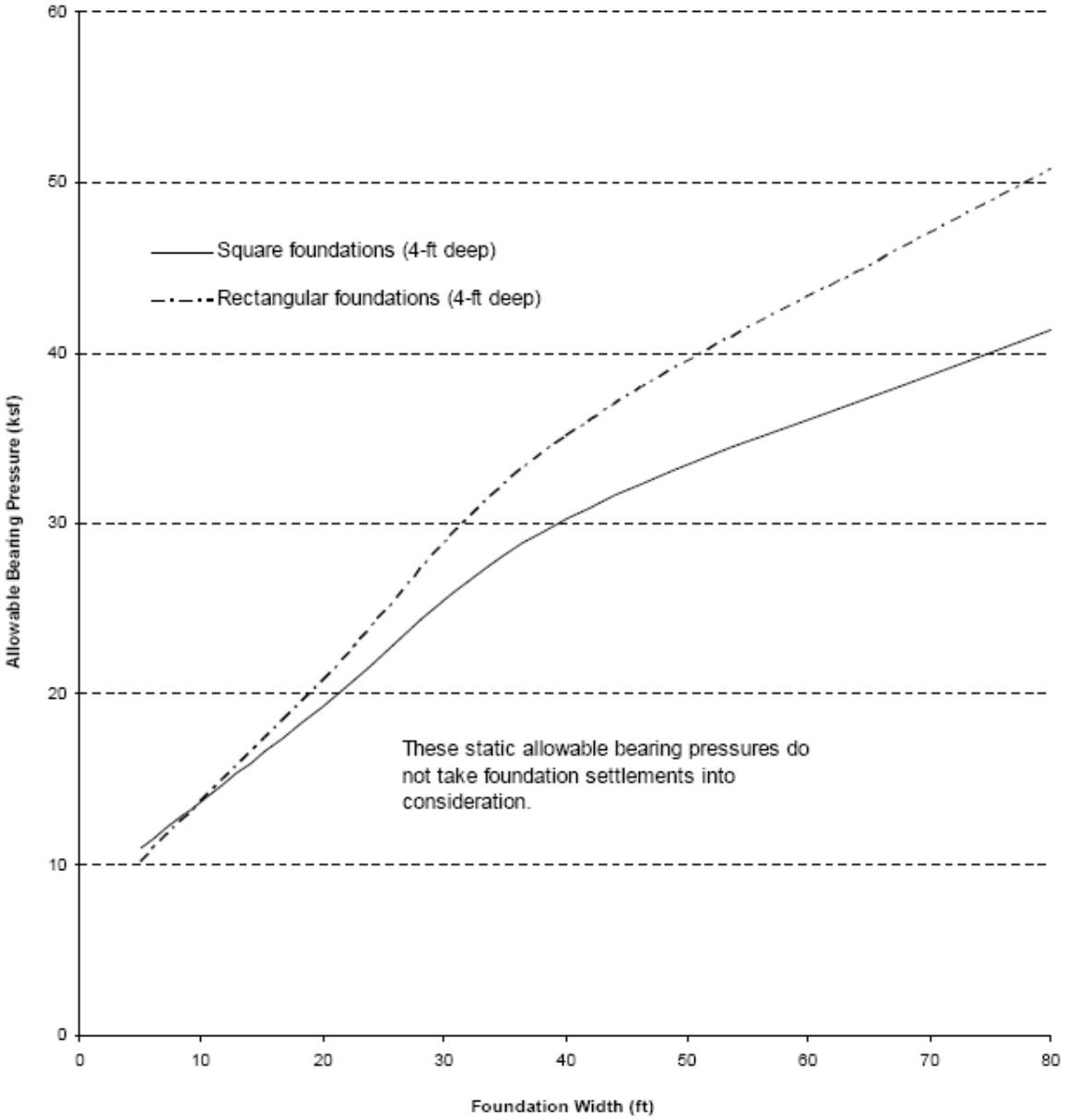


Figure 2.5.4-9 - Allowable Bearing Capacity of Typical Foundations (Reproduced from SSAR Figure 2.5.4-13)

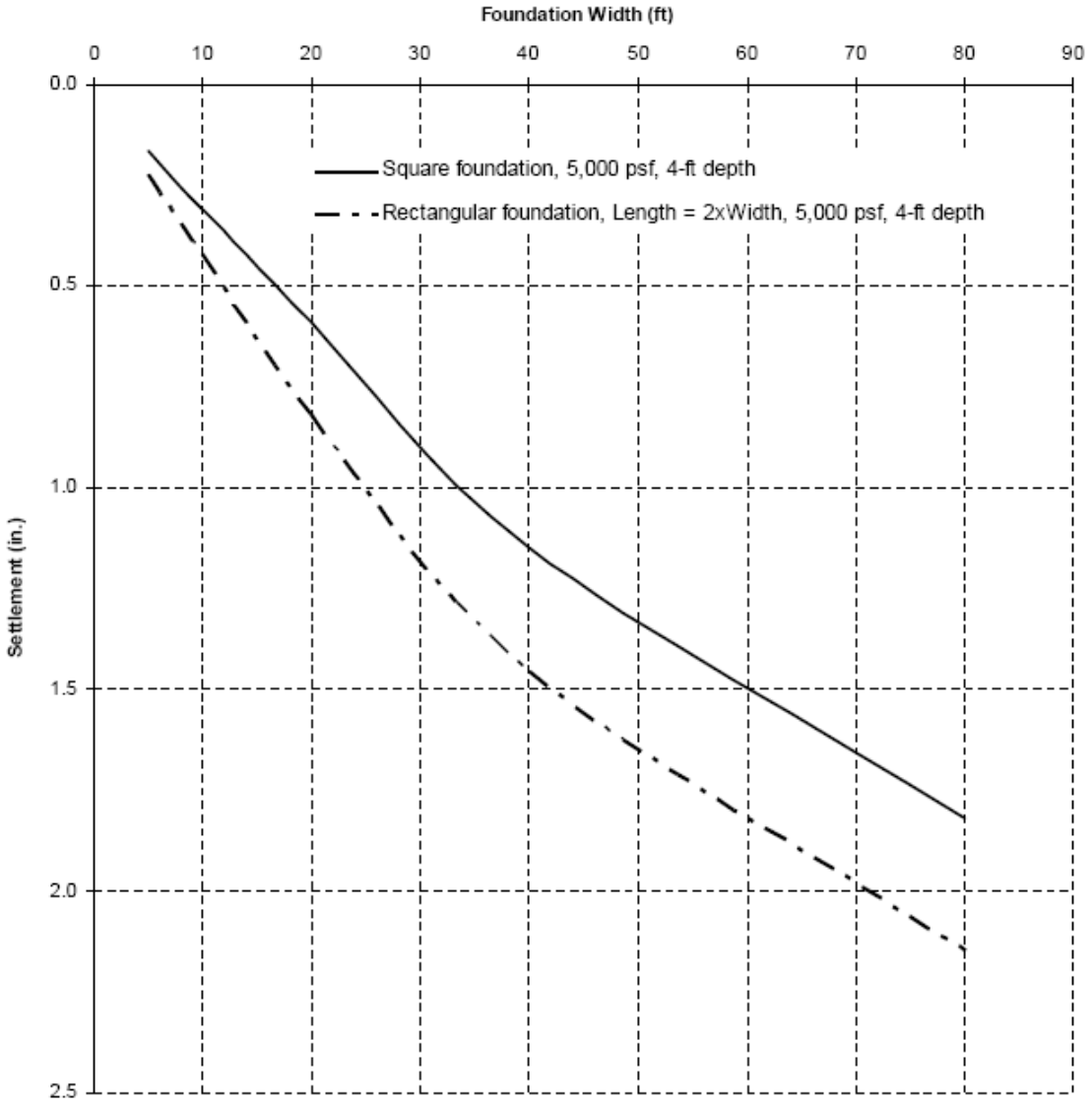


Figure 2.5.4-10 - Settlement of Typical Foundation
(Reproduced from SSAR Figure 2.5.4-14)

2.5.4.1.11 Design Criteria

In SSAR Section 2.5.4.11, the applicant summarized the geotechnical design criteria. The applicant stated that the acceptable factor of safety against liquefaction for the site soils is greater than or equal to 1.35. SSAR Section 2.5.4.10 presents bearing capacity and settlement criteria. Generally acceptable total and differential settlements are limited to 50.8 mm (2 in.) and 19.1 mm (0.75 in.) for mat foundations and 25.4 mm (1 in.) and 12.7 mm (0.5 in.), respectively, for footings. The applicant indicated that SSAR Section 2.5.5.2 specifies that the minimum acceptable long-term static factor against slope stability failure is 1.5; SSAR Section 2.5.5.3 specifies that the minimum acceptable long-term seismic factor for safety against slope stability is 1.1.

2.5.4.1.12 Techniques to Improve Subsurface Conditions

SSAR Section 2.5.4.12 states that the applicant did not consider any ground improvement beyond removing and replacing the Upper Sand Stratum, but that it will consider additional ground improvement methods, as warranted for specific locations, during the COL phase. The applicant stated that, for areas outside the power block excavation, it can improve surficial ground by densification with heavy vibratory rollers. Finally, the applicant stated that it will apply other ground improvement methods as needed.

2.5.4.2 Regulatory Basis

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, "Geologic and Seismic Siting Criteria." The applicant also applied the guidance in the following documents:

- NUREG-0800, issued March 2007
- RG 1.70, Revision 3, issued November 1978
- RG 1.198, "Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites," issued November 2003
- RG 1.132, Revision 2, issued in 2003
- RG 1.138, Revision 2, issued in 2003

The staff reviewed SSAR Section 2.5.4 for conformance with the regulatory requirements and guidance applicable to the characterization of the stability of subsurface materials, as identified below. 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 NUREG-0800 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 geophysical investigations.

2.5.4.3 Technical Evaluation

This SER section provides the staff's evaluation of the geotechnical investigations conducted by the applicant to determine the static and dynamic engineering properties of the subsurface materials at 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 these investigations to confirm the large volume of geotechnical data obtained by the SNC for the existing VEGP Units 1 and 2. The applicant used the subsurface material properties from its limited field and laboratory testings to evaluate the site geotechnical conditions and to derive various design values for the ESP application.

Based on its review of SSAR Section 2.5.4, the staff determined whether the applicant adequately demonstrated the stability of the subsurface materials responding to static and dynamic loading at the ESP site. The staff also reviewed the applicant's field and laboratory investigations and its determination of the geotechnical engineering properties of the soils and rocks underlying the ESP site. In addition, the staff made multiple trips to the site to observe some of the applicant's onsite borings and field explorations as well as to observe the site geotechnical conditions in order to determine whether the applicant followed the guidance of RG 1.132 and other relevant guidance in its site-specific investigations.

2.5.4.3.1 Description of Site Geologic Features

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

2.5.4.3.2 Properties of Subsurface Materials

The staff focused its review of SSAR 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. In general, the applicant relied heavily on the previous field and laboratory investigations for the existing VEGP Units 1 and 2 to evaluate the engineering properties and the stability of the soils and rocks underlying the ESP site, instead of implementing a complete field investigation and sampling program.

The applicant drilled 12 borings and sampled at regular depth intervals. The depths of these borings ranged from 30.1 to 92.7 meters (98.9 to 304 ft), except for boring B-1003, which reached a depth of 407.8 meters (1338 ft). The applicant drilled two additional borings without sampling to depths of 32.0 meters (105 ft) (B-1002A) and 18.3 meters (60 ft) (C-1005A) to perform suspension P-S velocity tests. The applicant also performed limited engineering property tests on the Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum, including 15 undrained shear strength tests for the Blue Bluff Marl. The field tests included SPTs, suspension P-S velocity, and static and seismic CPTs.

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

In RAI 2.5.4-3, the staff asked the applicant to provide justification for developing geotechnical parameters for the Blue Bluff Marl and Lower Sand Stratum (the main load-bearing layers) using only the data from four borings with no significant sampling in the Lower Sand Stratum. In its response, the applicant stated that three ESP borings completely penetrated the Blue Bluff Marl and another nine borings extended partially into the marl. Among the three, borings B-1002 and B-1004 penetrated through the marl into the Still Branch and Congaree Formations and boring B-1003 went as deep as 407.8 meters (1338 ft) into the bedrock. The applicant obtained a total of 58 SPT N-values and corresponding samples, as well as 12 tube samples from the Blue Bluff Marl and the Lower Sand Stratum, and performed P-S velocity logging in the three borings that penetrated the marl. Despite its ESP investigation, the applicant stated that it considered the soil engineering properties from the previous investigations of Units 1 and 2.

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

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

Considering the existence of the very low SPT N-values measured from the ESP field tests, in RAI 2.5.4-3(c), the staff asked the applicant to explain if there were any indications of soft

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

After reviewing the applicant's response to RAI 2.5.4-3, the staff concludes that, because the extent of the excavation and backfill will be limited in both the vertical and horizontal directions at the ESP site, it is not clear from the response that the purpose of the placement of backfill material is to eliminate the existence of such soft zones located outside the foundation area. Although these soft zones are outside of the immediate foundation area, these soft zones can still have potential adverse impacts on the foundation and the structures of the nuclear power plant. Since the applicant, in its response, committed to take six more deep borings (250ft to 400 ft deep) during the COL subsurface investigation, and this information is not necessary to determine whether 10 CFR Part 100 is satisfied, the issue of confirming the locations of the soft zones and to evaluate the potential impact of the soft zones on the foundation and structures is **COL Action Item 2.5-2**.

In RAI 2.5.4-7, the staff asked the applicant to explain why the undrained shear strength values (7.2 kpa (150 psf) to 205.9 kPa (4300 psf)) from the unconsolidated undrained tests performed on the Blue Bluff Marl samples were significantly lower than the SSAR specified design value, 478.9 kPa (10,000 psf), and to explain why these values differed substantially from the values (12.0 kpa (250 psf) to 23,946.4 kPa (500,000 psf)) obtained from previous investigations for Units 1 and 2. The staff also asked the applicant to justify the use of a 478.9 kPa (10,000 psf) design value based on the SPT N-values measured during the ESP investigations. In its response to RAI 2.5.4-7, the applicant stated that the laboratory measurements of undrained shear strength for the Blue Bluff Marl (Lisbon Formation) yielded low values because it performed the tests using one confining pressure corresponding to the overburden pressure. The applicant also listed some qualitative factors to explain why these laboratory values were low. These factors included (1) being unable to push the CPTs below the Barnwell Group and into the Lisbon Formation (Blue Bluff Marl), (2) Shelby tubes being unable to penetrate into the Lisbon Formation without being damaged, which indicated that the soils were very hard, and (3) possible disturbance of samples obtained by pitcher barrel due to sampling, storage, and transportation processes. Therefore, the applicant adopted an undrained shear strength design value for the Blue Bluff Marl from the FSAR for VEGP Units 1 and 2. The applicant further provided empirical correlations between the PI value, SPT N-value, shear wave velocity, and the undrained shear strength to justify the use of the SSAR design value of 478.9 kPa (10,000 psf).

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

1. The design strength value obtained from the previous investigation for Units 1 and 2 was generated using different regulatory requirements, different industry standards, and

different testing technologies. The applicant can use the data or engineering values from the previous investigation as a reference to support the current decision, but may not use the data as a direct input to calculate engineering parameters or previous engineering values directly for the ESP site.

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

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

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

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

Based on the above considerations, the staff concludes that the applicant did not provide sufficient data to reliably derive the undrained shear strength value for the Blue Bluff Marl for the design. This is **Open Item 2.5-12**.

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

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

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

3. justification for the conclusion that "the pre-consolidation pressure of the Blue Bluff Marl was estimated to be 3831.4 kPa (80,000 psf)"

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

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

1. The original data and interpretation were based on laboratory tests performed for VEGP Units 1 and 2, which included 191 one-point unconsolidated undrained triaxial tests and 38 consolidation tests. It used vertical pressures that reached 3065.1 kPa (64000 psf) to perform consolidation tests for all 38 samples. Most of the test results (void ratio versus vertical effective stress curves) showed very flat curves which indicated that the preconsolidation pressure had not been achieved.

2. The undrained shear strength of 766.3 kPa (16,000 psf) was an average value based on VEGP Unit 1 and 2 test data calculated from 185 one-point unconsolidated undrained triaxial tests that disclosed undrained shear strength values of less than 2,394.6 kPa (50,000 psf).

3. The applicant used the Skempton (1957) method to estimate the preconsolidation pressure of the Blue Bluff Marl by relating the preconsolidation pressure to the PI value and the undrained shear strength. The applicant concluded that the Lisbon Formation was highly overconsolidated because its calculations showed that the overconsolidation ratios (OCRs) were in the range of 3.6 to 5, and most of the consolidation test results on 38 samples from the Lisbon Formation, reported in Bechtel (1974b), showed very flat curves which indicated that the preconsolidation pressure exceeded 3,065.1 kPa (64,000 psf).

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

Based on its review of the applicant's response to RAI 2.5.4-8, the staff finds that it is inappropriate to use the average undrained shear strength value for VEGP Units 1 and 2 as an input value to calculate preconsolidation pressure and OCRs for the Blue Bluff Marl at the ESP site. The staff finds it inappropriate because the previous value was obtained based on different regulatory requirements, regulatory guidelines, industry standards, and testing technologies. In addition, the spatial variation of the soil properties also makes reliance on the VEGP Units 1 and 2 values inappropriate. Moreover, the previous shear strength value differs

significantly from the one obtained during the ESP testing. Therefore, the applicant did not have sufficient sampling and testing results to reliably derive the input undrained shear strength used in calculating the precoconsolidation pressure and OCRs of the Blue Bluff Marl. The issue discussed above is designated as **Open Item 2.5-13**.

In RAI 2.5.4-9, the staff asked the applicant to clarify how the effective angle of internal friction was determined for the soils underlying the ESP site. The applicant responded that it estimated the effective angle of internal friction of 34° using an empirical correlation associated with SPT N-values (Bowles 1982). From its review of the applicant's response, the staff considers that the internal friction angle calculated based on SPT N-values varies significantly, depending on the correlations used. For example, for N-values between 10 and 40, the corresponding soil internal friction angle values vary from 30° to 36° (Peck 1974) or from 35° to 40° (Bowles 1982). More importantly, the N-values measured for the ESP site are all below 20 (from 3 to 19), according to SER Table 2.5.4-4. Therefore, the use of a friction angle of 34° based on an N-value of 25 for the Upper Sand Stratum appears to be inappropriate. The staff concludes that the applicant did not provide reliable effective angles of internal friction for the subsurface soils because it did not have sufficient SPT N-values from the ESP investigation to support its calculation. The internal friction angle for the subsurface soils is one of the input parameters in calculating bearing capacity and settlement, as well as liquefaction potential. The issue regarding the effective angles of internal friction for the subsurface soils is designated as **Open Item 2.5-14**.

In RAI 2.5.4-10, the staff asked the applicant to provide relative density for the Blue Bluff Marl. The applicant stated in its response that the design value of the undrained shear strength for the soil was 478.9 kPa (10000 psf) and its preconsolidation pressure could be as high as 3831.4 kPa (80000 psf); therefore, the Blue Bluff Marl was highly overconsolidated and behaves as hard clay or soft rock material, not as a granular material. The applicant further stated that relative density did not apply to the Blue Bluff Marl. From its review of the applicant's response, the staff concludes that test data for the Blue Bluff Marl are very limited. As described in the SSAR, the limited laboratory test data showed that the percent fines content ranged from 24 to 77 percent, the moisture content ranged from 14 to 67 percent, and the PI ranged from nonplastic to 58 percent. Each of the above-mentioned parameters does not exclude the possibility of the marl being liquefied. In addition, the undrained unconsolidated tests yielded undrained shear strength values from 7.2 to 205.9 kPa (150 to 4300 psf), which significantly differ from the design shear strength value of 478.9 kPa (10000 psf), as indicated in the discussion of RAI 2.5.4-7. Therefore, the applicant's response did not support its conclusion that the Blue Bluff Marl will behave as a hard clay or soft rock material because it did not use its own ESP soil engineering values to calculate relative density for the Blue Bluff Marl. The issue of demonstrating that the Blue Bluff Marl will behave as a hard clay or soft rock material thus not need to be addressed using relative density is designated as **Open Item 2.5-15**.

In RAI 2.5.4-11, the staff asked the applicant to explain: (1) why it used the Davie and Lewis' (1988) relationship to estimate the high strain elastic modulus (E) for the Upper and Lower Sand Strata underlying the ESP site; (2) what the consensus is about using the Davie and Lewis relationship between SPT and elastic modulus E; (3) and the extent of the application of the Davie and Lewis relationship. In response to RAI 2.5.4-11, the applicant stated that Bechtel used the relationship extensively to estimate settlement when compared to observed

settlements for a wide range of foundation sizes on granular materials from clean sands to silty sands to gravels. The Upper Sand Stratum is a medium-dense, silty sand and the Lower Sand Stratum is a very dense silty sand. Therefore, the applicant believed that the Davie and Lewis relationship is applicable to those sands. In addition, the applicant found that the Davie and Lewis relationship provided an E value that was closer to the median value of five different relationships for both sand strata than the four other E and N (the SPT N-value) relationships detailed in SER Table 2.5.4-4, which is taken from the applicant's response to RAI 2.5.4-11. The applicant also pointed out that Davie and Lewis' relationship provided reasonable predictions of settlement when compared to measured settlements, and it has a reasonable consensus.

Table 2.5.4-4 - Summary of Calculation of Elastic Modulus E

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

Based on its review of the applicant's response to RAI 2.5.4-11, the staff concurs with the applicant's conclusion about the applicability of the Davie and Lewis' relationship in estimating elastic modulus E. However, the applicant needs to use appropriate SPT N-values to obtain a reasonable E value. Since the N-values obtained from the ESP investigation and the design undrained shear strength values determined by the applicant for the ESP soils are not reliable for very limited data, the applicant did not have sufficient site-specific data to justify the determination of the design parameter E for the Upper and Lower Sand Strata. The issue of using appropriate SPT N-values to determine a reasonable elastic modulus value E for the Upper and Lower Sand Strata is designated as **Open Item 2.5-16**.

In RAI 2.5.4-12, the staff asked the applicant to explain how it determined the unit weight values for different soils and why there was a discrepancy between the average values given in the SSAR text and those listed in SSAR Table 2.5.4-1. The applicant explained in its response that it determined the unit weight values based on the laboratory test during the ESP subsurface investigation. However, the applicant used the average values of unit weight based on VEGP Unit 1 and 2 laboratory test results because there were more test data available, despite results that differed from those obtained from ESP tests. The staff considers that the unit weight values for underlying soils are very basic soil property parameters used in many calculations/analyses. However, the applicant did not have sufficient data to calculate the unit weight values for the ESP subsurface soils and instead used the values from previous

investigations. The staff concludes that it is not acceptable for the applicant to use these previously determined engineering parameters in this manner. This issue is designated as **Open Item 2.5-17**.

The staff noted that, in SSAR Section 2.5.4.2.5.3, the applicant stated that chemical tests were not included in the ESP laboratory testing program. The SSAR also states that chemical tests would be required for the backfill materials placed in proximity of planned concrete foundations and buried metal piping, and commits to conduct these chemical tests in the COL investigation phase. The need to provide chemical test results on the backfill is **COL Action Item 2.5-3**.

As discussed above, the staff concludes that the applicant performed limited field and laboratory tests during the ESP subsurface investigation, and therefore, did not provide sufficient and reliable data in characterizing the engineering properties of subsurface materials. Instead, the applicant relied significantly on the test data for the existing VEGP Units 1 and 2 (obtained more than 20 years ago) to determine engineering properties of the subsurface materials. Therefore, the applicant's ESP investigation data became reference data and VEGP Units 1 and 2 became the critical input data used to calculate the soil engineering properties. Considering that the engineering properties of the subsurface material, especially the load-bearing layers, provide critical information on bearing capacity and foundation settlement, and that those parameters directly relate to soil-structure interaction (SSI) analysis as well as foundation stability, the information provided by the applicant is inadequate to establish reliable engineering properties for the subsurface material at the ESP site. The staff concludes that the applicant did not conduct sufficient field and laboratory tests to reliably determine subsurface soil static and dynamic properties.

2.5.4.3.3 Foundation Interfaces

Section 2.5.4.3 of NUREG-0800 directs the staff to compare the applicant's plot plans and profiles of seismic Category I facilities with the subsurface profile and material properties. Based on the comparison, the staff can determine if (1) the applicant performed sufficient exploration of the subsurface materials and (2) if the applicant's foundation design assumptions contain an adequate margin of safety.

In RAI 2.5.4-20, the staff asked the applicant to justify why it did not provide the relationship of foundations to the underlying materials in the form of plot plans and profiles, the foundation stability with respect to ground water conditions, and a detailed dewatering plan. In its response, the applicant stated that it would provide this information as part of a COL application, once more details become available regarding the foundation and site interaction. The staff agrees with the applicant that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied. This is **COL Action Item 2.5-4**.

2.5.4.3.4 Geophysical Surveys

The staff focused its review of SSAR Section 2.5.4.3 on the adequacy of the applicant's geophysical investigations to determine the soil and rock dynamic properties. The applicant conducted three down-hole seismic CPT tests and five suspension P-S velocity tests during the ESP site investigation. The applicant compared the soil and rock dynamic properties obtained

from the tests with the results from the previous geophysical surveys for the existing Units 1 and 2.

In RAI 2.5.4-3, the staff asked the applicant to explain how it developed the base case shear wave velocity profile based on only 12 borings, since most of them did not go deeper than 91.4 meters (300 ft). In its response to RAI 2.5.4-3, the applicant stated that it developed the base case shear wave velocity profile associated with the Lisbon Formation (Blue Bluff Marl), Still Branch Formation, and the upper portion of the Congaree Formation from the results of the three suspension P-S velocity logging tests performed at the ESP site. One of the suspension P-S velocity logging tests extended into bedrock below the Lower Sand Stratum, and the applicant used those results to derive the base case shear wave velocity profile below the top of the Congaree Formation. The applicant explained that its randomization model, which captures the uncertainty in the base case shear wave velocity profile for the in situ soils, used logarithmic standard deviation of shear wave velocity as a function of depth, which is set to values obtained from soil randomization performed at SRS. After reviewing the applicant's response, the staff, however, finds that shear wave velocities vary significantly among the three profiles (ESP, VEGP, Units 1 and 2 and SRS, most of them terminate at a depth of about 85.34 to 60.96 meters (280 to 300 ft)), shear wave velocities measured from down-hole seismic tests are lower than the suspension P-S velocity measurements. Furthermore, the shear wave velocities from previous investigations were relatively lower than those obtained from the ESP investigations. Therefore, the staff concludes that the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile. This is **Open Item 2.5-18**.

Based on its review of SSAR Section 2.5.4.4 and the applicant's response to RAI 2.5.4-3, described above, the staff concludes that the applicant used various methods to determine compressional and shear wave velocities, including some of the latest technologies recommended in RG 1.132. However, the applicant did not provide sufficient shear wave velocity measurements to define the site-specific shear wave velocity profile and address the velocity difference from different methods.

2.5.4.3.5 Excavation and Backfill

The staff reviewed SSAR Section 2.5.4.5 focusing on the applicant's description of anticipated foundation excavations for safety-related structures, backfills, and slopes; excavation methods and stability; backfill sources and quality control; and control of ground water during excavation. The applicant stated that it would remove the Upper Sand Stratum and perform filling from the top of the Blue Bluff Marl to the bottom of the containment and auxiliary buildings at a depth of about 12.19 meters (40 ft) below the final grade. Filling would continue up around those structures to final grade. The excavation would be open-cut, with slopes no steeper than 2:1 (horizontal-to-vertical ratio). The applicant indicated that it would follow the guidelines used for VEGP Units 1 and 2 to develop excavation and backfill plans during the COL application phase.

Since there was no specific description on excavation, in RAI 2.5.4-2, the staff asked the applicant to clarify whether the excavation and backfill would only cover the footprint of the power block or extend to a certain distance beyond the foundation footprint. In response to RAI 2.5.4-2, the applicant explained that safety-related footprints of the future Units 3 and 4 would have two respective backfilled excavations, and those excavations would extend beyond their respective power block footprints. The applicant established the minimum lateral extent of each excavation

by determining the stress zone as defined by a 1:1 (horizontal-to-vertical) slope ratio, extending from the bottom of the turbine, containment, and auxiliary building foundations. The approximate bottom of the foundation elevations would be 65.8 meters (216 ft) above the msl for the turbine building, 54.9 meters (180 ft) above msl for the containment, and 39.6 meters (130 ft) above msl to the top of the Lisbon Formation (Blue Bluff Marl) for the auxiliary buildings. The stress zone at the top of the Lisbon Formation would extend approximately 26.2 meters (86 ft) horizontally beyond the footprint of the power block structures. The applicant considered that the turbine building foundation governed this horizontal extension (highest foundation), therefore the 26.2-m (86-ft) extension was conservatively set for all four sides of the excavation. The applicant planned to backfill the entire excavation, including the power block footprint, stress zone, and areas beyond the stress zone, using compacted structural fill.

Due to the concern of a possible backfill impact on the seismic response evaluation of the site and structures, in RAI 2.5.4-2, the staff also asked the applicant whether it would implement the seismic hazard calculations to the free-ground surface, including the Barnwell Group in the base case site soil column, if the site excavations were not to extend significant distances to the side of the plant. In addition, the staff asked the applicant to explain the basis for its column analysis that presumed uniform fill in all horizontal directions, while the actual excavation and backfill would extend only to the immediate vicinity of the plant. In its response, the applicant stated that the site excavations would extend to significant horizontal distances from the structures. With the base of the excavation extending approximately 26.2 meters (86 ft) outside of the building footprint, and with the excavation side slope ratio at 2:1(horizontal to vertical), the structural fill would extend more than 54.9 meters (180 ft) beyond the containment and auxiliary buildings at their foundation level and would extend more than 76.2 meters (250 ft) beyond the edge of the turbine building at its foundation level.

Since there was no specific description regarding the backfill compaction control, in RAI 2.5.4-2, the staff also asked the applicant to explain how it would implement compaction control if the backfill was to contain as much as 25 percent fines content. In its response, the applicant stated that it used sand and silty sand with no more than 25 percent fines, obtained from onsite sources, as structural backfill for Units 1 and 2, and that it would use the same structural backfill criterion for Units 3 and 4. It would implement compaction controls for placement of the backfill through an independent soil testing firm. This testing firm would maintain an onsite soils testing laboratory to control the quality of the fill material and the degree of compaction. The testing firm would monitor the compaction through field density tests performed at a minimum frequency of one test per 928 square meters (10,000 square ft) per lift of placed fill. In addition, the applicant committed to develop more detailed testing compaction control criteria during the COL phase. Because the site excavation and backfill will not be performed until the COL stage, the staff considers that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied.

After reviewing the responses from the applicant to RAI 2.5.4-2, the staff concludes that, although the applicant provided more information on the extent of excavation, backfill material, and its compaction control, the applicant needs to consider some related issues during the COL stage including (1) the stress zone described in the applicant's response to RAI 2.5.4-2 was based on normal static stress evaluations, but the applicant needs to consider both static and dynamic load-induced stresses and (2) since the applicant indicated that excavations would extend from about 26.2 meters (86 ft) outside of the building footprint with 2:1 (horizontal-to-vertical) side slope ratios and then extend away from the power block, the applicant needs to include the backfill material placed in and around the power block structures in the structural model when evaluating SSI, as

indicated in the currently revised Section 3.7 of NUREG-0800. The applicant commitment to provide detailed excavation and backfill plans during the COL stage is **COL Action Item 2.5-5**.

Because the applicant did not describe the determination of shear wave velocity for the backfill, in RAI 2.5.4-4, the staff asked the applicant to explain how it would determine shear wave velocity values at depths of 15.2 meters (50 ft) and deeper for the backfill materials and whether it considered the effects of confinement. In its response, the applicant reiterated this statement from SSAR Section 2.5.2.5.1.2.1.1:

> Shear-wave velocity was not measured for the compacted backfill during the ESP subsurface investigation (APPENDIX 2.5A). Interpolated values based on measurements made on fill for existing Units 1 and 2 (Bechtel 1984) are used instead.

The applicant also clarified that the measurements made of backfill soil for existing Units 1 and 2 were laboratory measurements using resonant column tests. The applicant developed shear wave velocity profiles for the backfill using equations presented in the response.

After reviewing the response to RAI 2.5.4-4, the staff finds that the applicant was attempting to apply the estimated shear wave velocity from the backfill for the existing units to the backfill for the ESP site. But the equation used in the estimation dated back to the 1960s and there was significant variability, or uncertainty, for the parameter K_2 in the equation. The calculation also did not account for confinement effects. Since the ability to show that the backfill meets the minimum shear wave velocity requirement with minimum in situ variability is a major concern in the COL phase, and the procedures presented in the SSAR did not provide such information, additional information to address the backfill shear wave velocity should be submitted in the COL application. This is **COL Action Item 2.5-6**.

In summary, based on a review of SSAR Section 2.5.4.5 and the applicant's responses to RAI 2.5.4-2 and RAI 2.5.4-4 described above, the staff believes that the applicant did not provide detailed information on excavation and backfill plans due to the limited knowledge of the exact location of reactors and fill materials. Therefore, the staff cannot fully evaluate the applicant's excavation and backfill plans until it submits this related information. Regulatory Position C.6 of RG 1.132 recognizes that there may be limitations on the extent of geologic mapping that may be performed prior to a site being approved under the 10 CFR Part 52 licensing procedures. However, the need for construction mapping applies equally under the ESP procedures. To address this need for construction mapping, the staff is proposing to include a permit condition requiring that the ESP holder or an applicant referencing the ESP perform geologic mapping of future excavations for safety-related structures, evaluate any 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 examination and evaluation. This is **Permit Condition 2**.

2.5.4.3.6 Ground Water Condition

In SSAR Section 2.5.4.6, the applicant provided some basic ground water conditions based on the water well observations and a summary of the dewatering plan implemented for VEGP Units 1 and 2. The staff finds that this information is necessary to understand the ground water conditions and potential dewatering plan at the ESP site.

In RAI 2.5.4-6, the staff asked the applicant to explain the dewatering procedures it will use for the new units. In its response to this RAI, the applicant stated that it would implement the same dewatering program as that developed for the VEGP Units 1 and 2 but with some deviations. It considered the dewatering program deployed at Units 1 and 2 to be successful, and subsurface conditions at the ESP site and at Units 1 and 2 are similar.

After reviewing the applicant's response, the staff concludes that, since the applicant has not finally determined the reactors' location within the ESP site and does not have a site-specific dewatering program, the staff cannot evaluate the ground water conditions as they affect the loading and stability of foundation materials. The staff is also unable to assess the applicant's dewatering plans during construction as well as ground water control throughout the life of the plant. Because the plant specific dewatering program cannot be planned until the reactor location is decided, the staff considers that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied. Therefore, the need of the submission of ground water condition evaluations and a detailed dewatering plan during the **COL stage is COL Action Item 2.5-7**.

2.5.4.3.7 Response of Soil and Rock to Dynamic Loading

The staff reviewed SSAR Section 2.5.4.7 focusing on how the applicant developed the base shear wave velocity profile and modeled soil modulus reduction and damping with respect to cyclic shear strain. The applicant derived shear modulus for the soil strata from the relationship related to the unit weight and shear wave velocity, as well as the dynamic shear modulus reduction and damping ratio curves derived from EPRI (EPRI TR-102293 1993). The applicant used the SHAKE2000 (Bechtel 2000) computer program to evaluate the site dynamic responses.

The applicant derived ESP soil shear modulus degradation and damping curves from the curves developed by EPRI (1993). In RAI 2.5.4-5, the staff asked the applicant to justify its application of the EPRI curves to fine-grained soils. In its response, the applicant stated that EPRI (1993) developed degradation curves for soils from gravels to high plasticity clays, and thus it was appropriate to apply the curves to fine-grained soils. EPRI (1993) presented fine-grained soils in Figures 7.A-16 (shear modulus reduction curves) and 7.A-17 (damping ratio curves) in terms of soil plasticity and required the use of the PI. The applicant referred the staff to its response to RAI 2.5.4-17 for more details on how it derived the degradation curves from the EPRI (1993) curves. The applicant further indicated that it would verify the soil degradation relationships for fine-grained soil (and coarse-grained soils) used in the SSAR by laboratory testing during COL subsurface investigation.

After reviewing the applicant's response and its references, the staff finds that Section 7A.6 of the EPRI (1993) report recommends the modulus degradation and hysteretic damping strain-dependent curves for generic CEUS sites. According to the report, these curves are intended for gravelly sands to low plasticity silty or sandy clays and should not be applied to either very gravelly or very clayey deposits. The curves presented in the report for silts and clays of high plasticity are significantly different from those for sandy soils. In its response to RAI 2.5.4-10, however, the applicant indicated that the Blue Bluff Marl "is described as hard, slightly sandy, cemented calcareous clay, and with less than 50% fine material," which is different from the type of materials for which the curves are intended. Therefore, the applicant did not adequately explain why it is appropriate to apply those relationships to the silts and clay soils at the ESP

site. The report further states that, while the generic curves are appropriate for preliminary site studies, one should use site-specific data for final evaluations. In conclusion, the staff concurs with the applicant that it needs to verify the soil modulus degradation and damping curves. However, this verification should not wait until the COL stage. Without site-specific soil modulus degradation and damping curves, the determination of site-specific GMRS (SSE) is inadequate. To provide site-specific soil degradation and damping ratio curves for the site-specific soil amplification calculation discussed in SER Section 2.5.2 is **Open Item 2.5-19**.

Because the applicant stated in the SSAR that it used values of shear modulus and damping ratio provided by Silva (2006) to extend the EPRI curves beyond the 1- to 3.3 percent strain level, in RAI 2.5.4-13, the staff asked the applicant to justify how it extended the values beyond the 1 percent strain level and to provide a complete description and supporting data. In its response, the applicant stated that, even though it extended the EPRI curves beyond the 1 percent strain level, the maximum strains calculated during the site amplification analyses remained below 1 percent. But, the applicant then stated that it would revise SSAR Sections 2.5.2.5.1.5, 2.5.4.7.2.1, and 2.5.4.7.2.2, along with associated tables and figures, to show the degradation curves only at a 1- percent or less cyclic shear strain. Due to the applicant's commitment to revise the curves back to a 1 percent strain level without extrapolation, the staff concludes that this RAI is not resolved until the revised SSAR sections are submitted for review. This is **Open Item 2.5-20**.

In RAI 2.5.4-17, the staff asked the applicant to provide a complete description, including sample calculations, to show how it derived the shear modulus reduction and damping curves and how it incorporated uncertainties in the site characteristics into the curves' development. The applicant explained in its response that it used the shear wave velocity to calculate the low-strain dynamic shear modulus (G_{max}) only. The EPRI (1993) curves simply showed the ratio G/G_{max} versus cyclic shear strain, regardless of the initial value of G_{max}. The shear modulus reduction and damping ratio curves for cohesionless materials were based on confining pressure at depth, or simply depth, but were based on the PI for cohesive material like Blue Bluff Marl. The applicant then described how it derived the shear modulus reduction and damping ratio curves from the EPRI (1993) curves shown in SER Figures 2.5.4-5 to 2.5.4-8. The applicant described how it derived shear modulus reduction and damping curves for each layer included in the base shear wave velocity profile. The applicant also stated that, "shear modulus reduction and damping curves will be obtained using undisturbed samples collected during the COL subsurface investigation."

In addressing how it incorporated uncertainties, the applicant stated that it extended EPRI shear modulus reduction curves from the strain level of 1 percent to 3 percent and that it incorporated uncertainties in the site parameters during the randomization process. Figures 2.5.4-5 through 2.5.4-8 show shear modulus reduction curves and damping ratio curves for each layer in the profile. The applicant randomized the shear modulus reduction and damping ratios at one strain level using log-normal distributions with median values given by the corresponding base-case curves and logarithmic standard deviations taken from the statistical summaries obtained by Costantino (1997) for natural soils. For the engineered backfill, it reduced these standard deviations by one-third to account for a more homogeneous soil mass. It used a hyperbolic parametric form to generate the shear modulus reduction and damping ratios at other strains from the randomized values obtained above. The applicant stated that this approach produced realistic curves with logarithmic standard deviations that approximate

the Costantino (1997) values over a wide range of strains. It assumed that the normal random variables associated with the log-normal shear modulus reduction and damping ratios had a correlation coefficient of -0.75.

After reviewing the responses from the applicant, the staff reached the following conclusions:

1. Although the EPRI (1993) curves were up to the 1 percent strain level, the applicant did not provide information on the strain levels associated with the 10^{-4}, 10^{-5}, and 10^{-6} uniform hazard response spectra (UHRS) at the bedrock in the site response analyses and did not indicate whether the laboratory data developed during the SRS testing program carried to those levels of strain.

2. The adequacy of the equivalent-linear approximations for site response deteriorates as strain levels exceed about 0.5 percent effective shear strain. The applicant did not justify the applicability of the equivalent-linear method used in the SHAKE2000 model analysis if the strain levels were to exceed 1 percent.

3. In its response to RAI 2.5.4-13, the applicant indicated that it would revise the 3.3 percent strain level extrapolation back to 1 percent for the EPRI (1993) modulus reduction and damping curves, however, its response to this RAI indicated otherwise.

4. The applicant needs to demonstrate that it can confidently obtain undisturbed samples for deeper depths (e.g., in the Blue Bluff Marl and lower sands of the Congaree and Lower Snapp formations) for use in site response and SSI studies.

5. The applicant also needs to test disturbed samples of the compacted fill material to estimate appropriate modulus reduction and damping properties for the SSI analysis.

6. Other RAI responses indicated that the applicant used both SRS and EPRI (1993) models in the site response analyses and weighted them equally. Considering that site-specific data are almost always desired over generic models, the applicant needs to evaluate the strain level difference in the surface UHRS at different exceedance levels that result from application of these different models and to justify if the equal-weighting approach is appropriate.

Based on its review of SSAR Section 2.5.4.7, the related references, and the applicant's responses to RAIs described above, the staff concludes that the applicant did not have sufficient site-specific laboratory data to support the determination of the site response to dynamic loading. Although the applicant committed to provide the site-specific modulus reduction and damping curves during the COL stage, this issue, raised with a different perspective from the RAI 2.5.4-13 needs to be resolved in the ESP application to provide site-specific shear modulus reduction and damping curves for the site SSE determination. Therefore, as stated earlier, resolving this issue is designated as Open Item 2.5-19.

2.5.4.3.8 Liquefaction Potential

In its review of SSAR Section 2.5.4.8, the staff evaluated the applicant's description on liquefaction potential and its plan on future liquefaction study. The staff's review focused on the applicant's conclusion that liquefaction could occur only in the Upper Sand Stratum, based on the previous investigations and excavation completed for the VEGP Units 1 and 2.

In RAI 2.5.4-14, the staff asked the applicant to justify why it did not perform liquefaction analyses on the Blue Bluff Marl since the marl has a relatively high variable fines content (24–77 percent) and saturation level (14–67 percent) and a potentially high ground motion level at the site. In its response, the applicant first discussed the liquefaction potential for the Blue Bluff Marl (Lisbon Formation) based on its material and age and then examined the field strength and shear wave velocity results to determine whether the marl could liquefy based on these results.

The applicant stated that, although the Blue Bluff Marl frequently contained less than 50 percent of fine material, it had the appearance and characteristics of a calcareous claystone or siltstone and is a hard, slightly sandy, cemented calcareous clay. Its design undrained shear strength was set as 478.93 kPa (10000 psf) and its preconsolidation pressure could be as high as 3831.42 kPa (80000 psf) (i.e., the marl is a highly overconsolidated material). Although it would be below the ground water table, its compressed structure would prevent it from having the free water characteristic of a saturated granular material. The applicant then concluded that the Blue Bluff Marl is not a material with liquefaction potential, regardless of the ground motion level. The applicant further indicated that liquefaction resistance would increase markedly with geologic age. Based on Youd et al. (1997, 2001), Pleistocene (1.8 mya to 10,000 year) sediments were more resistant, and pre-Pleistocene sediments were generally immune to liquefaction. The Blue Bluff Marl's age is late middle Eocene (40 to 41 million years old), much older than Pleistocene.

The applicant also stated that, based on Youd et al. (2001), there are thresholds for the N-values, tip resistance, and shear wave velocity beyond which the material is considered nonliquefiable (e.g., a sand with 35 percent or more fines or a soil with a corrected N-value over about 21 are not liquefiable). Of the 58 N-values measured in the marl for the ESP investigation, 5 were below 50, ranging from 27 to 46. Thus, if the marl were a potentially liquefiable material, a liquefaction analysis would be run for these five samples. An initial analysis of these five samples showed factor-of-safety values in excess of the accepted 1.35 value in all cases. All of the CPTs that penetrated into the marl had refusal at or near the top of the stratum. Thus, measured tip resistance showed the material to be nonliquefiable. The typical shear wave velocities in the marl ranged from 426.72 to 807.72 m/s (1400 to 2650 ft/s). When corrected for overburden, these values would range from about 301.75 to 512.06 m/s (990 to 1680 ft/s). Youd et al. (2001) indicated that, for a sand with 35 percent or more fines, soils with a corrected shear wave velocity in excess of about 190.5 m/s (625 ft/s) were nonliquefiable.

The applicant stated that, based on material and age, the Blue Bluff Marl does not have the potential to liquefy, and the CPTs, as well as shear wave velocities, consistently indicated the marl is nonliquefiable material. In addition, the applicant pointed out that over 90 percent of the

SPT N-values indicated the marl as nonliquefiable material and the remaining N-values showed satisfactory factors of safety.

After review of the applicant's response, however, the staff is concerned that (1) the general observation of liquefaction occurrence with respect to age and material type does not exclude the liquefaction potential of the Blue Bluff Marl because of the limitation of the observations, such as the possible gravel engagement during the SPT and CPT tests, and (2) limited test data, including N-values, tip resistance, and shear wave velocity, cannot reliably exclude the liquefaction potential for the Blue Bluff Marl. The staff concludes that limited data prevented the applicant to make a conclusion on the liquefaction potential for the Blue Bluff Marl; therefore, the applicant does not have sufficient ESP soil property data to confirm that the Blue Bluff Marl is not liquefiable. This is designated as **Open Item 2.5-21**.

2.5.4.3.9 Earthquake Design Basis

SSAR Sections 2.5.2.6 and 2.5.2.7 present the applicant's derivation of the SSE, and Section 2.5.2.8 presents the OBE. Sections 2.5.2.3.6 and 2.5.2.3.8 of this SER provide the staff's evaluation of the applicant's determination of the SSE and OBE. Shear wave velocity profiles, soil modulus reduction, and damping curves described in Section 2.5.4 are critical inputs to the site seismic response and therefore to the SSE and OBE. This issue is discussed in detail in SER Section 2.5.2.

2.5.4.3.10 Static Stability

In its review of SSAR Section 2.5.4.10, the staff focused on the applicant's evaluation of bearing capacity and settlement of the bearing strata at the ESP site. The applicant used the following assumptions in calculating soil-bearing capacity and structure settlement — (1) placing all safety-related structures on the structural backfill above the Blue Bluff Marl after removal of the Upper Sand Stratum; (2) placing the base of the containment and auxiliary building foundations about 12.19 meters (40 ft) below final grade, or 15.3 to 18.3 meters (50 to 60 ft) above the top of the Blue Bluff Marl Stratum; and (3) placing other foundations in the power block area at depths of about 1.2 meters (4 ft) below final grade. The applicant modeled the containment building mat as a circle with a diameter of about 43.3 meters (142 ft) placed at a depth of 12.0 meters (39.5 ft) below finish grade in the calculations. The applicant determined that the allowable bearing pressure was 1470.3 kPa (30700 psf) under static loading conditions and 2203.1 kPa (46000 psf) under dynamic loading conditions. The settlement under an average bearing pressure of 239.5 kPa (50000 psf) was 41 mm (1.6 in.).

In RAI 2.5.4-15, the staff asked the following of the applicant:

1. Justify the adoption of the Peck et al. (1974) settlement and differential settlement values as guidelines which suggest total settlement of no more than 50 mm (2 in.), and differential settlement of no more than 19 mm (0.75 in.). For footings that support smaller plant components, the total settlement should be no more than 25 mm (1 in.), and the differential settlement no more than 13 mm (.5 in,).

2. Explain the main causes for exceeding these settlement values at the foundation levels of Units 1 and 2 and whether it would take any measures to prevent settlements and differential settlements for the new units.

3. Justify the use of an average bearing pressure of 239.5 kPa (50000 psf) for the settlement analyses of compacted fills

In its response to this RAI, the applicant stated the following:

1. The geotechnical community has widely accepted and used the Peck et al. (1974) total settlement guidelines of 25 mm (1 in.) for column footings and 50 mm (2 in.) for mats. When limiting foundation settlements to these values, differential settlements usually are very small. The applicant further stated that, even if these settlement values were exceeded, it would not necessarily have adverse effects on structures, especially for large mat foundations which can efficiently distribute structural loads to the soil. The applicant used the VEGP Units 1 and 2 as an example where the measured settlements of the containment buildings ranged from 102 to 109 mm (4 to 4.3 in.)

2. It (the applicant) will not use the settlement guidelines from Peck et al. (1974) for Units 3 and 4. The approach used for Units 3 and 4 consisted of estimating settlements for power block structures and using them as design values. The "VEGP Report on Settlement" prepared by Bechtel in 1986 provides comparisons of measured versus calculated settlements and concludes that the measured values did not exceed calculated or design values. The applicant would reanalyze and employ corrective measures in the event that monitored settlements exceed the design values. The applicant committed to follow the same approach for Units 3 and 4 and to revise SSAR Sections 2.5.4.10.2 and 2.5.4.11 accordingly in the next revision to the ESP application.

3. It (the applicant) used a bearing pressure value of 239.5 kPa (50000 psf) in foundation settlement analysis for illustrative purposes because no design value was available during the ESP. The applicant will revise the calculation using design values during the COL application.

After reviewing the responses, the staff concluded the following:

1. A primary concern of potential total and differential settlements is how these settlements compare with what the design of the reactor takes into consideration. It is important to compare the estimated settlements, which are appropriate for evaluation of the acceptability of the site at the ESP stage, with those incorporated into the plant design to evaluate the degree of conservatism because there will be severe impact to the safety of the SSCs once unexpected differential settlements occur.

2. The contact pressures associated with the planned reactor model are of interest and need to be considered at the ESP stage to estimate potential settlement. Since the data for a given reactor facility are available, the applicant should incorporate it into the site evaluation.

Based on the above considerations and in lieu of the fact that large settlements were observed at VEGP Units 1 and 2, the staff concludes that the applicant did not demonstrate quantitatively

whether the observed large settlement that occurred at the existing VEGP units will occur at the VEGP site and have no impact on the new units. This is **COL Action Item 2.5-8**.

In RAI 2.5.4-16, the staff asked the applicant to justify not analyzing the stability of all planned safety-related facilities in terms of bearing capacity, rebound, settlement, and differential settlements with the consideration of dead loads of fills and the reactor facility, as well as the lateral loadings. In its response, the applicant explained that this kind of information is not available at the ESP stage. Based on the applicant's response, the staff concludes that, since the applicant committed to provide more details regarding the bearing capacity, the staff agrees with the applicant that this information will not be available until the COL stage, and considers that this design-related information is not necessary to determine whether 10 CFR Part 100 is satisfied. This issue is designated as **COL Action Item 2.5-9**.

In RAI 2.5.4-18, the staff asked the applicant to provide detailed information on its determination of the allowable bearing capacity value. In its response, the applicant provided a detailed description of bearing capacity evaluations based on the Vesic (1975) formula. In addition, the applicant later clarified that the calculated value was net allowable bearing capacity, not the gross bearing capacity; therefore, the formula used in the actual calculation was slightly different from that presented in the reference. From its review of the applicant's response, the staff considers that the Vesic (1975) formula is based on primary assumptions of gross shear failure of soils under the foundation. Although this allowable bearing capacity formulation is applicable for general foundation analysis, it is inappropriate to use it in nuclear power plant foundation design. The control factors of allowable contact pressure for a large and heavy structure typically are not general shear failure but are (1) settlements; (2) allowable pressures used in design of the wall/basemat intersection; and (3) toe pressures developed during potential overturning and sliding of the facility. Based on the above considerations, the staff concludes that the allowable bearing capacity value provided by the applicant is not appropriate when considering the expected governing issues controlling the site evaluation. This is **Open Item 2.5-22**.

Based on its review of SSAR Section 2.5.4.10 and the applicant's responses to the RAIs, as described above, the staff concludes that the applicant did not provide an adequate preliminary assessment of the static stability of the ESP site. Although the applicant committed to provide more information in the COL phase, the applicant needs to resolve all open items related to some key issues, such as bearing capacity evaluation and settlement, at the ESP stage.

2.5.4.3.11 Design Criteria

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

In RAI 2.5.4-19(a), the staff asked the applicant to justify not providing complete design parameters or actual design methods that will be employed in the geotechnical review. In its response, the applicant stated that it would provide the information as part of the COL application, when the complete design criteria and actual design methods become available.

In RAI 2.5.4-19(b), the staff asked the applicant to explain why it did not provide factor of safety for slope stability in SSAR Section 2.5.5. In its response, the applicant stated that it would revise SSAR Section 2.5.5 in the next revision of the ESP application to include the factors of safety for slope stability. On the basis discussed above, the staff considers that this issue will not be resolved until the revised ESP application is submitted.

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 did not adequately present the necessary design criteria for the ESP site. The need for a COL or CP applicant to describe the design criteria and design methods, including the factor of safety for slope stability, is **COL Action Item 2.5-10**.

2.5.4.3.12 Techniques to Improve Subsurface Conditions

In SSAR Section 2.5.4.12, the applicant stated that it would not consider any ground improvement techniques beyond the removal and replacement of the Upper Sand Stratum and that it would consider additional ground improvement methods as warranted for specific locations of the project during the COL phase. The need for the COL or CP applicant to employ ground improvement (such as the planned engineering backfill to bring the bearing soil elevation up to 50 ft above the lower sand stratum) after removal of Upper Sand Stratum for the ESP site is **COL Action Item 2.5-11**.

2.5.4.4 Conclusions

Based on its review of SSAR Section 2.5.4, related references, and the applicant's responses to the associated RAIs described above, the staff finds the following:

1. The applicant conducted a limited site investigation to determine the engineering properties of subsurface soils at the ESP site. The applicant performed few field and laboratory tests to determine static and dynamic and other engineering properties of the underlying soils. Because of the quantity and quality of the test results, the applicant did not have sufficient data to determine the engineering properties reliably for the subsurface materials. Therefore, the applicant relied heavily on the previous database developed for the existing VEGP Units 1 and 2 without considering the different regulatory environments, investigation requirements, and testing technologies as well as the spatial variation of the soil properties between the two investigations.

2. The applicant provided a site-specific shear wave velocity profile in a situation that the shear wave velocity measured from the down-hole tests were lower than the ones obtained from the suspension P-S velocity measurements; the shear wave velocities from previous investigations associated with VEGP Units 1 and 2 were also lower. Additionally, the applicant did not perform soil dynamic testing on the samples from the ESP site to provide soil modulus reduction and damping curves to feed into the site response study and the site-specific shear wave velocity profile. The shear wave velocity profile and the shear modulus reduction curves, as well as the damping curves, are critical input for the site-specific ground motion spectrum and future SSI.

3. The proposed Units 3 and 4 will be located above the similar load-bearing strata, and the existing units already observed an unusually large settlement (both total and

differential). The applicant did not provide a detailed settlement analysis to ensure that the SSCs for a particular reactor model or envelope of various models (in the case that no definitive model is selected) are safe.

4. The applicant used some general methods in its calculations and analyses without thoroughly examining all important factors and considering all possible scenarios. For example, the applicant used the Vesic (1975) formula to determine the allowable bearing capacity of the bearing stratum. Since the formulation was based on primary assumptions of gross shear failure of soils under the foundation, it is applicable for general foundation analysis, but not appropriate for use in nuclear power plant foundation design because the typical control factors of the allowable contact pressure for such large and heavy structures are settlements, allowable pressures that are used in the design of the wall/basemat intersection, and toe pressures developed during potential overturning and sliding of the facility.

Based on the above findings, the staff concludes that, although the applicant conducted its own ESP site investigation and performed limited field and laboratory tests and associated analyses, it did not provide sufficient information to describe soil conditions underlying the ESP site, such as the possible existence of "soft zones" in the foundation-bearing layer. The applicant also did not demonstrate reliable engineering properties of the soils during the ESP site investigation. Instead of using its own data and referencing the previous investigation data, the applicant frequently defined the design parameters or input to the design criteria using the data from the previous investigation associated with VEGP Units 1 and 2 and used the ESP data as reference data. Therefore, some key engineering parameters do not reliably characterize the soil properties at the site and this affects the applicant's evaluation of important analyses, such as SSE determination, SSI analyses, soil allowable bearing capacity, and foundation settlement evaluations, as well as liquefaction potential and seismic stability analyses. Therefore, the applicant needs to resolve these issues and to characterize the site's geotechnical properties reliably.

2.5.5 Stability of Slopes

SSAR Section 2.5.5 describes the applicant's review of existing slopes at the ESP site and the applicant's plan for permanent cut and fill slopes during construction excavation. The applicant also discussed its plans for future slope stability analysis to take place during the design phase. The applicant did not perform slope stability analysis for the ESP site because there is no existing slope and the applicant cannot determine the future slope for the ESP phase.

2.5.5.1 Technical Information in the Application

The applicant stated that, since there were no existing slopes or embankments near the proposed location of VEGP Units 3 and 4, it did not perform a dynamic slope stability analysis. The applicant further stated that the site grading for construction of new units would result in nonsafety-related permanent cut and fill slopes. Permanent cut slopes would have a height of 15.2 meters (50 ft) or less and would be located several hundred meters away from planned or existing safety-related structures. Permanent fill slopes would have a height of 6.1 meters (20 ft) or less and would also be several hundred meters away from planned or existing safety-related structures. During the construction phase, the applicant will remove the soils

above the Blue Bluff Marl and replace them with compacted structural fill. The applicant stated that the construction excavation cut slopes would be temporary (i.e., only during the construction period) and that they will be far away from the safety-related structures of the existing VEGP Units 1 and 2. The applicant committed to perform nonsafety-related permanent slope stability analysis for dynamic and static conditions, as well as excavation cut slope analysis for static conditions during the design stage, to ensure that these slopes will not pose a hazard to the public.

2.5.5.2 Regulatory Basis

SSAR Section 2.5.5 states that the applicant did not perform a slope stability analysis for the ESP site application. However, the applicant stated in SSAR Section 1.8 that it followed the guidance of NUREG-0800, Section 2.5.5, when it described the slope-related issues in SSAR Section 2.5.5. In its review of SSAR Section 2.5.5, the staff considered the regulatory requirements in 10 CFR 100.23(c) and 10 CFR 100.23(d). According to 10 CFR 100.23(c), applicants must investigate the engineering characteristics of a site and its environs in sufficient scope and detail to permit an adequate evaluation of the proposed site. Pursuant to 10 CFR 100.23(d)(4), applicants must evaluate siting factors such as natural and artificial slope stability.

2.5.5.3 Technical Evaluation

The staff focused its review of SSAR Section 2.5.5 on whether there are any existing or planned new slopes that would adversely affect the safety-related structures of the proposed new units due to any possible loading conditions and/or natural events. After reviewing the information provided by the applicant, the staff concludes that, because there are no existing significant slopes near the proposed ESP site, a detailed slope stability analysis is not necessary at the ESP stage. The staff considers the creation of permanent slopes during construction to be a design-related issue, which must be addressed at the COL stage. The applicant committed to provide detailed slope stability analyses for the permanent slopes at the COL stage. On this basis, this issue is designated as **COL Action Item 2.5-12**.

2.5.5.4 Conclusions

Because there are no existing slopes near the ESP site, and the applicant did not know the exact location of the future reactors, the applicant did not perform any slope stability analysis. Although this information is not necessary to determine whether 10 CFR Part 100 is satisfied, excavation will create nonsafety-related permanent cut and fill slopes during the new units' construction stage, therefore, the COL applicant will address slope stability for the site at the COL stage.

2.5.6 Embankments and Dams

SSAR Section 2.5.6 presents a general description of existing and potential new embankments and dams at the ESP site.

2.5.6.1 Technical Information in the Application

SSAR Section 2.5.6 indicates that there are no earth, rock or earth, and rock fill embankments required for plant flood protection or for impounding the cooling water required for the operation of the plant. The applicant indicated that there are three existing nonsafety-related impoundments at the site—Mallard Pond, Debris Basin Dam 1, and Debris Basin Dam 2. The Mallard Pond is located to the north of the proposed switchyard, Debris Basin Dam 1 is located to the southeast of the proposed cooling towers, and Debris Basin Dam 2 is located to the southwest of the proposed cooling towers. The applicant stated that it would not use the impoundments for plant flood protection or for impounding cooling water for the operation of the plant. The pool level in Mallard Pond is below the elevation of 38.1 meters (125 ft) above msl. In the event of a dam breach at Mallard Pond, the water would drain to the north and away from the proposed new units. The pool levels in Debris Dams 1 and 2 are also below the elevation of 45.7 meters (150 ft) above msl, and, in the event of a dam breach, the water would drain to the south, away from the proposed new units. Therefore, the applicant concluded that there would be no need for embankments or dams for flood protection or for impounding the cooling water at the site.

2.5.6.2 Regulatory Basis

The applicant did not state which regulations SSAR Section 2.5.6 addressed; these topics are covered in NUREG 0800, Sections 2.4.4 and 2.5.5. However, in SSAR Section 1.8, Table 1-2, the applicant stated that it used RG 1.70 for guidance on format and content. Section 2.5.6 of RG 1.70 describes the necessary information and analysis related to the investigation, engineering design, proposed construction, and performance of all embankments used for plant flood protection or for impounding cooling water.

2.5.6.3 Technical Evaluation

In its review of SSAR Section 2.5.6, the staff evaluated the possible impact of a breach of existing embankments and dams on the proposed new units at the ESP site and evaluated the need for construction of any embankments or dams for flood protection. Based on the information provided by the applicant, the staff notes that the proposed finished grade elevation for the new units is approximately 67 meters (220 ft) above msl, and the existing pool levels for the three impoundments are 38.1 meters (125 ft) above msl for Mallard Pond, and 45.7 meters (150 ft) above msl for both Debris Basin Dams 1 and 2. These elevations are all below the proposed finished grade elevation. In addition, as the applicant discussed in Sections 2.4.3 and 2.4.4 of the SSAR, both probable maximum flood elevation (45.8 m (150.13 ft) msl) and the dam break level (54.3 m (178.10 ft) msl) are much lower than the proposed finished grade elevation Therefore, the staff concurs with the applicant's conclusion that no embankments and dams are required.

2.5.6.4 Conclusions

The applicant provided adequate information and analysis in SSAR Section 2.5.6, with reference to Sections 2.4.3 and 2.4.4 of the SSAR, regarding the embankments and dams at the ESP site. The applicant demonstrated that no embankments or dams are needed for flood

protection at the ESP site under possible flood and dam breach conditions because of the proposed finished grade elevation.