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2.5.4 Stability of Subsurface Materials and Foundations

This section presents information on the stability of subsurface materials and foundations at the VEGP site that may affect the proposed new unit's seismic Category 1 facilities. This geological, geophysical, geotechnical, and seismological information is developed and used as a basis to evaluate the stability of subsurface materials and foundations at the site. Field and laboratory test data was initially gathered during the ESP phase site investigation and subsequently augmented with field and laboratory data from a COL level investigation in support of the ESP limited work authorization (LWA) request.

Information presented in this section was developed from onsite geotechnical and geophysical investigations, a review of analysis and reports prepared for the existing VEGP units, and a review of geotechnical literature. Site specific reports prepared by Bechtel Power Corporation were included in this review; these reports addressed foundation investigation **(Bechtel 1974b)**, backfill material investigations **(Bechtel 1978a, 1978b and 1979)**, dynamic properties of the backfill **(Bechtel 1978c)**, and the test fill program **(Bechtel 1978d)**.

The ESP geotechnical field and laboratory investigation performed by MACTEC Engineering and Consulting, Inc. for the application was intended to enhance the understanding of the VEGP site and complement the existing geotechnical data developed for VEGP Units 1 and 2. The ESP geotechnical investigation data report is included as Appendix 2.5A. Portions of this geotechnical data report were revised by MACTEC. A discussion of these revisions is provided in Section 2.5.4.3.2.4. These revisions are reflected in Appendix 2.5A. The ESP seismic reflection/refraction data report is included as Appendix 2.5B.

A comprehensive site geotechnical field and laboratory investigation was performed by MACTEC to support the COL application. This investigation was conducted to augment the existing ESP geotechnical data and to further develop geotechnical data at specific proposed VEGP Units 3 and 4 structure locations and backfill borrow source locations. The COL investigation field work was substantially completed on April 20, 2007. The MACTEC geotechnical data report is included in Appendix 2.5C. A test pad program was conducted in late 2007. A report on this effort is provided in Appendix 2.5D.

2.5.4.1 Geologic Features

Section 2.5.1.1 describes the regional geology, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, and the regional tectonic setting. Section 2.5.1.2 addresses site-specific geology and structural geology, including site physiography and geomorphology, site geologic history, site stratigraphy, site structural geology, and a site geologic hazard evaluation.

2.5.4.2 Properties of Subsurface Materials

2.5.4.2.1 Introduction

This section describes the static and dynamic engineering properties of the VEGP site subsurface materials. An overview of the subsurface profile and materials is given in Section 2.5.4.2.2. The field investigations, described in Section 2.5.4.3, are summarized in Section 2.5.4.2.3. The descriptions of the subsurface materials provided in the following sections are based primarily on two recent field investigations, ESP and COL, and review of previous investigations. Within each section the ESP basis description is followed by the COL-basis description. The soils encountered during the ESP and COL subsurface investigations constitute alluvial and Coastal Plain deposits and can be placed in three groups for stability of subsurface materials and foundation purposes (i.e., for geotechnical purposes). These soils include, from top to bottom, sands with silt and clay (Group 1), clay marl (Group 2), and coarse-to-fine sand with interbedded thin seams of silt and/or clay (Group 3). The Upper Sand Stratum (Group 1 soils) will be completely removed and replaced with compacted structural fill prior to the construction of VEGP Units 3 and 4. The static and dynamic engineering properties of the three principal soil groups, along with the compacted structural fill, were determined by field investigation and laboratory testing. The laboratory tests and their results are summarized in Section 2.5.4.2.4. The engineering properties of the subsurface materials are presented in Section 2.5.4.2.5.

2.5.4.2.2 Description of Subsurface Materials

The site soils and bedrock are divided into five strata (Upper Sand Stratum, Marl Bearing Stratum, Lower Sand Stratum, Dunbarton Triassic Basin bedrock, and Paleozoic Crystalline bedrock), which correspond to the three soil groups mentioned in Section 2.5.4.2.1 plus the two bedrock units:

I. Upper Sand Stratum (Barnwell Group) – predominantly sands, silty sands, and clayey sands with occasional clay seams, soft zones, and shell zones. A shelly limestone (Utley Limestone) layer was encountered at the base of the Upper Sand Stratum or the top of the Blue Bluff Marl. The limestone contains shell zones, solution channels, cracks, and discontinuities. Severe fluid loss was observed in this layer during drilling for the ESP and COL subsurface investigations.

II. Marl Bearing Stratum (Blue Bluff Marl or Lisbon Formation) - slightly sandy, cemented, overconsolidated, calcareous silt and clay with some shells and partially cemented, well indurated layers.

III. Lower Sand Stratum (comprises several formations from the Still Branch just beneath the Blue Bluff Marl to the Cape Fear just above the Dunbarton Triassic Basin rock) – fine-to-coarse sand with interbedded silty clay and clayey silt.

IV. Dunbarton Triassic Basin Rock – red sandstone, breccia, and mudstone, weathered through the upper 120 ft.

V. Paleozoic Crystalline Rock – a competent rock with high shear wave velocities that underlies the non-capable Pen Branch Fault, which underlies the site.

These strata have been previously used as a means for classifying the soils and rock with regard to engineering properties, and are also used in this ESP SSAR.

The following sections provide brief descriptions of the subsurface materials, giving the soil and rock constituents, and their range of thickness encountered at the site. The information has been taken from the 14 borings and 10 cone penetrometer tests (CPT) performed during the ESP subsurface investigation. The locations of the ESP borings and CPTs are shown on Figure 2.5.4-1. Reference is made, as appropriate, to borings performed for VEGP Units 1 and 2. For reference, the VEGP site elevations in the areas explored range from about El. 219 to 256 ft msl, with a median of about El. 222 ft msl. It is noted that most of the VEGP ESP site is flat at about El. 220 ft msl with surrounding areas at higher elevations of about 250 ft msl. A finished plant grade of El. 220 ft msl is used for the new unit ESP analysis. The engineering properties are provided in Section 2.5.4.2.5. Figures 2.5.4-3, 2.5.4-4, and 2.5.4-5 provide illustrations of the subsurface conditions across the VEGP site observed in the ESP borings. A profile legend is provided as Figure 2.5.4-2.

Information produced from 70 borings and 8 CPTs, located in the immediate area of the nuclear islands, from the COL subsurface investigation has also been used to develop the following descriptions of the subsurface materials. The locations of the explorations performed for the COL investigation are shown on Figures 2.5.4-1a and 2.5.4-1b. Figures 2.5.4-3a, 2.5.4-5a, and 2.5.4-5b provide illustrations of the subsurface conditions across the Nuclear Islands (NIs) for Units 3 and 4, observed in the COL borings.

2.5.4.2.2.1 Upper Sand Stratum (Barnwell Group)

The ESP subsurface investigation (Appendix 2.5A) determined that the Upper Sand Stratum ranged in thickness from 78 to 157 ft beneath the ground surface at the completed boring locations. The wide range of thickness was due to two factors. First, three borings (B-1004, B-1005, and B-1006) were drilled from elevations about 30 ft higher than the remaining borings. Second, the top of the Blue Bluff Marl dips down toward the west and northwest portions of the VEGP site. The average thickness of the Upper Sand Stratum was 102 ft, and the median thickness was 94 ft at the ESP boring locations.

Field Standard Penetration Test (SPT) N-values obtained according to ASTM D 1586 **(ASTM D 1586 1999)** within the Upper Sand Stratum during the ESP subsurface investigation ranged from weight of rod (WOR) to 50 blows for 0-in. penetration (50/0”). The very high blow count values are indicative of zones containing the shelly limestone and shell hash. The average field SPT N-value was 25 blows per foot (bpf), and the median N-value was 21 bpf. These field values are un-corrected for hammer efficiency of the respective drill rig hammers used. Measurements of hammer energy were performed in borings B-1006 and B-1013. The measured energy transfer efficiency ranged from 65 to 87 percent, with an average value of 76 percent and a median value of 75 percent.

Selected samples recovered within the Upper Sand Stratum were submitted for laboratory testing, including percent fines, moisture content, and Atterberg Limits. The percent fines ranged from 3 to 60 percent, with an average value of 21 percent and a median value of 19 percent. The Plastic Limit ranged from 19 to 30, with an average value of 25 and a median value of 26. The Liquid Limit ranged from 43 to 97, with an average value of 62 and a median value of 53. The Plasticity Index ranged from 21 to 67, with an average value of 37 and a median value of 29. The natural moisture content of samples tested for Atterberg Limits ranged from 20 to 93 percent, with an average value of 63 percent and a median value of 70 percent.

Site geotechnical investigations for the existing units determined that the Upper Sand Stratum (Barnwell Group) is approximately 90 ft thick. A shelly limestone (Utley Limestone) is encountered at the base of this stratum and/or the top of the Blue Bluff Marl. The Upper Sand Stratum was determined to be susceptible to liquefaction during a seismic event equivalent to the safe shutdown earthquake (SSE) developed for VEGP Units 1 and 2. In addition, the underlying Utley limestone layer was determined to contain significant channeling, cracking, and other discontinuities. Therefore, it was considered necessary to remove both the Upper Sand Stratum and limestone layers before constructing VEGP Units 1 and 2. The standard penetration test data from previous studies indicate that the relative density of the Upper Sand Stratum is highly variable with a range from very loose to dense. Clay lenses encountered within the stratum ranged in consistency from soft to medium stiff.

Existing Units 1 and 2 unconsolidated undrained (UU) triaxial test results of samples within the Upper Sand Stratum indicate that the Mohr strength envelope of total stresses ranges from c=2,100 pounds per square foot (psf), ϕ=6° to c=440 psf, ϕ=32°, depending on the clay and sand content within the sample. Likewise, previous consolidated undrained (CU) triaxial test results for samples within the Upper Sand Stratum indicate that the Mohr strength envelope ranges from c=1,650 psf, ϕ=17° to c=4,000 psf, ϕ=25° for total stress and ϕ'=33° to ϕ'=34.5° for effective stresses. Because of the large number of UU and CU triaxial tests previously performed on Upper Sand Stratum samples, and the fact that this stratum would be completely removed before constructing the ESP units, no new strength tests were performed during the ESP subsurface investigation.

The COL subsurface investigation, Appendix 2.5C, with 70 borings located in the immediate area of the excavations for Units 3 and 4, was used to verify the characterization of the Upper Sand Stratum. From these data, the Upper Sand Stratum ranged in thickness from 81 to 97 feet with an average of 88 feet and a median thickness of 87 feet. One thousand four hundred and fourteen field SPT N-values were measured and ranged from WOR to 50/0" with a median of 18 bpf. Measurements of hammer energy were performed on each of the 12 drill rigs used for the COL investigation as presented in Appendix 2.5C. One hundred and sixty-eight sieve analyses disclosed a range of 5 percent to 96 percent fines with an average value of 22 percent and a median value of 20 percent. Seventeen Atterberg Limits test results on samples from the clay lenses disclosed an average liquid limit of 72, an average plastic limit of 33, and an average calculated plasticity index of 39. The moist unit weight of 15 samples was calculated and ranged from 94 pcf to 124 pcf with an average of 113 pcf and a median value of 113 pcf. The specific gravity of two samples was calculated as 2.69 and 2.75. Results of CU triaxial tests indicate average shear strength values of c=980 psf, ϕ=18° for total stress and c'=260 psf, ϕ=30° for effective stress.

The design properties of the Upper Sand Stratum are provided in Table 2.5.4-1 and were developed from laboratory and field test results from the ESP and COL investigations as previously described, and published engineering correlations.

2.5.4.2.2.2 Blue Bluff Marl (Lisbon Formation)

The ESP subsurface investigation (Appendix 2.5A) determined that the Blue Bluff Marl was found to range in thickness from 63 to 95 ft at three locations where the stratum was fully penetrated, with an average thickness of 76 ft and a median thickness of 69 ft. The typical thickness of the Blue Bluff Marl is illustrated on the subsurface profiles on Figures 2.5.4-3, 2.5.4 4, and 2.5.4-5. The profiles on Figures 2.5.4-3 and 2.5.4-4 also illustrate the downward dip of the top of the Blue Bluff Marl toward the west side of the VEGP site.

The data and laboratory test results from penetrations taken in the immediate area of the excavations for Units 3 and 4 for the COL subsurface investigation (Appendix 2.5C) were found to validate the ESP characterization of the Blue Bluff Marl except as noted in the following paragraphs. In the area of the excavations, the Blue Bluff Marl was penetrated at 42 of the 70 boring locations. The top of stratum elevation ranged from 122 ft to 140 ft with an average elevation of 132 ft. The thickness of the Blue Bluff Marl ranged from 60 ft to 77 ft with an average thickness of 67 ft and a median thickness of 68 ft. The representative thickness of the Blue Bluff Marl as determined by the COL borings is illustrated on the subsurface profile in Figure 2.5.4-3a.

Field SPT N-values obtained within the Blue Bluff Marl during the ESP subsurface investigation ranged from 26 bpf to 50 blows for 1-in. penetration (50/1"). The average field SPT N-value was 83 bpf, and the median N-value was 100+bpf. As noted in the revised MACTEC ESP Data Report (Appendix 2.5A) fossiliferous limestone, cemented layers, and cemented nodules were encountered in the Blue Bluff Marl. The high blow counts are attributed to the presence of these cemented layers as evidenced by the angular, gravel-sized, carbonate particles recovered in the split barrel samples. SPT blow counts corresponding to less than 12 in. of sampler penetration were linearly extrapolated to the 12 in. standard penetration. SPT blow counts that were linearly extrapolated to more than 100 bpf were truncated at 100 bpf when calculating SPT averages.

The field values are uncorrected for hammer efficiency of the respective drill rig hammers used. It is noted that the 26 bpf value was measured near the bottom of the stratum in boring B-1002, and most measured values were above 50 bpf. Also, the SPT N-values did not suggest the presence of a likely weathered portion at the top of the stratum.

During the COL investigation, 742 SPT samples were taken in the Blue Bluff Marl. Field SPT N-values ranged from 13 bpf to 50/0" with a median value of 71 bpf. The field values are uncorrected for hammer efficiency of the respective drill rig hammers used. Many SPTs did not achieve the full sampler penetration (e.g., 50 blows/3"). These high blow counts were attributed to the presence of abundant, partially cemented, well indurated layers as described in the MACTEC COL Data Report (Appendix 2.5C). Most of the measured N-values were greater than 30 bpf indicating hard to very hard consistencies. In addition, SPT N-values appear to behave as expected, increasing with depth. None of the 742 measured SPT N-values was less than 10 bpf, which is twice as much as one of the criterion used to identify soft zones at the nearby Savannah River Site (SRS) site (N<5bpf). A review of the borings logs did not reveal any layers below the Upper Sand stratum similar to the soft zones found at SRS. A summary of the SPT blow counts, corrected for hammer efficiency, collected from the borings within the power block is presented in Figure 2.5.4-18.

Selected samples recovered within the Blue Bluff Marl during the ESP subsurface investigation were submitted for laboratory testing, including percent fines, moisture content, and Atterberg Limits. The percent fines ranged from 17.8 to 97.8 percent, with an average value of 48 percent and a median value of 41 percent. The plastic limit ranged from non-plastic (NP) to 51 percent, with an average value of 29 percent and a median value of 27 percent. The liquid limit ranged from NP to 99 percent, with an average value of 51 percent and a median value of 43 percent. The plasticity index ranged from NP to 58 percent, with an average value of 22 percent and a median value of 16 percent. The natural moisture content of samples tested for Atterberg Limits ranged from 14 to 67 percent, with an average value of 35 percent and a median value of 29 percent. In addition, 15 UU tests were performed on Blue Bluff Marl samples. The laboratory measured undrained shear strength ranged from 150 to 4,300 psf. The low end of measured values (150 psf) is lower than previously reported (260 psf) for VEGP Units 1 and 2, and the high end of measured values (4,300 psf) is significantly lower than previously reported (500,000 psf) for VEGP Units 1 and 2. The SPT N-values measured during the ESP and values previously measured in the laboratory for VEGP Units 1 and 2 support the use of a 10,000-psf design value. The reason for the sharp disagreement between the ESP laboratory values and previously reported undrained shear strength for the Blue Bluff Marl is severe sample disturbance due to sampling technique (pitcher sampler) and preparation of testing specimen. The SPT N-values measured during the ESP and values previously measured in the laboratory for VEGP Units 1 and 2 support the use of a 10,000-psf design value.

Selected samples of the Blue Bluff Marl collected during the COL investigation subsurface investigation were submitted for laboratory testing and included percent fines, moisture content, and Atterberg Limits. Sieve analyses tests were conducted on 90 representative samples that disclosed a range of fines from 29 percent to 98 percent, with an average value of 74 percent and a median value of 75 percent. Atterberg Limit tests were conducted on 92 representative samples and disclosed a liquid limit range from 34 to 112 percent, with an average value of 67 percent and a median value of 63 percent. The plastic limit ranged from 20 to 64 percent, with an average value of 34 percent and a median value of 33 percent. The calculated plasticity index ranged from 11 to 62 percent, with an average value of 33 percent and a median value of 30 percent. The natural moisture content of samples tested ranged from 14 to 62 percent, with an average value of 33 percent and a median value of 32 percent. The moist unit weight of 69 samples ranged from 95 pcf to 133 pcf with an average of 115 pcf and a median value of 115 pcf. The specific gravity of 8 samples was calculated and ranged from 2.61 to 2.66 with an average value of 2.64.

Site investigations for the existing units determined that the marl stratum (Blue Bluff Marl or Lisbon Formation) consists of hard, slightly sandy, cemented, overconsolidated, calcareous clay and ranges in thickness from approximately 60 ft to 100 ft. The comparative consistency of the Blue Bluff Marl ranges from hard to very hard. The materials are moderately brittle and resemble a calcareous claystone or siltstone. Previous seismic exploration within this stratum indicates a velocity interface approximately 15 ft beneath the top of the stratum. The upper 15 ft, a likely weathered portion, of the stratum recorded a compressive wave velocity of approximately 5,000 ft per second (fps), while the underlying material recorded a compressive wave velocity of approximately 7,000 fps. The static engineering properties of the Blue Bluff Marl stratum are summarized in Table 2.5.4-1.

Previous laboratory results indicate the Blue Bluff Marl to be highly overconsolidated. Plasticity index values ranged from 2 to 70 with an average value of 25. Based on work by Skempton **(1957)**, using the average PI value yields an s_u/p ratio of approximately 0.2, where s_u is undrained shear strength and p is the effective overburden pressure for a normally consolidated clay. An undrained shear strength of 16,000 psf was determined using the average value of shear strength test results which failed at less than 50,000 psf. However, given a shear strength (s_u) one can use the same relationship and compute p (in the case of an overconsolidated deposit, p would be equivalent to the preconsolidation pressure). Therefore, using the 16,000 psf value for undrained shear strength and a s_u/p ratio of 0.2, the preconsolidation pressure of the Blue Bluff Marl stratum was estimated to be 80,000 psf (an OCR of about 8). Settlements due to loadings from new structures would be small due to this high preconsolidation pressure, demonstrated by the settlement measurement for Units 1 and 2 as discuss in Section 2.5.4.10.2.

The undrained shear strength of the Blue Bluff Marl was verified during the excavation for VEGP Units 1 and 2. Core samples of the Blue Bluff Marl were obtained and tested. The design value of c = 10,000 psf, ϕ = 0° was found to be appropriately conservative. The average undrained shear strength of the core samples was 20,000 psf, and the lowest value obtained was 11,700 psf.

The heave of the Blue Bluff Marl stratum was monitored during the excavation for VEGP Units 1 and 2. Measurements were taken at nine locations at regular intervals. After excavation completion, an average heave of 1.25 in. was observed. Based on the heave measurements, the undrained Young's modulus, E, of the Blue Bluff Marl stratum was calculated to be 10,000 kips/ft^2, similar to values of E estimated from Menard pressuremeter and seismic velocity measurements during previous field investigations.

Strength tests were conducted in the laboratory during the COL investigation (Appendix 2.5C) on relatively undisturbed (intact) samples of the Blue Bluff Marl. Strength testing included 27 unconfined compression tests, 11 UU triaxial tests, and 27 CU triaxial tests. Eighteen consolidation tests were conducted. The UU and CU triaxial tests were conducted at various confining pressures. Test results disclosed that the shear strength of the BBM increased with increasing confining pressure, as expected. The Blue Bluff Marl is approximately located from a depth of 90 to 165 ft with a design ground water level at a depth of 55 feet. Based on this overburden condition, the range of confining pressures in the Blue Bluff Marl is between 6.5 ksf and 9.7 ksf. UU test results at a confining pressure of 8.1 ksf disclosed a minimum undrained shear strength of 1.7 ksf and a maximum of 11.7 ksf with an average value of 6.5 ksf. The CU test results disclosed a minimum undrained shear strength of 2.8 ksf and a maximum value of 32.2 ksf with an average value of 9.3 ksf in this range of confining pressure. Given that the Blue Blue Marl is characterized as an overconsolidated, calcareous clay, the undrained shear strength can be represented by considering the preconsolidaton pressure. At a confining pressure of 16 ksf (the upper limit of the UU and CU test program) which represents approximately twice the in-situ confining pressure, UU test results disclosed an average undrained shear strength of 8.6 ksf and CU test results disclosed an average value of 14.9 ksf. The averaged undrained shear strength from the UU and CU tests is 11.8 ksf, which supports the design value of 10,000 psf used for VEGP Units 1 and 2.

Consolidation tests were conducted in the laboratory during the COL investigation (Appendix 2.5C) on 18 relatively undisturbed samples of the Blue Bluff Marl. The compression and recompression ratios were determined from the compression and recompression indices provided in the test results. Compression ratios ranged from 0.034 to 0.156 with an average value of 0.094. Recompression ratios ranged from 0.004 to 0.017 with an average of 0.010.

The static design properties of the Blue Bluff Marl stratum for VEGP Units 3 and 4 are provided in Table 2.5.4-1 and were developed from laboratory and field test results from ESP and COL subsurface investigations, available data from VEGP Units 1 and 2, as well as published engineering correlations.

A summary of the design dynamic shear modulus at strain levels of 10^{-4} percent, or lower, for the Blue Bluff Marl stratum, based on ESP investigation, is given in Table 2.5.4-2. Dynamic shear modulus values were computed from the in situ shear wave velocity measurements shown in Table 2.5.4-6. Additional in situ shear wave velocity measurements were taken during the COL investigation. These data, combined with ESP data as described in Section 2.5.4.7.1, are shown in Table 2.5.4-11a.

2.5.4.2.2.3 Lower Sand Stratum

The ESP subsurface investigation (Appendix 2.5A) determined that the Lower Sand Stratum encompassed a number of geologic formations, including, listed in top to bottom order, the Still Branch, Congaree, Snapp, Black Mingo, Steel Creek, Gaillard/Black Creek, Pio Nono/Unnamed, and Cape Fear formations. The Lower Sand Stratum was fully penetrated at boring B-1003 and found to have a thickness of 900 ft at this location. Boring B-1003 also disclosed that the Lower Sand Stratum rests upon Dunbarton Triassic Basin rock. Typical depths are illustrated on the subsurface profile in Figure 2.5.4-4.

Field SPT N-values obtained to depths of about 300 ft within the Lower Sand Stratum during the ESP subsurface investigation ranged from 9 bpf to 50 blows for 4-in. penetration (50/4”). The average field SPT N-value was 59 bpf, and the median N-value was 47 bpf. These field values are uncorrected for hammer efficiency of the respective drill rig hammers used and comprise values measured mostly in the Still Branch Formation directly beneath the Blue Bluff Marl.

During the COL investigation, the Lower Sand Stratum was encountered in 42 of the COL borings with 36 nominal penetrations (one to 7 feet) and 6 substantial penetrations (84 to 263 feet) into this stratum. The maximum depth of penetration into the stratum was 263 ft in B-3001. One hundred and eleven field SPT N-values obtained in this stratum ranged from weight of hammer (WOH) to 50/1". The median field SPT N-value was 70 bpf. These field values are uncorrected for hammer efficiency of the respective drill rig hammers used and comprise values measured mostly in the Still Branch Formation directly beneath the Blue Bluff Marl. Nearly all of the N-values taken in the Lower Sand stratum are greater than 30 bpf indicating dense to very dense material. In addition, SPT N-values appear to behave as expected, increasing with depth. One N-value (B-4001, SS-38: WOH/18) taken in this stratum indicated very loose material. This sample was taken in the Still Branch Formation of the Lower Sand strata at an elevation of -41.5 to -43 feet. No recovery was obtained in the split barrel sample. An undisturbed sample (UD-11) was attempted prior to SS-38 from elevation -39.5 to -41.5 feet and no recovery was obtained in this sample. The material above this elevation was identified as light gray SAND (SP). The difficulty in sampling this material along with the weight of hammer reading in SS-38 is considered an anomaly and can be attributed to disturbed soil conditions at the bottom of the borehole. These conditions are likely the result of a hydrostatic pressure imbalance between the borehole and the in situ hydrostatic pressure. The resulting imbalance likely caused a quick condition to develop in the poorly graded sands at the sampling depth. Such quick conditions are difficult to sample, as evidenced by the lack of sample recovery in SS-38 and UD-11, as the now disturbed poorly graded sand will flow out of the sampler. Besides this anomalous condition, no other evidence of soft zones or loose material was encountered in Lower Sand stratum.

ESP subsurface investigation selected samples recovered within the Lower Sand Stratum were submitted for laboratory testing, including percent fines, moisture content, and Atterberg Limits. The percent fines ranged from 3 to 80 percent, with an average value of 23.6 percent and a median value of 15 percent. The plastic limit ranged from NP to 38 percent, with average and median values of 30 percent. The liquid limit ranged from NP to 53 percent, with average and median values of 47 percent. The plasticity index ranged from NP to 19 percent, with average and median values of 17 percent. The natural moisture content for samples tested for Atterberg Limits ranged from 21 to 41 percent, with an average value of 30 percent and a median value of 28 percent. Samples with the higher percent fines and plasticity were from the silty clay and clayey silt layers in the Congaree and Snapp Formations within the Lower Sand Sratum.

Representative samples of the Lower Sand Stratum collected during the COL subsurface investigation were submitted for laboratory testing. Sieve analyses were conducted on 14 samples with a range from 5 to 70 percent fines, an average value of 23 percent, and a median value of 17 percent. The moist unit weight of 16 samples was calculated ranged from 113 pcf to 136 pcf with an average of 123 pcf and a median value of 122 pcf. The average specific gravity of four samples was calculated as 2.67. Results of CU triaxial tests indicate average shear strength values of c=4,725 psf, ϕ=26° for total stress and c'=215 psf, ϕ'=36° for effective stress.

During the COL investigation, alternating layers of fine-grained and coarse-grained soils were identified in the upper and lower portions of the Congaree Formation and in the upper portion of the Snapp Formation. These fine-grained soils were characterized as silts and clays and, where encountered, were on the order of 20 to 40 feet thick. The coarse-grained soils were characterized as silty to clayey, and poorly-graded sands. In boring B-3001, the fine-grained materials in the lower portion of the Congaree and the upper portion of the Snapp were on the order of 70 feet thick.

Site geotechnical investigations for the existing units determined that the Lower Sand Stratum consists of sands with interbedded silty clay or clayey silt. The thickness of this stratum was estimated to be 900 to 1,000 ft. SPT N-values obtained to depths of about 300 to 400 ft below grade during previous field investigations within the Lower Sand Stratum ranged from 70 to 100 bpf, indicative of a very dense material.

The static design properties of the Lower Sand Stratum for VEGP Units 3 and 4 are provided in Table 2.5.4-1 and were developed from laboratory and field test results from ESP subsurface investigations, available data from VEGP Units 1 and 2, as well as published engineering correlations.

A summary of the design dynamic shear modulus at strain levels of 10^{-4} percent, or lower, for the Lower Sand Stratum, based on the ESP investigation, is given in Table 2.5.4-2. Dynamic shear modulus values were computed from the in situ shear wave velocity measurements shown in Table 2.5.4-6. Additional in situ shear wave velocity measurements were taken during the COL investigation. These data, combined with ESP data as described in Section 2.5.4.7.1, are shown in Table 2.5.4-11a.

2.5.4.2.2.4 Dunbarton Triassic Basin Rock

The Dunbarton Triassic Basin Rock was cored at ESP borehole B-1003 only, and consisted of red sandstone, breccia, and mudstone, weathered through the upper 120 ft. The deepest COL borehole was advanced to a depth of 263 ft into the Lower Sand Stratum for a total depth of 420 ft and did not reach bedrock. Further details are provided in Section 2.5.1. Because the rock was too deep to be of any interest to foundation design, no laboratory tests were performed on the rock cores. Shear wave velocity was measured in the upper 274 ft of the rock profile, and these results were used to develop the shear wave velocity profile for site amplification that are presented in Section 2.5.4.7.1.

2.5.4.2.2.5 Paleozoic Crystalline Rock

As indicated in Figure 2.5.4-4, the VEGP site sits on over 1,000 feet of Coastal Plain sediments underlain by Triassic Basin sedimentary rock. Borehole B-1003 encountered the bottom of the Coastal Plain sediments and the start of a weathered section of the Triassic Basin at a depth of 1,049 feet. Under the part of Savannah River Site adjacent to the VEGP site, the southeast dipping Pen Branch fault separates the Triassic Basin rock from Paleozoic crystalline rock to the northwest (**Lee et al. 1997**). A seismic reflection survey in and around the VEGP site (shown in Appendix 2.5B and discussed in section 2.5.1.2.4.2), has been interpreted to show the southwest continuation of the Pen Branch fault beneath the site and to indicate that the depth to the bottom of the Coastal Plain sediments is about 1,000 feet (Figure 2.5.1-40). This and interpretation of flexures within the older Coastal Plain sediments suggest that the Pen Branch fault lies below the area of the new containment units. Therefore, the information available implies that at some depth below the VEGP site the Paleozoic crystalline rock underlies the Triassic Basin rock.

2.5.4.2.2.6 Subsurface Profiles

Figures 2.5.4-3, 2.5.4-4, and 2.5.4-5 illustrate typical subsurface profiles across the power block area proposed for the proposed VEGP Units 3 and 4 based on ESP borings. A profile legend is provided as Figure 2.5.4-2. The locations of the borings used to develop profiles are shown in Figure 2.5.4-1. Figures 2.5.4-3a, 2.5.4-5a, and 2.5.4-5b illustrate typical subsurface profiles across the limited power block area proposed for VEGP Units 3 and 4 based on COL borings. The locations of the borings used to develop profiles are shown in Figure 2.5.4-1b. These profiles are discussed in Section 2.5.4.5 with respect to excavation for the new units and in Section 2.5.4.10.1 for bearing capacity considerations.

2.5.4.2.3 Field Investigations

The exploration programs performed previously for VEGP Units 1 and 2 are referenced, as warranted. The ESP and COL subsurface investigations are described in Section 2.5.4.3. The boring logs from previous explorations are not included here; however, the locations of referenced borings from VEGP Units 1 and 2 are provided on Figure 2.5.4-1b. The borings and cone penetrometer tests from the ESP subsurface investigation program are summarized in Tables 2.5.4-7. Previous geophysical surveys and new geophysical surveys for the ESP investigation are described in Section 2.5.4.4. Boring logs and CPT logs from the ESP field exploration are included in Appendix 2.5A.

The exploration program for the COL subsurface investigation included borings, CPTs, seismic CPTs, geophysical surveys, and test pits. The boring, CPT, and test pit locations are summarized in Table 2.5.4-7a and illustrated on Figures 2.5.4-1a and 2.5.4-1b. Geophysical surveys for the COL investigation are described in Section 2.5.4.4. Boring logs, CPT logs, geophysical survey results, and test pit logs from the COL field exploration are included in Appendix 2.5C.

Field investigations also included the construction and testing of a 20-foot thick below grade test pad using proposed borrow materials. This program was conducted to aid in evaluating the static and dynamic properties of the compacted backfill. Additional details of the program are provided in Section 2.5.4.5.3. Results of this program are included in Appendix 2.5D.

2.5.4.2.4 Laboratory Testing

2.5.4.2.4.1 Testing Overview

Numerous laboratory tests of soil samples were performed previously for VEGP Units 1 and 2, and new tests have been performed as part of the ESP and COL subsurface investigations. Previous test results are contained within Bechtel Power Corporation's Report on Foundation Investigations **(Bechtel 1974b)**. The types and numbers of tests completed during the ESP subsurface investigation are shown in Table 2.5.4-3, and the test results are contained within the MACTEC report for the ESP subsurface investigation (Appendix 2.5A). A summary of all laboratory test results performed as part of the ESP subsurface investigation is provided in Table 2.5.4-4. The types and numbers of tests completed during the COL subsurface investigation are shown in Table 2.5.4-3a and the test results are contained within the MACTEC data report for the COL subsurface investigation (Appendix 2.5C). Results of resonant column torsional shear (RCTS) tests conducted on samples from the COL investigation are included in Attachment G of Appendix 2.5C. Laboratory tests, including RCTS, were also conducted as part of the Phase I test pad program and are included in Appendix 2.5D.

2.5.4.2.4.2 Laboratory Tests for the ESP Subsurface Investigation

Laboratory testing for the ESP investigation was performed in accordance with the guidance presented in Regulatory Guide 1.138, Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants, US Nuclear Regulatory Commission, 2003 (RG 1.138). The laboratory work was performed under an approved quality program with work procedures developed specifically for the ESP application. Soil samples were shipped under Chain-of-Custody protection from the on-site storage area (described in Section 2.5.4.3.2) to the testing laboratory. Laboratory testing was performed at the MACTEC laboratories in Atlanta, Georgia.

The types and numbers of laboratory tests performed on the soil samples from the ESP exploration program are included on Table 2.5.4-3. The ESP tests focused primarily on verifying the basic properties of the Upper Sand Stratum, Blue Bluff Marl, and the upper formations in the Lower Sand Stratum.

The details and results of the laboratory testing are included in Appendix 2.5A. This appendix includes references to the industry standard used for each specific laboratory test. The results of the tests on soil samples are shown on Table 2.5.4-4.

2.5.4.2.4.3 Laboratory Tests for the COL Subsurface Investigation

Laboratory testing for the COL investigation was performed in accordance with the guidance presented in RG 1.138. The laboratory work was performed under an approved quality assurance program with work procedures developed specifically for the COL application. Soil samples were shipped using Chain-of-Custody tracking procedures from the on-site storage area (described in Section 2.5.4.3.3) to the testing laboratory. Laboratory tests were performed at various laboratories. RCTS tests were performed at the FUGRO laboratories in Houston, Texas. The details and results of the laboratory testing are included in Appendix 2.5C. This appendix includes references to the industry standard used for each specific laboratory test.

2.5.4.2.4.4 Laboratory Test for the Phase I Test Pad Program

Laboratory testing was conducted during construction of the test pad under an approved quality assurance program with work procedures developed specifically for the test pad program. An onsite laboratory was established to conduct most of the testing. Soil samples that were shipped to offsite laboratories were done so using Chain-of-Custody procedures. RCTS testing was conducted offsite at laboratories in Houston, Texas and Austin, Texas. Details and results of the laboratory testing are included in Appendix 2.5D.

2.5.4.2.5 Engineering Properties

The engineering properties for the Upper Sand, Blue Bluff Marl, Lower Sand Strata, and compacted structural fill, as provided in Table 2.5.4-1, were derived from the ESP subsurface investigation and laboratory testing program and from previous VEGP site studies. A similar table, Table 2.5.4-1a, was derived from the COL investigation and Phase I test pad program. The COL data, used as the basis for these properties, was taken from the borings in the immediate vicinity of the combined NI (power block) excavation footprint. The engineering properties of the proposed borrow materials that were developed from the COL data are presented in section 2.5.4.5.3. The engineering properties for the structural backfill were derived from the COL investigation and the Phase I test pad program. The engineering properties developed from the ESP and COL subsurface investigation and laboratory testing programs and the Phase I test pad program (Appendix 2.5A, 2.5C, and 2.5D, respectively) were similar to those obtained from the previous VEGP Units 1 and 2 field and laboratory testing programs.

Rock densities were derived from Tables 5-2 and 5-3 of WSRC **(1998)** for crystalline and Triassic rock, respectively. Rock densities increased with depth from 2.75 gm/cc (171.6 pcf) to 3.42 gm/cc (213.4 pcf) in the crystalline rock, and from 2.53 gm/cc (157.9 pcf) to 3.42 gm/cc (213.4 pcf) in the Triassic rock.

The following sections briefly describe the sources and/or methods used to develop the selected properties shown in Table 2.5.4-1 and Table 2.5.4-1a.

2.5.4.2.5.1 Rock Properties

The Recovery and Rock Quality Designations (RQD) are based on the results provided from the deep boring, B-1003. Rock coring was not performed during the previous investigations for VEGP Units 1 and 2. Geophysical testing at the deep boring, B-1003, extended for about 290 ft into the bedrock encountered at depth of 1,049 ft below the ground surface. The shear and compressional wave velocities are based on the suspension P-S velocity seismic test performed in borehole B-1003 as part of the ESP subsurface investigation (Appendix 2.5A). Laboratory strength testing of rock cores was not performed because the rock is deemed to be too deep to provide any additional useful engineering information.

2.5.4.2.5.2 Soil Properties

The properties of the soils underlying the site were developed from ESP and COL investigations, including laboratory testing programs, review of previous investigations for VEGP Units 1 and 2, and the Phase I test pad program. The following paragraphs describe the properties of the Upper Sand, Blue Bluff Marl, and Lower Sand Strata. The properties of the proposed compacted structural backfill are discussed in Section 2.5.4.5.3.

Sieve analyses of 29 Upper Sand Stratum samples (including 1 fill sample), 28 Blue Bluff Marl samples, and 14 Lower Sand Stratum samples were performed as part of the ESP laboratory testing program (Appendix 2.5A).

The natural moisture content and Atterberg Limits of 4 Upper Sand Stratum, 20 Blue Bluff Marl, and 4 Lower Sand Stratum samples were determined as part of the ESP laboratory testing program. Design values shown on Table 2.5.4-1 were taken as the average of these test results for the respective soil strata.

The COL laboratory testing program (Appendix 2.5.C) included sieve analyses of 168 Upper Sand Stratum samples, 90 Blue Bluff Marl samples, and 14 Lower Sand Stratum samples. Atterberg Limits tests were conducted on 17 Upper Sand Stratum samples and 92 Blue Bluff Marl samples. Specific gravity measurements were made on two Upper Sand Stratum samples, 8 Blue Bluff Marl samples, and 4 Lower Sand Stratum samples.

The undrained shear strength of the Blue Bluff Marl bearing stratum was determined from laboratory test data, SPT N-values, and shear wave velocity measurement. Laboratory test data included unconsolidated undrained triaxial test and consolidated undrained triaxial test results from the ESP and COL investigations. Laboratory strength testing during previous investigations as well as during the construction of VEGP Units 1 and 2 were also reviewed.

The effective angle of internal friction of the Upper Sand Stratum was determined to be 34 degrees **(Bowles 1982)** from correlation with the average SPT N-value from the ESP investigation (based on N_{60} = 25 bpf). The N-value of 25 bpf represents the measured value of 20 bpf corrected to account for the higher automatic hammer efficiency measured in the field. This correction was made following the guidelines in ASTM D 6066 (1996). The median measured N-value from the COL investigation was 18 bpf, corresponding to a N_{60}-value of 25 bpf.

The effective angle of internal friction of the Lower Sand Stratum was determined to be 41 degrees **(Bowles 1982)** from correlation with the average SPT N-value from the ESP investigation (based on N_{60} = 62 bpf). The N-value of 62 bpf represents the measured value of 50 bpf corrected to account for the higher automatic hammer efficiency measured in the field. This correction was made following the guidelines in ASTM D 6066 (1996). The median measured N-value from the COL investigation was 70 bpf, corresponding to a N_{60}-value of 94 bpf.

Moist unit weights were measured in selected samples from the ESP laboratory testing program of the Blue Bluff Marl and Lower Sand Stratum. The unit weights of 15 Blue Bluff Marl samples ranged from 102 pounds per cubic foot (pcf) to 140 pcf, with an average of 120 pcf. Unit weights of three Lower Sand Stratum samples were 119.4 pcf, 121.7 pcf, and 128.3 pcf, with an average of 123 pcf.

The COL laboratory testing program included moist unit weight measurements of 15 samples in the Upper Sand Stratum, 69 in the Blue Bluff Marl, and 16 in the Lower Sand Stratum. The values in the Upper Sand Stratum ranged from 94 to 124 pcf with an average value of 113. The values in the Blue Bluff Marl ranged from 95 to 133 pcf with an average value of 115 pcf. The values in the Lower Sand Stratum ranged from 113 to 134 pcf with an average of 123 pcf.

The in situ moist unit weights of the Upper Sand Stratum, Blue Bluff Marl, and Lower Sand Stratum for VEGP Units 1 and 2 were 118 pcf, 119 pcf, and 117 pcf, respectively.

The design SPT N-value for the Upper Sand Stratum is taken as 25 bpf. This value is based on the ESP results reported in Table 2.5.4-5 and includes correction for hammer efficiency. The results in Table 2.5.4-5 show an average uncorrected field SPT N-value of 25 bpf and median value of 21 bpf. The design corrected N-value of 25 bpf corresponds to a field N-value of 20 bpf, which is lower than the average and median values. The median uncorrected field SPT N-value for the Upper Sands from the COL investigation was 18 bpf. SPT N values for VEGP Units 1 and 2 ranged from 2 to 60 bpf with an average of 30 bpf. The design value is within the range and near the average of the COL investigation and previous investigation values.

The design SPT N-value for the Blue Bluff Marl is taken as 100 bpf. This value is based on the results reported in Table 2.5.4-5 and includes correction for hammer efficiency. The results in Table 2.5.4-5 show an average uncorrected field SPT N-value of 83 bpf and median value of 100 bpf. The design corrected N-value of 100 bpf corresponds to a field N value of 80 bpf, which is lower than the average and median values. The median uncorrected field SPT N-value for the Blue Bluff Marl from the COL investigation was 71 bpf (N_{60}=95bpf). SPT N-values for VEGP Units 1 and 2 ranged from 10 to over 100 bpf with an average of over 100 bpf. The design value is within the range and near the median of the COL investigation and previous site investigation values.

The design SPT N-value for the Lower Sand Stratum is taken as 62 bpf. This value is based on the results reported in Table 2.5.4-5 and includes correction for hammer efficiency. The results in Table 2.5.4-5 show an average uncorrected field SPT N-value of 59 bpf and median value of 47 bpf. The design corrected N-value of 62 bpf corresponds to a field N-value of 50 bpf, which is lower than the average value and slightly higher than the median value. The median uncorrected field SPT N-value for the Lower Sands from the COL investigation was 70 bpf (N_{60}=94bpf). SPT N-values for VEGP Units 1 and 2 ranged from 70 to 100+ bpf with an average of 100+ bpf. The design value is somewhat less than the previous site investigations range of values.

Shear wave velocities were measured by suspension P-S velocity tests and seismic CPTs during the ESP and COL subsurface investigations (Appendix 2.5A and Appendix 2.5C, respectively). The suspension P-S velocity tests were performed in 5 boreholes for the ESP investigation, although only three of these tests extended into the Blue Bluff Marl and Lower Sand Strata. P-S velocity tests were performed in 6 boreholes for the COL investigation. Three seismic CPTs were performed in accordance with ASTM D 5778 (2000) for the ESP investigation and 8 for the COL investigation. Due to penetration resistance, the seismic CPT tests did not extend into the very hard underlying Blue Bluff Marl stratum. Further discussion of suspension P-S velocity and seismic CPT testing is provided in Section 2.5.4.4.

A complete shear wave velocity profile was developed during the ESP investigation from the ground surface to about 300 ft into the Dunbarton Triassic Basin rock for a total depth of about

1,340 ft using both suspension P-S velocity and seismic CPT testing taken during the ESP investigation. Shear wave velocities within the Upper Sand Stratum ranged from about 570 fps to 3,310 fps. Shear wave velocities ranged from 1,060 fps to 4,260 fps within the Blue Bluff Marl stratum, 930 fps to 4,670 fps within the underlying Lower Sand Stratum, and 2,320 fps to 9,350 fps within the Dunbarton Triassic Basin. Shear wave velocity measurements were made to depths of up to 290 ft during previous investigations for VEGP Units 1 and 2. In addition, shear wave velocity data were reviewed from seven deep borings performed at the neighboring Savannah River Site. Typical shear wave velocity values were determined for the Upper Sand Stratum, Blue Bluff Marl, Lower Sand Stratum, and the Dunbarton Triassic Basin rock data based upon review of all the available data and are provided in Table 2.5.4-6. Shear wave velocity values within the Lower Sand Stratum were determined for each of the geologic formations contained within. A more detailed discussion of shear wave velocity values and establishment of the shear wave velocity profile for site amplification are presented in Section 2.5.4.7.1.

Shear wave velocity measurements were made during the COL investigation to a maximum depth of 420 feet. Shear wave velocities within the Upper Sand Stratum disclosed an average shear wave velocity of 940 fps. An average shear wave velocity of 2,050 fps was disclosed in the Blue Bluff Marl stratum. The shear wave velocity measurements in the Lower Sand stratum disclosed an average shear wave velocity of 1,750 fps. A more detailed discussion of shear wave velocity values and establishment of the shear wave velocity profile for site amplification are presented in Section 2.5.4.7.1.

The high strain (i.e., in the range of 0.25 to 0.5 percent) elastic modulus values, tabulated in Table 2.5.4-1, for the Upper Sand Stratum and Lower Sand Stratum have been derived using the relationship with the SPT N-value given in **Davie and Lewis (1988)**. The high strain elastic modulus for the Blue Bluff Marl stratum has been derived using the relationship with undrained shear strength given in **Davie and Lewis (1988)**. The shear modulus values have been obtained from the elastic modulus values using the relationship between elastic modulus, shear modulus, and Poisson's ratio **(Bowles 1982)**.

The low strain (i.e., 10^{-4} percent) shear modulus, tabulated in Table 2.5.4-2, for the Upper Sand Stratum has been derived from the average shear wave velocity of 940 fps. The low strain shear modulus of the Blue Bluff Marl stratum has been derived from the average shear wave velocity of 2,050 fps. The low strain shear modulus of the Lower Sand Stratum has been derived from the average shear wave velocity of 1,750 fps. The elastic modulus values have been obtained from the shear modulus values using the relationship between elastic modulus, shear modulus, and Poisson's ratio **(Bowles 1982)**. The low strain shear modulus for the compacted backfill has been derived assuming an average shear wave velocity of 1,000 fps.

The values of unit coefficient of subgrade reaction are based on values for medium dense sand (Upper Sand Stratum), replaced as compacted structural fill, very-stiff-to-hard clay (Blue Bluff Marl), and dense-to-very-dense sand (Lower Sand Stratum) provided by Terzaghi (1955).

The earth pressure coefficients are Rankine values, assuming level backfill and a zero friction angle between the soil and the wall.

2.5.4.2.5.3 Chemical Properties

Chemical tests were not included in the ESP laboratory testing program. There were no aggressive chemical subsurface conditions identified in the license renewal aging management analysis of Unit 1 and 2 buried concrete **(SNC 2007)**. Chemical property testing of proposed backfill material (Upper Sand Stratum, switchyard borrow and Borrow Area 4) was conducted as part of the COL investigation. Laboratory tests included pH, chloride, and sulfate and were conducted on five split barrel samples from Upper Sand Stratum in the powerblock area; two bulk soil samples taken from test pits excavated in the switchyard borrow area; and three from bulk soil samples from Borrow Area 4. Average pH test results disclosed values of 6.8, 5.2 and 5.4 for the Upper sand, switchyard and Borrow Area 4, respectively, indicating the soil to be mildly corrosive **(API 1991, STS 1990)**. Corresponding average chloride test results disclosed values of 188, 76 and 138 ppm indicating the soil is mildly corrosive **(API 1991, STS 1990)**. Corresponding average sulfate test results disclosed values of 21, 9.8 and 16.3 ppm indicating the soil/concrete interaction will provide a mild exposure for sulfate attack **(ACI 1994)**. Tests were performed by Severn Trent Laboratories, Inc., working as a subcontractor to MACTEC. Test results are included in Appendix 2.5C.

2.5.4.3 Exploration

Section 2.5.4.3.1 summarizes previous subsurface investigation programs performed at the VEGP site, while Section 2.5.4.3.2 describes the ESP subsurface investigation program and Section 2.5.4.3.3 describes the COL subsurface investigation program.

2.5.4.3.1 Previous Subsurface Investigation Programs

Field investigations for VEGP Units 1 and 2 were initiated in January 1971. Field investigations consisted of borings, geophysical methods, and groundwater studies. Additional investigation was completed during excavation for VEGP Units 1 and 2 to verify and obtain further details concerning subsurface conditions in the power block area. A total of 474 borings and 60,000 ft of drilling were completed during these investigations. An additional 111 borings were completed after the initial investigations mentioned above for the following purposes: 41 borings were drilled to define soil conditions and lateral extent of the Blue Bluff Marl in the river facilities, 38 borings were drilled in the power block to collect samples of the Blue Bluff Marl and perform confirmatory testing, and 32 borings were drilled to collect subsurface data for the natural draft cooling tower foundation design. During the previous investigations, electric logging, natural gamma, density, neutron, caliper, and 3-D velocity logs (Birdwell) were performed at selected borings. Water pressure tests and Menard pressuremeter tests were completed to determine properties of the

Blue Bluff Marl bearing stratum. Fossil, mineral, or soluble carbonate tests were performed on recovered samples as warranted.

Geophysical methods were applied to supplement the test borings. The geophysical methods are described in Section 2.5.4.4. For the previous investigations, a total of 28,400 ft of shallow refraction lines, 5,000 ft of deep refraction lines, and cross-hole velocities of subsurface were performed extending from the ground surface to a depth of 290 ft.

Twenty of the previously drilled borings for VEGP Units 1 and 2 fall within, or in the immediate proximity of, the proposed combined power block excavation footprint for the VEGP Units 3 and 4 site. The locations of these borings are provided in Figure 2.5.4-1b. Results of previous investigations are referenced as needed to support the subsurface data obtained during the ESP and COL subsurface investigations.

2.5.4.3.2 ESP Subsurface Investigation Program

The ESP subsurface investigation was performed during September through December 2005 over a substantial portion of the site enveloping the area that would contain the new reactors as well as the switchyard and the cooling towers for the proposed VEGP Units 3 and 4. This investigation consisted of exploration points that were located primarily to confirm the results obtained from the previous extensive investigations. Portions of the original ESP data report were revised as discussed in Section 2.5.4.3.2.4.

The ESP exploration point locations are shown in Figure 2.5.4-1. The exploration points from the ESP investigation are combined with selected boring locations from the previous investigations in Figure 2.5.4-1.

The scope of work and the special methods used by the subsurface investigation contractor (MACTEC) and its subcontractors to collect data are listed below:

- Thirteen exploratory borings were drilled by MACTEC. Two of these borings (B-1002A and C-1005A) were drilled without sampling to allow suspension P-S velocity testing to be performed above zones of drilling fluid loss encountered in the Upper Sand Stratum above the Blue Bluff Marl.
- The efficiency of the automatic hammers employed by the two rotary drill rigs was determined by SPT energy measurements. These services were provided by GRL Engineers, Inc., of Cleveland, Ohio, working as a subcontractor to MACTEC.
- One continuous soil and rock coring borehole was completed at B-1003 by MACTEC.
- Ten CPTs were performed, including three down-hole seismic CPTs. These services were provided by Applied Research Associates (ARA) of South Royalton, Vermont, working as a subcontractor to MACTEC.

- In-situ hydraulic conductivity testing was performed by MACTEC (Section 8 of ASTM D 4044 2002) in 15 groundwater observation wells. Southern Company Services installed these wells and the report is in Appendix 2.4A.
- Geophysical down-hole suspension P-S velocity logging was performed in five completed boreholes (B-1002, B-1002A, B-1003, B-1004, and C-1005A). These services were provided by GEOVision Geophysical Services (GEOVision) of Corona, California, working as a subcontractor to MACTEC. GEOVision also performed caliper, natural gamma, resistivity, and spontaneous potential measurements in boreholes B-1002, B-1003, and B-1004, and a borehole deviation survey at B-1003.
- A topographic survey of all exploration points was performed by MACTEC.
- Laboratory testing of selected borehole samples was performed by MACTEC in its Atlanta, Georgia, laboratories.

The exploration program was performed following the guidelines in Regulatory Guide 1.132, *Site Investigations for Foundations of Nuclear Power Plants*, US Nuclear Regulatory Commission, 2003 (RG 1.132). The fieldwork was performed under an audited and approved quality program and work procedures developed specifically for the ESP application. The subsurface investigation and sample/core collection were directed by the MACTEC site manager, who was on site at all times during the field operations. A Bechtel geotechnical engineer or geologist, along with an SNC representative, was also on site during these operations. MACTEC's QA/QC expert made periodic visits to the site and was on site to audit MACTEC's subcontractors. The draft boring and well logs were prepared in the field by MACTEC geologists.

An on-site storage facility for soil samples and rock cores was established before the fieldwork began. Each sample and core was logged into an inventory system. Samples removed from the facility were noted in the sample inventory logbook. A Chain-of-Custody form was also completed for all samples removed from the facility.

Complete details and results of the exploration program appear in Appendix 2.5A. The borings, CPTs, field permeability testing, and geophysical surveys are summarized below. The laboratory tests are summarized and the results discussed in Section 2.5.4.2. The geophysical tests are summarized and the results discussed in Section 2.5.4.4.

Additionally, a seismic reflection and refraction survey was performed at the site in early 2006 to collect data to help delineate the rock profile associated with the non-capable Pen Branch fault. The results of the seismic reflection and refraction survey are presented in Appendix 2.5B and interpreted results are discussed in Section 2.5.1.2.4.2.

2.5.4.3.2.1 Borings and Samples/Cores

Thirteen borings (excluding B-1003) were drilled to depths ranging from 90 ft (C-1005A) to 304 ft (B-1004). The borings were advanced in the soil using mud-rotary drilling techniques and polymer and/or bentonite drilling fluids. Table 2.5.4-7 provides a summary of the ESP boring and CPT locations and depths, and identifies geophysical testing performed in the boreholes.

The soil was sampled using an SPT sampler at continuous intervals to a 15-ft depth and at 5- or 10-ft intervals below 15 ft. The SPT was performed with automatic hammers and was conducted in accordance with ASTM D 1586 (1999). The recovered soil samples were visually described and classified by the onsite geologist in accordance with ASTM D 2488 (2000). A selected portion of the soil sample was placed in a glass sample jar with a moisture-proof lid. The sample jars were labeled, placed in boxes, and transported to the on-site storage area. Additionally, undisturbed samples of the Blue Bluff Marl (Lisbon Formation) were obtained using rotary pitcher samplers. Disturbed materials were removed from the upper and the lower ends of the tube, and both ends were trimmed square to establish an effective seal. Pocket penetrometer tests were taken on the trimmed lower end of the samples. Both ends of the sample were then sealed with hot microcrystalline wax and protected with plastic caps. Tubes were labeled and transported to the on-site storage area. Table 2.5.4-8 provides a summary of undisturbed samples of the Blue Bluff Marl collected during the ESP subsurface investigation.

The energy transfer efficiency of the automatic SPT hammers used by the drill rigs was obtained using a PAK model pile driving analyzer for both drill rigs. Testing was performed at borings B-1006 and B-1013 from depth ranges of 5 to 20 ft, 30 to 50 ft, and 75 to 100 ft. Resultant energy transfer efficiency measurements ranged from 65 to 87 percent. The average energy transfer efficiency was 75 percent. Table 2.5.4-9 provides the SPT hammer energy transfer efficiency results.

The continuous core boring, B-1003, was performed with a Christensen 94 mm wire line system. A Speedstar Quickdrill 275 drill rig was used. Casing was installed through the soil column to prevent cave-ins and to allow coring of rock at depths below 1,049 ft. Rock coring was performed using a HW-size, double-tube core barrel in accordance with ASTM D 2113 (1999). The recovered soil and rock core samples were placed in wooden core boxes, lined with plastic sheeting. The onsite geologist visually described the core, noting the presence of joints and fractures, and distinguishing natural breaks from mechanical breaks. The geologist also computed the percentage recovery and the RQD. The average core recovery was 77 percent for the entire borehole depth (Appendix 2.5A). Filled core boxes were transported to the on-site sample storage facility, where a photograph of each core was taken.

The boring logs and the photographs of the rock cores appear in Appendix 2.5A. The soil materials encountered in the ESP borings are similar to those found in the previous borings conducted at the VEGP site.

2.5.4.3.2.2 Cone Penetrometer Tests

The CPTs were advanced in accordance with ASTM D 5778 (2000) using a 30 ton self-contained truck rig. Each CPT was advanced to refusal at depths ranging from 6 to 116.7 ft. using a Type 2 piezocone (shouldered). Shallow refusal was encountered at locations C-1001 and C-1009, and offset CPT tests were performed at locations C-1001A and C-1009A. All remaining CPT locations met refusal at or near the top of the Blue Bluff Marl bearing stratum. Down-hole seismic testing was performed at 5 ft intervals in CPTs C-1003, C-1005, and C-1009A (see Section 2.5.4.4) to measure the shear wave velocity in the Upper Sand Stratum. Pore pressure dissipation tests were performed at 68 ft and 79 ft depths in C-1003; 66 ft depth in C-1004; 56 ft, 73 ft, and 82 ft depths in C-1005; and 60 ft, 77 ft, 90 ft, and 99 ft depths in C-1009A.

The CPT logs, shear wave velocity results, and pore pressure versus time plots, for the dissipation tests, are contained in Appendix 2.5A. CPT locations and depths are summarized in Table 2.5.4-7.

2.5.4.3.2.3 In Situ Hydraulic Conductivity Testing

Fifteen observation wells were installed at the ESP project limits during May and June 2005, and a replacement observation well was installed in October 2005. Observation well details are provided in Appendix 2.4A and discussed in Section 2.4.12.

Each well was developed by pumping. The well was considered developed when the pH and conductivity stabilized and the pumped water was reasonably free of suspended sediment. Permeability tests were then performed in each well in accordance with Section 8 of ASTM D 4044 (2002) using a procedure that is commonly termed the slug test method. Slug testing involves establishing a static water level, lowering a solid cylinder (slug) into the well to cause an increase in water level in the well, and monitoring the time rate for the well water to return to the pre-test static level. The slug is then rapidly removed to lower the water level in the well, and the time rate for the water to recover to the pre-test static level is again measured. Electronic transducers and data loggers were used to measure the water levels and times during the test.

Appendix 2.5A contains the well permeability test results and Appendix 2.4A contains the boring logs for the observation wells and the well installation records.

2.5.4.3.2.4 Sample Re-evaluation

The MACTEC ESP data report was revised on November 18, 2007, as provided in Appendix 2.5A. Revisions include changes to the elevation of the top of Utley Limestone, changes to borings logs, and additional laboratory data. Some material descriptions in the Blue Bluff Marl and Utley Limestone were revised to clarify the descriptors of the coarse grained fraction of the sample. The coarse grained fractions, previously described as gravel, upon re-examination of the samples were found to consist of angular, gravel-sized, carbonate particles and were attributed to the mechanical breakage of cemented nodules, shells, cemented limestone, and fossiliferous limestone by the split barrel sampler. The top of Utley Limestone was redefined in some of the ESP boreholes based on the identification criteria developed for the COL investigation program.

2.5.4.3.3 COL Subsurface Investigation Program

The COL subsurface investigation was performed by MACTEC from November 2006 through April 2007 over a large portion of the site, including the power block areas for VEGP Units 3 and 4, cooling towers, switchyard/borrow areas, haul road, intake structure, pumphouse, pipeline, and construction-related areas. The exploration points were located in accordance with the guidelines in RG 1.132. The following paragraphs describe the overall COL investigation program. Other portions of this document primarily address the portion of the COL investigation associated with safety-related structures, principally the combined footprint of the power block excavation.

The COL exploration point locations are shown on Figures 2.5.4-1a and 2.5.4-1b. The scope of work and the methods used by the subsurface investigation contractor (MACTEC) and its subcontractors to collect data are listed below:

- A total of 174 exploratory borings were drilled across the site.
- Seventy-seven exploratory borings were drilled in the power block and cooling tower areas with the 3000 series conducted on the east side, Unit 3, and the 4000 series conducted on the west side, Unit 4. Continuous sampling was conducted in two of these borings, B-3013(C) and B-4013(C), to depths of 155 feet and 165 feet, respectively.
- Sixty-six borings in the 1100 series were drilled in the proposed switchyard, borrow, roadway, batch plant, intake, pumphouse, and other areas across the site.
- Thirty-one borings in the 5000 and 6000 series were drilled in the laydown, roadway, and other areas across the site.
- The efficiency of the automatic hammers employed by the 12 rotary drill rigs was determined by SPT energy measurements.
- Twenty-one CPTs utilizing a Type 2 piezocone were performed, including eight seismic CPTs taken in the power block areas. These services were provided by Gregg In-Situ, Inc., of Columbia, South Carolina, working as a subcontractor to MACTEC.
- Eight test pits were excavated in proposed borrow locations to obtain bulk samples for laboratory testing. The test pit excavations were logged by a MACTEC geologist.
- Geophysical down-hole suspension P-S velocity logging was performed in six completed boreholes, B-3001(DH), B-3002(DH), B-3003(DH), B-4001(DH), B-4002(DH), and B-4003(DH). These services were provided by GEOVision Geophysical Services (GEOVision) of Corona, California, working as a subcontractor to MACTEC. GEOVision also performed caliper, boring deviation, spontaneous potential, natural gamma, and resistivity measurements in these boreholes.
- Electrical resistivity testing was performed by MACTEC along 10 arrays across the site.
- Geophysical refraction microtremor (ReMi) testing was performed by MACTEC at four arrays.

- A horizontal and vertical survey of all exploration points was performed by Toole Surveying Company, Inc., working as a contractor to Southern Company Services.
- Laboratory testing of selected borehole samples was performed by MACTEC laboratories in Atlanta, Georgia and Charlotte North Carolina.
- RCTS testing was performed by FUGRO laboratories in Houston, Texas.

The exploration program was performed following the guidelines in RG 1.132. The fieldwork was performed under an audited and approved quality assurance program, along with approved work procedures developed specifically for the COL application. The subsurface investigation and sample/core collection were directed by the MACTEC site manager, who was on site at all times during the field operations. A Bechtel geotechnical engineer or geologist, along with an SNC representative, was also on site during these operations. MACTEC's QA/QC supervisor made periodic visits to the site, and additional QA/QC personnel visited the site to audit MACTEC's subcontractors. Draft boring logs were prepared in the field by MACTEC geologists and geotechnical engineers. A data report was also prepared by MACTEC as provided in Appendix 2.5C.

An on-site storage facility for soil samples was established before the fieldwork began. Each sample was logged into an inventory system. Samples removed from the facility were noted in the sample inventory logbook. A Chain-of-Custody form was also completed for all samples removed from the facility.

2.5.4.3.3.1 Borings and Samples/Cores

One hundred and seventy-four borings were drilled to depths ranging from 21.5 ft to 420 ft. The borings were advanced in the soil using mud-rotary drilling methods and polymer and/or bentonite drilling fluids. Table 2.5.4-7a provides a summary of the COL boring locations and depths.

The soil was sampled using an SPT sampler at intervals 2.5 ft within the upper 15 ft and thereafter at 5- or 10-ft intervals. The SPT was performed with automatic hammers and was conducted in accordance with ASTM D 1586 (1999). The recovered soil samples were visually described and classified by the onsite geologist or geotechnical engineer in accordance with ASTM D 2488 (2000). A selected portion of the soil sample was placed in a glass sample jar with a moisture-proof lid. The sample jars were labeled, placed in boxes, and transported to the on-site storage area. Additionally, relatively undisturbed samples were obtained. In the Upper Sand stratum, these samples were taken with the direct push method in accordance with ASTM D 1587. In the Blue Bluff Marl and Lower Sand strata these samples were taken using a Pitcher sampler, a double-tube core barrel sampler, due to the very hard/dense nature of the materials. Disturbed materials were removed from the upper and the lower ends of the tube, and both ends were trimmed square to establish an effective seal. Pocket penetrometer tests were taken on the trimmed lower end of the samples. Both ends of the sample were then sealed with hot microcrystalline wax and protected with plastic caps. Tubes were labeled and transported to the on-site storage area.

Twelve drill rigs were used during the COL investigation. The energy transfer efficiency of the automatic SPT hammers was measured for each drill rig in accordance with ASTM D 4633 (2005). Resultant average energy transfer efficiency measurements ranged from 70.1 to 90.2 percent. Table 2.5.4-9a provides a summary of the SPT hammer energy transfer efficiency results.

The boring logs are provided in Appendix 2.5C. The soil materials encountered in the COL borings are similar to those found in the ESP borings and previous borings conducted at the VEGP site.

2.5.4.3.3.2 Cone Penetrometer Tests

Twenty-one CPTs for the COL investigation were advanced in accordance with ASTM D 5778 (2000) using a 20 ton self-contained truck rig mounted on a tracked ATV carrier. Each CPT was advanced to refusal (utilizing a Type 2 piezocone) which generally was encountered at or near the top of the Blue Bluff Marl bearing stratum. Eight of the 21 CPTs included seismic testing as discussed in Section 2.5.4.4.3.2. These eight SCPTs were located in the power block and cooling tower areas of Units 3 and 4. Refusal depths encountered in these soundings ranged from 65.4 to 100.4 ft. The CPT logs, shear wave velocity results, and pore pressure versus time plots for dissipation test are contained in Appendix 2.5C. CPT locations and depths are summarized in Table 2.5.4-7a.

2.5.4.3.3.3 Test Pits

Test pits were excavated at eight locations identified in proposed borrow areas using a track-mounted backhoe (Caterpillar 315L) capable of 12-foot reach. A MACTEC geologist logged the excavation by observing the walls of the excavation and collected representative bulk samples of the various material types. Glass jar samples were also obtained and sealed for moisture retention. The geologist prepared a Geotechnical Test Pit Log based on visual description of the excavated materials according to ASTM D 2488. The backhoe was used to backfill the test excavation using the excavated materials and the completion of logging and sample collection. The Geotechnical Test Pit Logs are included in Appendix 2.5C. Test pit locations and elevations are summarized in Table 2.5.4-7a.

2.5.4.3.3.4 Resistivity

Field electrical resistivity testing was performed along 10 arrays in the proposed switchyards, the cooling towers, and the circulating water line areas of the site. The locations and array lengths were field adjusted to accommodate obstructions. Array locations are shown on Figure 2.5.4-1a and 2.5.4-1b. The Wenner four electrode method was used to perform the tests in accordance with ASTM G57 (2006). Electrode spacing ranged from 3 feet up to 300 feet in order to determine the soil resistivity at increasing depths. The resistivity data interpreted from the tests are contained Appendix 2.5C.

2.5.4.4 Geophysical Surveys

Section 2.5.4.4.1 summarizes previous geophysical investigations performed at the VEGP site: Section 2.5.4.4.2 summarizes the VEGP site geophysical program for this ESP investigation; and Section 2.5.4.4.3 summarizes the geophysical surveys performed for the COL investigation program. Geophysical surveys were also performed for the Phase I test pad program and are summarized in Section 2.5.4.4.4.

2.5.4.4.1 Previous Geophysical Survey Programs

Field investigations that included geophysical methods for VEGP Units 1 and 2 were initiated in January 1971. Geophysical seismic refraction and cross-hole surveys were conducted at the site to evaluate the occurrence and characteristics of subsurface materials. The seismic refraction survey was used to determine depths to seismic discontinuities, based on measured compressive wave velocities. Shallow and deep refraction profiles were obtained throughout the site area, totaling 28,400 and 5,000 linear ft, respectively. The cross-hole seismic survey was conducted in the VEGP Units 1 and 2 power block area to determine in situ velocity data for both compressional and shear waves to a depth of 290 ft (82 ft below sea level) in bore holes 136, 146G, 148, 149, 151, and 154. In this procedure, three-component geophones were lowered into four of the bore holes to equal elevation levels. Energy was generated in a fifth bore hole, at the same elevation level, to determine cross-hole velocities. Boreholes spacing varied from a minimum of about 36 ft to a maximum of about 200 ft.

The seismic (compressional wave) velocities measured in the subsurface soils from depths of 0 to 290 ft ranged from 1,400 fps to 6,800 fps. The shear wave velocities measured in the subsurface soils from depths of 0 to 290 ft ranged from 600 to 1,800 fps. The Upper Sand Stratum, extending from a depth of 0 to 90 ft, has a compressional wave velocity range of 1,400 to 6,650 fps and a shear wave velocity range from 600 to 1,650 fps. The Blue Bluff Marl stratum (and underlying Lower Sand Stratum), extending from a depth of 90 to 290 ft, has a compressional wave velocity of 6,800 fps and shear wave velocities ranging from 1,600 to 1,800 fps. Young's Modulus and Shear Modulus were determined from these results. For the Upper Sand Stratum, Young's Modulus ranged from 0.2×10^5 to 2.0×10^5 pounds per square inch (psi), and Shear Modulus ranged from 0.8×10^4 to 6.8×10^4 psi. For the Blue Bluff Marl (and underlying Lower Sand Stratum), Young's Modulus was 2.3×10^5 psi, and Shear Modulus was 8.0×10^4 psi.

2.5.4.4.2 ESP Geophysical Surveys

Three down-hole seismic CPT tests and five suspension P-S velocity tests were performed during the VEGP site investigation, as described in Section 2.5.4.3.2. In addition a seismic reflection and refraction survey was performed to image the subsurface and characterize the basement lithology and velocities beneath the VEGP site. This survey provided an image of the basement rock across the VEGP ESP site. The results of this survey are presented in Appendix 2.5B and the interpreted results are discussed in Section 2.5.1.2.4.2. The incorporation of these results into the development of the rock shear wave velocity profile is described in Section 2.5.4.7.1.2.

2.5.4.4.2.1 Suspension P-S Velocity Tests in Boreholes

Suspension P-S velocity testing was conducted in ESP borings B-1002, B-1002A, B-1003, B-1004, and C-1005A. Borings B-1002A and C-1005A did not extend below the Upper Sand Stratum. Details of the equipment used to create the seismic compressional and shear waves and to measure the seismic wave velocities are described in detail by Ohya (1986) and are also provided in Appendix 2.5A. Appendix 2.5A also contains a detailed description of the results and the method used to compute the results. Because no ASTM standard is currently available for the suspension P-S velocity testing, a brief description is provided here. The suspension P-S velocity logging system uses a 23-ft (7-m) probe containing a source near the bottom, and two geophone receivers spaced 3.3 ft (1 m) apart, suspended by a cable. The probe is lowered into the borehole to a specified depth, where the source generates a pressure wave in the borehole fluid (drilling mud). The pressure wave is converted to seismic waves (P-wave and S-wave) at the borehole wall. Along the wall, at each receiver location, the P- and S-waves are converted back to pressure waves in the fluid and received by the geophones, which send the data to the recorder on the surface. This procedure is typically repeated at every 1.65 ft (0.5 m) or 3.3 ft (1 m) as the probe is moved up the borehole. The elapsed time between arrivals of the waves at the geophone receivers is used to determine the average velocity of a 3.3-ft (1-m) high column of soil around the borehole. Source to receiver analysis is also performed for quality assurance. The results are summarized below.

The shear wave velocity was defined to the maximum explored depth of 1,338 ft (Appendix 2.5A). For the Upper Sand Stratum, shear wave velocities ranged from 590 to 3,300 fps, with an average value of 1,089 fps. For the Blue Bluff Marl, shear wave velocities ranged from 1,060 to 4,260 fps, with an average value of 2,354 fps. For the Lower Sand Stratum, shear wave velocities ranged from 930 fps to 4,670 fps, with an average value of 2,282 fps. Typical values for the shear wave velocities of each geologic formation contained within the Lower Sand Stratum are as follows: 1,700 fps for the Still Branch, 1,950 fps for the Congaree, 2,050 fps for the Snapp, 2,350 fps for the Black Mingo, 2,650 fps for the Steel Creek, 2,850 fps for the Gaillard/ Black Creek, 2,870 fps for the Pio Nono, and 2,710 fps for the Cape Fear. The shear wave velocity in the portion of the Dunbarton Triassic Basin rock measured ranged from 2,320 to 9,350 fps. There was an upper weathered rock zone about 120 ft thick, where shear wave velocities increased linearly with depth at a very high rate. This high rate of linear increase with depth abated once shear wave velocities achieved values of about 5,300 fps, and shear wave velocities increased linearly with depth at a smaller rate. It is noted that sound rock with an average shear wave velocity of 9,200 fps was not encountered. However, enough data are available to linearly extrapolate to the sound rock horizon from the measurements.

The compressional wave was also defined to the maximum explored depth of 1,338 ft (Appendix 2.5A). For the Upper Sand Stratum, the compressional wave velocity ranged from 1,300 to 7,960 fps, with an average value of 2,572 fps. For the Blue Bluff Marl, compressional wave velocities ranged from 4,640 to 9,830 fps, with an average value of 6,793 fps. For the Lower Sand Stratum, compressional wave velocities ranged from 4,990 to 9,030 fps, with an average value of 6,610 fps. The compressional wave velocity in the Dunbarton Triassic Basin rock ranged from 7,300 to 18,360 fps.

Poisson's ratio was determined from the shear wave and compressional wave velocities (Appendix 2.5A). Poisson's ratio ranged from 0.09 to 0.49 within the Upper Sand Stratum, 0.33 to 0.48 within the Blue Bluff Marl, 0.32 to 0.49 within the Lower Sand Stratum, and 0.10 to 0.46 within the Dunbarton Triassic Basin.

2.5.4.4.2.2 Down-Hole Seismic Tests with Cone Penetrometer

The tests were performed at 5-ft intervals in ESP soundings C-1003, C-1005, and C-1009A. A seismic source, located on the surface, primarily generates shear waves and two geophones mounted horizontally inside near the bottom of the cone string record incoming seismic data. Measurements were only obtained at depths within the Upper Sand Stratum because all CPTs reached refusal at the top of the Blue Bluff Marl.

The shear wave speed and time of peak versus depth plots are included in Appendix 2.5A. The shear wave velocities ranged from 572 to 1,317 fps, with an average value of 930 fps. These values were lower than those measured using the suspension P-S velocity technique and may reflect site variability.

2.5.4.4.2.3 Discussion and Interpretation of Results

Shear and compressional wave velocity measurements made during the ESP subsurface investigation were used as the basis for developing the recommended design values for each stratum that are provided in Section 2.5.4.2. Results from seismic CPTs and suspension velocity logging were used to develop recommended values for the Barnwell Group. Because the seismic CPTs could not penetrate into the Blue Bluff Marl, the recommended values for the Blue Bluff Marl and the Lower Sand Stratum are based on suspension velocity logging results only. No shear or compressional wave velocity measurements were made for the compacted fill during the ESP subsurface investigation. Recommended values for the compacted fill were initially based on data for existing VEGP Units 1 and 2 **(Bechtel 1984)**, as discussed in Section 2.5.4.7.1. Results from the COL investigation and Phase I test pad program were used to confirm the recommended values.

The profile of shear wave velocity versus depth for the subsurface strata is provided in Section 2.5.4.7.

2.5.4.4.3 COL Geophysical Surveys

Eight down-hole seismic CPT tests, six suspension P-S velocity tests, and four refraction microtremor testing (ReMi) arrays were performed during the COL site investigation. The results of these tests, with the exception of the ReMi data, are provided in Appendix 2.5C.

2.5.4.4.3.1 Suspension P-S Velocity Tests in Boreholes

Suspension P-S velocity testing was conducted in COL borings B-3001, B-3002, B-3003, B-4001, B-4002 and B-4003. Details of the equipment used to create the seismic compressional and shear waves and to measure the seismic wave velocities are described in detail by Ohya (1986) and are also provided in Appendix 2.5C. Appendix 2.5C also contains a detailed description of the results and the method used to compute the results. A summary of the results is provided in the following paragraphs.

The shear wave velocity was defined to the maximum explored depth of 420 ft. For the Blue Bluff Marl, shear wave velocities ranged from 1,267 to 2,984 fps, with an average value of 2,050 fps. For the Lower Sand Stratum, shear wave velocities ranged from 745 fps to 2,563 fps with average values for each geologic formation contained within the Lower Sand Stratum as follows: 1,621 fps for the Still Branch, 1,863 fps for the Congaree, and 1,871 fps for the Snapp.

Poisson's ratio was determined from the shear wave and compressional wave velocities and ranged from 0.40 to 0.48 with an average of 0.45.

2.5.4.4.3.2 Down-Hole Seismic Tests with Cone Penetrometer

The tests were performed at 0.6-ft intervals in COL soundings C-3001, C-3002, C-3003, C-3005, C-4001, C-4002, C-4003, and C-4005. A seismic source, located on the surface, primarily generates shear waves and two geophones mounted horizontally inside near the bottom of the cone string record incoming seismic data. Generally, the CPT soundings could not penetrate the dense/hard materials encountered in the Utley and/or Blue Buff Marl; therefore, the shear wave measurements were limited to the Upper Sand stratum. The penetration of the seismic CPT soundings ranged from 65.4 ft to 100.4 ft.

The shear wave speed and time of peak versus depth plots are included in Appendix 2.5C. The shear wave velocities measurements ranged from 435 to 3,802 fps. A summary plot of the COL average shear wave velocity profiles in the Upper Sand Strata is provided in Figure 2.5.4-6a.

2.5.4.4.3.3 Refraction Microtremor Testing

ReMi testing was conducted at four arrays, two in the power block areas of the existing VEGP Units 1 and 2 and two in the footprint of proposed Units 3 and 4. The original intent of collecting these data was to establish the shear wave velocity characteristics of existing backfill at Units 1 and 2. During collection of the data, it was readily apparent that the frequency of the nearby operating plant equipment was interfering with the ReMi data. Unsuccessful attempts were made in the field to overcome this interference. SNC requested Dr. K Stokoe of the University of Texas-Austin to review the ReMi results. He expressed doubt that the test results truly represented the shear wave velocity profile. Therefore these data have not been considered in the COL geophysical survey and are not included in Appendix 2.5C.

2.5.4.4.4 Geophysical Surveys in Compacted Fill

Geophysical surveys were conducted during construction (at three different levels) and upon completion of the 20-ft test pad in the Phase I test pad program in order to evaluate the shear wave profile in compacted fill. The SASW (Spectral Analysis of Surface Waves) method was used to determine shear wave velocity during various stages of construction and upon completion of construction. A more detailed description of this method and the measurements taken is provided in Appendix 2.5D. The cross-hole method (ASTM D 4428) was also used to measure shear wave velocity through the fill. Upon completion of the test pad, 3 cased boreholes were installed through the 20-ft test pad, extending 20 feet below the test pad into native materials. Compressional and shear wave velocity measurements were made between the boreholes. Results from these geophysical tests, along with RCTS test results, were incorporated into the analysis to develop the shear wave velocity profile for the entire depth of backfill (approximately 90 feet) as discussed in Section 2.5.4.7.1.1.

2.5.4.5 Excavation and Backfill

This section covers the following topics:

- The extent (horizontally and vertically) of anticipated safety-related excavations, fills, and slopes.
- Excavation methods and stability.
- Backfill design
- Backfill sources
- Quality control and ITAAC

- Construction dewatering impacts
- Retaining wall

2.5.4.5.1 Extent of Excavations, Fills, and Slopes

Within the VEGP Units 3 and 4 footprint (Figure 2.5.4-1) that will contain all safety-related structures, existing ground elevations are about El. 220 ft msl. The subsurface profiles in Figures 2.5.4-3, 2.5.4-4, and 2.5.4-5 provide an impression of the grade elevation range across the VEGP ESP site. Plant grade for the proposed VEGP Units 3 and 4 will be at El. 220 ft msl. The base of the Nuclear Island foundations for the new units will be about El. 180 ft msl. This level corresponds to a depth of approximately 40 ft below final grade (below El. 220 ft msl), or approximately 50 to 60 ft above the top of the Blue Bluff Marl bearing stratum based on the borings completed during the ESP and COL subsurface investigations. Other foundations in the power block area will be placed at nominal depths near final grade.

Construction of the new units will require a substantial amount of excavation. The excavation will be necessary to completely remove the Upper Sand Stratum. Excavation total depth to the Blue Bluff Marl bearing stratum will range from approximately 80 to 90 ft below existing grade, based on the borings completed during the ESP and COL subsurface investigations. Deeper localized excavations will be required to remove shelly, porous, or weathered material that may be encountered near the top surface of the Blue Bluff Marl.

Seismic Category 1 backfill will be placed from the top of the Blue Bluff Marl to the bottom of the Nuclear Island (NI) foundation at a depth of about 40 ft below final grade. Seismic Category 2 backfill will be placed above the NI foundation level. All backfill placed in the excavation above the NI foundation level will be to the same criteria as Seismic Category 1 backfill. A retaining wall will be constructed along the perimeter of the NI as described in Section 2.5.4.5.7 to facilitate backfilling and construction. Category 2 backfill will be placed behind the retaining wall to final grade or foundation elevation of non NI structures. The backfill material will consist of granular materials, selected from portions of the excavated Upper Sand Stratum and from other acceptable onsite borrow sources. Backfill material properties and source locations are described in more detail in Sections 2.5.4.5.3 and 2.5.4.5.4.

2.5.4.5.2 Excavation Methods and Stability

Excavation in the Upper Sand Stratum will be achieved with conventional excavating equipment. Excavation must adhere to OSHA regulations **(OSHA 2000)**. The excavation will be open-cut, with slopes no steeper than 2-horizontal to 1-vertical. Since the sandy soils can be highly erosive, even temporary slopes cut into the Upper Sand Stratum will be sealed and protected. Where insufficient space for open-cut slopes exists, vertical cuts will be supported with sheet pile or soldier pile and lagging walls. Dewatering will be required once the excavation progresses to depths beneath the groundwater table (approximately El. 150 to 155 ft in the excavation, based on the groundwater monitoring results contained in Section 2.4.12).

Temporary slopes will be graded as the excavation through the Upper Sand Stratum progresses. There are no permanent slopes in the NI area planned for the project that need to be considered for stability.

Possible weathered zones that may be encountered in the upper portion of the Blue Bluff Marl will be removed using conventional excavating equipment. These excavations will be sloped to facilitate placement of compacted structural fill, and the excavation areas will be thoroughly cleaned of loose materials before fill is placed.

2.5.4.5.3 Backfill Design

The design of the Category 1 and Category 2 backfill for VEGP Units 3 and 4 was established through analysis and testing of the borrow material during the COL investigation, the Phase I test pad program, and previous site investigations. Selection and compaction requirements are discussed below.

Material selected for use as Category 1 and Category 2 backfill will be sand and silty sand and will meet the gradation requirements provided in Table 2.5.4-14. Material that falls outside the gradation requirements in Table 2.5.4-14 may be accepted on a case-by-case basis after an engineering evaluation has been performed to assess the overall impact of the material on the backfill design. Borrow material that does not meet the limits on the No. 200 sieve will not be accepted. A backfill specification will be developed to implement these requirements.

All Category 1 and Category 2 backfill will be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D 1557 (2002). Procedures will be developed to control all aspects of backfill placement including lift thickness, moisture conditioning, compaction, and testing. These procedures will result in a high quality, uniform, homogeneous backfill meeting all the requirements for supporting the AP1000 structures at the Vogtle site. The testing frequency for field density tests and ITAAC associated with backfill density and shear wave velocity are discussed in section 2.5.4.5.5.

The Phase I test pad program is complete and is documented in Appendix 2.5D. The objective of this program was to establish site-specific design properties for the backfill, including density, compaction, gradation, and shear wave velocity, and to show that the backfill will satisfy the AP1000 standard plant design. The test pad was constructed below grade, was 20 ft deep, and was 20 ft x 60 ft in plan area. The test pad was constructed in the switchyard borrow area using methods similar to those used to construct the backfill for VEGP Units 1 and 2. The placement and compaction of the backfill were monitored and tested. Results of the test pad program demonstrated that the siting criterion for shear wave velocity of 1,000 fps at the NI foundation depth was achieved with the backfill material within the 20 ft thickness of the test pad.

The backfill properties were determined by evaluating field and laboratory results. Laboratory testing was conducted to measure density, moisture content, grain size, plasticity, shear, and shear modulus reduction and damping relationships. Field testing was conducted to measure density (compaction), shear wave velocity (SASW and cross-hole testing), and standard penetration resistance. Measured SPTs in the test fill (neglecting the upper four feet due to the lack of confining pressure) disclosed a median uncorrected N-value of 32 bpf (corrected for hammer efficiency, N_{60} = 43). A shear wave velocity profile was calculated based on field measurements of velocity in the test pad and in laboratory samples. This profile is discussed in Section 2.5.4.7.1.1 and is presented in Table 2.5.4-10a. An average moist unit weight of 123 pcf was calculated. A drained internal angle of friction of 36° was determined from laboratory triaxial shear tests. Gradation requirements were developed based on the results of field and laboratory testing, including shear wave and compaction testing, from the COL investigation, test pad program, and results from previous investigations. These gradation requirements are presented Table 2.5.4-14. Shear modulus reduction and damping relationships are presented in Section 2.5.4.7.2. A summary of the engineering properties for compacted structural fill is presented in Table 2.5.4-1a.

A Phase II test pad program is scheduled for 2008. The purpose of this program will be to finalize the placement procedures and equipment. The program will use onsite material excavated from the switchyard borrow area. Test pad results will be used to finalize the details of the backfill construction program including construction methods, compaction methods and requirements, and testing protocol. These details will be incorporated into an earthwork specification and backfill placement procedures.

2.5.4.5.4 Backfill Sources

Sufficient sources of backfill have been identified on the Vogtle site through the boring and laboratory testing programs and analysis of their results as described below. Flowable fill may also be used as backfill in small restricted areas where adequate compaction can not be achieved. The flowable fill mix will be designed to have similar strength characteristics as the compacted backfill.

Identified onsite sources of borrow material for the proposed backfill include acceptable materials from the Upper Sand stratum excavated from the power block and a borrow area (switchyard) north of the power block. An alternative borrow area is located about 4,000 feet north of the power block. This alternative location (Borrow Area 4) was also identified and investigated during construction of VEGP Units 1 and 2.

Approximately 3,900,000 cubic yards of material (including an allowance for ramps) will be excavated for the Units 3 and 4 power blocks. Approximately 3,600,000 cubic yards of material will be required to backfill these excavations. Based on a review of the 70 SPT boring logs and laboratory test results on selected samples from the COL subsurface investigation, approximately 50 percent of the material excavated from the power block areas will qualify for reuse as Seismic Category 1 or 2 backfill. However, because a portion of the excavated material may be difficult to segregate, an estimated 30–50 percent of the excavated material is designated for borrow. This quantity accounts for approximately 1,200,000–2,000,000 cubic yards.

Additional backfill for the power blocks, approximately 1,600,000 cubic yards, is available from a borrow source located immediately north of the power blocks (Units 3 and 4 switchyard area). See Figures 2.5.4-15 and 2.5.4-16 for plan and section views, respectively. The switchyard borrow source was explored with 15 SPT borings and five test pits during the COL investigation. The engineering properties of these materials were evaluated with laboratory tests on disturbed, undisturbed, and bulk samples. The COL laboratory testing program (Appendix 2.5.C) included sieve analyses of 27 samples that disclosed an average value of 15 percent fines and a median value of 15 percent. Based on the subsurface data, suitable backfill materials at the switchyard borrow source were identified. These materials were classified according to ASTM D 2488 as silty sands (SM) and poorly graded sands (SP). Clayey sands (SC) were also encountered in some samples. Compaction tests (ASTM D 1557) were conducted on five bulk samples taken from representative soils. Test results disclosed a range of 111 pcf to 125 pcf for the maximum dry density with an average value of 116 pcf.

If additional material is needed, an alternative borrow source is located about 4,000 feet north of the power block area, designated Borrow Area 4. It was explored with four SPT borings and three test pits during the COL investigation. This area was previously explored but not utilized during the design and construction of Units 1 and 2. Sieve analyses were conducted on 31 representative samples and disclosed values ranging from 7 percent to 43 percent fines content with an average value of 16. Compaction tests (ASTM D 1557) were conducted on five bulk samples taken from representative soils. Test results disclosed a range of 113 pcf to 121 pcf for the maximum dry density with an average value of 116 pcf. Based on the subsurface data, suitable backfill materials at Borrow Area 4 are located at the surface (approximate El. 246 ft) to a depth of 36 ft (approximate El. 210 ft) and the borrow area is estimated to contain approximately 1,200,000 cubic yards.

2.5.4.5.5 Quality Control and ITAAC

A quality assurance and quality control program for the backfill will be established to verify that the backfill has been constructed to the design requirements. A soil testing subcontractor, independent from the earthwork contractor, will be used to perform soil testing as part of the quality control program for backfill. The soil testing subcontractor will have an approved quality program.

The backfill quality control program will cover all aspects of the backfill testing program from qualification of the borrow material to confirmatory shear wave velocity testing of the as-placed backfill. Qualification of the borrow material will include soil classification tests, grain size distribution tests, and laboratory compaction (modified Proctor) tests. These tests will determine the acceptability of borrow material and optimum moisture content for compaction. Field density testing will be performed to verify compaction requirements are met as the backfill is placed. For limited earthwork, where fill is compacted with hand equipment, one density test will be conducted for every 2,000 square feet per foot of fill placed. For mass earthwork, the frequency of field density tests for both Category 1 and Category 2 backfill will be a minimum of one test per 500 cubic yards of compacted fill, with at least one test per lift. At least two density tests per lift will be located within the footprint directly beneath the Nuclear Island.

The backfill testing results, non-conformances related to backfill, and QA audits of backfill operations will be reviewed to determine if the as-built backfill meets the minimum 95% modified Proctor compaction requirement under the Seismic Category 1 structures. The results of this evaluation will be documented in a report to support the Inspection Test and Acceptance Criteria (ITAAC) identified in the Backfill ITAAC Table. Field density tests performed on backfill directly beneath the Nuclear Island will be used to demonstrate this ITAAC has been met.

Shear wave velocity testing will be performed on the completed backfill to confirm that the shear wave velocity, at the bottom of the NI foundation and below, is greater than or equal to 1,000 fps. Shear wave velocity will be measured using the SASW method. A report will be developed to document that the required shear wave velocity has been achieved to satisfy the ITAAC identified in the Backfill ITAAC Table.

Preliminary measurements of the shear wave velocity characteristics of the backfill will be made when backfill placement reaches the approximate elevation of the bottom of the NI foundation (El. 180 ft). SASW measurements will be taken within the NI footprint. In addition, representative SASW measurements will be taken at a minimum of three reference locations outside the NI footprint. SASW results for testing at this elevation will be used as supplemental information to document the backfill characteristics. The reference SASW locations will be selected so there will be minimal influence from structures that could impact the SASW testing (e.g., circulating water piping, Lampson Crane pad, MSE wall) when the backfill is at finish grade.

Upon completion of backfill to finish grade, SASW testing will be performed at the three reference locations outside of the NI footprint to determine the backfill shear wave velocity profile down to the top of the Blue Bluff Marl. The results of the SASW measurements made at finish grade will be used to document that the backfill shear wave velocity profile at the elevation of the bottom of the NI foundation and below is greater than or equal to 1,000 fps.

A second method of measuring shear wave velocity (e.g., cross-hole testing, seismic CPT) will be performed at one of the reference locations at finish grade. The results from this test will be compared to the SASW results for the same reference location to validate the SASW results. In the event that velocity measurements do not provide adequate evidence to support closure of the ITAAC, additional evaluations and testing may be performed.

The final report submitted to close the ITAAC will present the shear wave velocity profile for the completed backfill at the referenced locations and any supporting analysis or testing used to conclude that the ITAAC has been met. For each velocity profile, the shear wave velocity profile at the elevation of the bottom of the NI foundation and below will be compared to the required 1,000 fps.

Backfill ITAAC Table

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

2.5.4.5.6 Control of Groundwater During Excavation

Construction dewatering is discussed in Section 2.5.4.6.2. Since the Upper Sand Stratum soils can be highly erosive, sumps and ditches constructed for dewatering will be lined. The tops of excavations will be sloped back to prevent runoff down the excavated slopes during heavy rainfall.

2.5.4.5.7 Retaining Wall

A retaining wall will be constructed within each power block excavation to facilitate construction of the nuclear islands. This retaining wall, planned as a mechanically stabilized earth (MSE) wall, will be constructed around the perimeter of each NI and will permit backfilling of the excavations before construction of the NI foundations and substructure walls. The MSE wall will act as the exterior form for the foundation and substructure walls. Waterproofing will be placed on the surface of the precast concrete MSE wall facing panels before placing NI foundation and substructure wall concrete. (Figure 2.5.4-17)

2.5.4.6 Groundwater Conditions

2.5.4.6.1 Groundwater Measurements and Elevations

Groundwater conditions at the site are discussed in detail in Section 2.4.12, and only a summary is presented here. Groundwater is present in unconfined conditions in the Upper Sand Stratum and in confined conditions in the Lower Sand Stratum at the VEGP site. The Blue Bluff Marl is considered to be an aquiclude that separates the unconfined water table aquifer in the Upper Sand Stratum from the confined Tertiary aquifer in the Lower Sand Stratum. In the powerblock area, the groundwater generally occurs at a depth of about 65 to 70 ft below the existing ground surface.

Fifteen observation wells were installed at the site during June and July 2005, before the start of the ESP subsurface investigation program. Ten of these wells were installed in the unconfined aquifer, and five were installed in the confined Tertiary aquifer. Additionally, 22 existing wells were used as part of the groundwater monitoring program for the ESP study. Thirteen of these wells were installed in the unconfined water table aquifer, and nine were installed in the confined aquifer. The wells installed in the unconfined water table aquifer exhibit groundwater levels ranging from about El. 132 to El. 165.5 ft, while the wells installed in the confined aquifer exhibit groundwater levels ranging from about El. 82 to El. 128 ft. The logs and details of well installation and testing are contained in Appendix 2.4A and Appendix 2.5A. Hydraulic conductivity (slug) tests were performed in the wells installed during the ESP field investigation, as described in Section 2.5.4.3.2.3. Hydraulic conductivity (k) values for the unconfined water table aquifer in the Upper Sand Stratum, based on the slug test results, range from 4.4×10^{-5} to 9.3×10^{-4} cm/second, with a geometric mean of 1.75×10^{-4} cm/second. The hydraulic conductivity of the confined Teritary aquifer in the Lower Sand Stratum, based on the slug test results, ranges from 1.3×10^{-4} to 7.5×10^{-4} cm/second, with a geometric mean of 2.95×10^{-4} cm/second. Detailed descriptions of current groundwater conditions, as well as post-construction groundwater conditions are provided in Section 2.4.12.

Groundwater levels at the site will require temporary dewatering of excavations extending below the water table during construction of new Units 3 and 4. Dewatering will be performed in a manner that will minimize drawdown effects on the surrounding environment and VEGP Units 1 and 2. Drawdown effects are expected to be limited to the VEGP site and to be negligible for VEGP Units 1 and 2. The relatively low permeability of the Upper Sand Stratum and underlying Blue Bluff Marl means that sumps and pumps should be sufficient for successful construction dewatering, as discussed in Section 2.5.4.6.2.

The design groundwater level for VEGP Units 3 and 4 will be taken at El. 165 ft msl based on the results of groundwater monitoring performed during a period of 10 years prior to the ESP subsurface investigation, and during the ESP subsurface investigation, as discussed in Section 2.4.12. This level corresponds to the design groundwater level for the existing VEGP Units 1 and

2. The static stability of the proposed structures based on this design groundwater level is discussed in Section 2.5.4.10.

2.5.4.6.2 Construction Dewatering

Dewatering for all major excavations could be achieved by gravity-type systems. Due to the relatively impermeable nature of the Upper Sand Stratum, sump-pumping of ditches will be adequate to dewater the soil. These ditches will be advanced below the progressing excavation grade.

During construction of VEGP Units 1 and 2, the excavation materials were dewatered by a series of ditches oriented in an east-west direction. They were connected by a north-south ditch, which drained to a sump in the southwest corner of the excavation. The sump was equipped with four pumps each with a capacity of 500 gal./min to remove inflows from groundwater. Additional capacity was provided for the removal of inflows of storm water in the excavation.

Similar dewatering procedures will be implemented during the excavation for VEGP Units 3 and 4.

2.5.4.7 Response of Soil and Rock to Dynamic Loading

All new safety-related structures will be founded on the planned structural backfill, which will completely replace the existing Upper Sand Stratum soils. The seismic acceleration at the sound bedrock level will be amplified or attenuated up through the soil and rock column. To estimate this amplification or attenuation, the following data are required.

- Shear wave velocity profile of the soils and rock
- Variation with strain of the shear modulus and damping values of the soils
- Site-specific seismic acceleration-time history

In addition, an appropriate computer program is required to perform the analysis.

2.5.4.7.1 Shear Wave Velocity Profile

2.5.4.7.1.1 Soil Shear Wave Velocity Profile

Various measurements have been made at the VEGP ESP site to obtain estimates of the shear wave velocity in the soil. Measurements were also made at the site during the COL investigation to confirm ESP estimates of shear wave velocity in the soil.

All safety-related structures will be founded on the structural backfill that will be placed on top of the Blue Bluff Marl after complete removal of the Upper Sand Stratum. Shear wave velocity was not determined for the compacted backfill during the ESP subsurface investigation. Data for existing Units 1 and 2 is used **(Bechtel 1984)**, and the backfill shear wave velocity values are summarized in Table 2.5.4-10.

During the COL investigation, shear wave velocity data for the compacted backfill was measured directly in the field during the Phase I test pad program. These data, with laboratory test data, were used to evaluate the shear wave velocity of the backfill. A summary of the Phase I test pad program, including a discussion of material properties, is included in Section 2.5.4.5.3. The results of the test pad program are presented in Appendix 2.5D. RCTS and other data from the COL investigation were also used to evaluate the shear wave velocity of the backfill. The RCTS data are presented in Attachment G of Appendix 2.5C. Results of the COL investigation and Phase I test pad were used to develop the shear wave velocity profile of the backfill based on COL data. This profile is presented in Table 2.5.4-10a and is in good agreement with the ESP backfill profile. Both of these profiles are included in the respective soil columns in Figure 2.5.4-7a.

Figure 2.5.4-6 shows the shear wave velocity values measured in the subsurface soil and rock strata for the ESP subsurface exploration program using suspension P-S velocity and CPT down-hole seismic testing. Figure 2.5.4-6a shows the shear wave velocity values measured in the Upper Sand Stratum using CPT down-hole seismic testing from COL data. The shear wave velocity profile shown in Figure 2.5.4-7 is the profile interpreted from the results of the ESP data shown in Figure 2.5.4-6 for strata below the Upper Sand Stratum, plus the shear wave velocity values for the backfill shown on Table 2.5.4-10. The shear wave velocity values corresponding to the profile shown on Figure 2.5.4-7 for the different soil strata encountered by the borings are provided in Table 2.5.4-11.

The shear wave velocity profile developed from the ESP investigation and shown in Figure 2.5.4-7 is used in the seismic amplification/attenuation analysis. The soil profile used consists of: Compacted backfill from 0 to 86 ft, Blue Bluff Marl from 86 to 149 ft, Lower Sand Stratum from 149 to 1,049 ft, Dunbarton Triassic Basin and Paleozoic Crystalline Rock below 1,049 ft.

During the COL investigation, shear wave velocity values were measured in the Blue Bluff Marl and the upper portions of the Lower Sand Stratum as previously described in Section 2.5.4.4.3. These data included measurements in 6 boreholes, extending to a maximum depth of 420 feet below ground surface. Shear wave velocity values were measured in the Still Branch, Congaree, and Snapp Formations of the Lower Sand Stratum. These COL data (6 profiles) were combined with two ESP profiles (located in the powerblock area of Units 3 and 4) and averaged. The average shear wave profile for the this COL data set is shown on Figure 2.5.4-7a. This profile also reflects the average stratigraphy within the powerblock excavation footprints based on data from the COL borings. The shear wave velocity profile includes the shear wave velocity profile of the backfill that was developed during the Phase I test pad program. The profile below the COL data (below the upper portion of the Snapp formation) incorporates the shear wave velocity data from the ESP profile. The COL profile consists of: compacted backfill from 0 to 88 ft, Blue Bluff Marl from 88 to 156 ft, Lower Sand Stratum from 156 to 1,058 ft, and Dunbarton Triassic Basin and Paleozoic Crystallilne Rock below 1,058 ft. The ESP profile, shown on Figure 2.5.4-7, is also illustrated on Figure 2.5.4-7a for comparison purposes. Figure 2.5.4-7a illustrates the relationship, including the similarity, between the two data sets. In general, within specific geologic formations, the two profiles demonstrate consistent shear wave velocity characteristics. The profile of the combined data set (COL) in the middle and upper portions of the Blue Bluff Marl is in good agreement with the ESP profile. At the lower portions of the Blue Bluff and in the Lower Sand Stratum, the COL profile exhibits slightly lower shear wave values than the ESP profile.

2.5.4.7.1.2 Rock Shear Wave Velocity Profile

As discussed in Section 2.5.4.2.2, the VEGP ESP site sits on over 1,000 feet of Coastal Plain sediments underlain by Triassic Basin sedimentary rock, which in turn is underlain by Paleozoic crystalline rock (see Figure 2.5.1-40). For the purpose of subsequent site response analysis, for which input rock time histories must be inserted at a depth where the material shear-wave velocity is approximately 9,200 ft/s, it is necessary to know the shear-wave velocity profile and materials properties for the site down to the depth at which this velocity is encountered. Because the site overlies both Triassic Basin and Paleozoic crystalline rocks, it is necessary to consider effect of shear-wave velocities and material properties of both rock types and their geometries.

As indicated in Figure 2.5.4-6, the shear-wave velocities measured at the top of the Triassic Basin, even through the weathered portion, do not reach the velocity of 9,200 ft/s. Inspection of available deep borehole shear-wave velocity at SRS **(SRS 2005)** along with the B-1003 data [Figure 2.5.4-8], however, suggests the following character of rock shear-wave in the Triassic Basin:

- A weathered zone of ~200 feet thickness occurs at the top of the Triassic Basin, characterized by a steep shear-wave velocity gradient, where the shear-wave velocity rapidly increases with depth to a point where a relatively high shear-wave velocity, but less than 9,200 ft/s is reached;
- Below the weathered zone the shear-wave velocity increases with a gentler gradient within the unweathered rock;
- Considering the SRS data as a guide for shear-wave velocity within deep portions of the Triassic Basin, there are a range of gentle gradients and a range of shear-wave velocities for the top of the unweathered Triassic Basin that could be considered as a continuation of the site-specific profile presented by B-1003.

Figure 2.5.1-41 indicates that the non-capable Pen Branch fault separates the Triassic Basin from the Paleozoic crystalline rocks. The structural geometry of these rock units and the fault, relative to the locations of boreholes B-1002 and B-1003 (approximate locations of the proposed nuclear units) and considering the velocity profiles shown in Figure 2.5.4-8, a shear-wave velocity profile through the Triassic Basin would not likely reach 9,200 ft/s before encountering the Paleozoic crystalline rock. Several observations and studies at SRS [e.g., (**Geovision 1999**,

Lee et al 1997, **Domaracki 1994**)] indicate that the shear-wave velocity of the Paleozoic crystalline rock is at least 9,200 ft/s.

Therefore, to represent the variability of the depth at which the Paleozoic crystalline rock is encountered, with a shear-wave velocity of at least 9,200 ft/s, and the uncertainty of the shear-wave velocity gradient and velocity at the top of the unweathered Triassic Basin, six rock shear-wave velocity profiles were considered to comprise the base case used in the seismic amplification/attenuation analysis. Figure 2.5.4-7 shows a plot of these six rock shear-wave velocity profiles and Table 2.5.4-11, Part B presents their tabulation.

Figures 2.5.1-40 and Figure 2.5.4-8 suggest additional geometries for the shear-wave velocity profiles of the Triassic Basin and the Paleozoic crystalline rock that could impact site response. As interpreted in Figure 2.5.1-41, further to the northwest of the footprint of the project site the coastal Plain sediments would be underlain immediately by the Paleozoic crystalline rock. Conversely, further to the southeast of the footprint of the project, the Paleozoic crystalline rock is at such a depth that the shear-wave velocity gradient in the Triassic Basin would result in 9,200 ft/s being reached in the shear-wave velocity profile while still within the Triassic Basin. Close inspection of the DRB-9 shear-wave velocity profile in Figure 2.5.4-8 suggests a low-velocity zone at the bottom of the Triassic Basin at the encountering of the Pen Branch fault. Sensitivity analyses were performed that indicated that alternate shear-wave velocity models suggested by these observations result in insignificant variations in the site response, relative to the six profiles that were explicitly considered, as discussed above.

2.5.4.7.2 Variation of Shear Modulus and Damping with Shear Strain

2.5.4.7.2.1 Shear Modulus

2.5.4.7.2.1.1 ESP Analysis

The variation of soil shear modulus values of sands, gravels, and clays with shear strain is well-documented by researchers such as Seed and Idriss (1970); Seed et al. (1984); and Sun et al. (1988). This research, along with additional work, has been summarized by EPRI **(EPRI TR-102293 1993)**.

Shear modulus is derived from the respective unit weight and shear wave velocity of the soil strata with the following equation:

$$G_{max} = \rho \cdot (V_s)^2 = \gamma \cdot (V_s)^2/g \qquad \text{Equation (20-27) on page 758 of Bowles (1982)}$$

Shear wave velocity data are shown on Table 2.5.4-11. Unit weight data are shown on Table 2.5.4-1. Values for shear modulus are tabulated during analysis with the SHAKE 2000 program **(Bechtel 2000)**, and the low strain values are also shown on Tables 2.5.4-2 for the existing soils and rock, and on Table 2.5.4-10 for the compacted backfill.

From EPRI **(EPRI TR-102293 1993)**, the dynamic shear modulus reduction is derived in terms of depth for granular soils (Upper and Lower Sand Strata) and in terms of Plasticity Index (PI) for cohesive soils (Blue Bluff Marl).

The EPRI curves for sands **(EPRI TR-102293 1993, Figure 7.A-18)** were used to derive the shear modulus reduction factors for the granular soil strata (compacted backfill and Lower Sand Stratum). The EPRI curves for clays **(EPRI TR-102293 1993, Figure 7.A-16)** were used to derive the shear modulus reduction factors for the Lisbon Formation using PI = 25 percent. The shear modulus reduction factors are provided in Table 2.5.4-12 and Figure 2.5.4-9.

The shear modulus reduction factors developed for the neighboring SRS and contained in Lee (1996) were also used in the analysis. The SRS curves were selected based on their stratigraphic relationship to the Voglte 3 and 4 site. The SRS curve labeled as Blue Bluff Marl in Table 2.5.4-13 and on Figure 2.5.4-10 is based on the Dry Branch Formation and the Santee Formation, the SRS stratigraphic equivalent to the Vogtle Blue Bluff Marl. Degradation curves for the compacted backfill were not developed for SRS. The mean site reduction site amplification factors using EPRI and SRS shear modulus degradation relationships were weighted equally as described in Section 2.5.2.5.1.2.1.

2.5.4.7.2.1.2 COL Analysis

Site-specific dynamic shear modulus reduction curves were developed from RCTS test results on samples from the Blue Bluff Marl and Lower Sand strata as well as proposed borrow materials for the compacted backfill, taken during the COL investigation. Index testing was also conducted on these samples. Results of index and RCTS testing are included in Attachment G of Appendix 2.5C.

In the Blue Bluff Marl, four relatively undisturbed samples (Pitcher samples) were tested. Two samples disclosed low plasticity indices (PI =26 and 27) while two disclosed high PI values (46 and 69). The shear modulus reduction data was plotted against shearing strain and overlain on the EPRI curves for clay **(EPRI TR-102293 1993, Figure 7.A-16)**. The site specific data followed trends consistent with the EPRI relationships for PI. Site specific curves were derived for low PI material and high PI material based on the similarity of the EPRI PI curves.

In the Lower Sand Stratum, five relatively undisturbed samples (Pitcher samples) were tested. Three were identified as sand and two were identified as low plasticity clays. The shear modulus reduction data were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-18)**. Note that RCTS data for the clayey samples were evaluated against the EPRI curves for clay; however, the damping relationships disclosed in these tests (as discussed later) were not consistent with the EPRI clay relationships. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific curves were derived for the sand and the clay materials in the Lower Sand stratum based on the similarity of the EPRI depth curves.

Five bulk samples from test pits in proposed borrow sources were identified for testing. Moisture-density (ASTM D 1557) and index testing were conducted on these samples. The fines content of these samples ranged from about 8 to 25 percent. RCTS tests were conducted on each bulk sample (using the same loading schedule) at two different levels of compaction (95% and 97% or 95% and 100%). The assigned confining pressures for the RCTS testing were determined based on representative depths throughout the proposed 90-ft column of backfill. Test results disclosed little variation based on the level of compaction. The shear modulus reduction data was plotted against shearing strain and overlaid on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-18)**. Test results for samples at low confining pressures disclosed similar trends, as did test results for samples at higher confining pressures. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific damping curves for borrow material were developed for samples under low confining pressure (depths less than 25 ft) and for samples under higher confining pressures (greater than 25 ft) based on the similarity of the EPRI curves for depth for granular soils.

Site specific shear modulus reduction curves developed from the RCTS testing of COL samples are provided in Table 2.5.4-12a and Figure 2.5.4-9a. These data were used to evaluate the site response as described in Section 2.5.2.9.3

2.5.4.7.2.2 Damping

2.5.4.7.2.2.1 ESP Analysis

The publications cited above address the variation of soil damping with cyclic shear strain as well as the variation of shear modulus with shear strain.

From EPRI **(EPRI TR-102293 1993)**, the damping ratio is derived in terms of depth for granular soils (Upper and Lower Sand Strata) and in terms of PI for cohesive soils (Blue Bluff Marl).

The EPRI curves for sands **(EPRI TR-102293 1993, Figure 7.A-19)** were used to derive the damping ratios for the granular soil strata (compacted backfill and Lower Sand Stratum). The EPRI curves for clays **(EPRI TR-102293 1993, Figure 7.A-17)** were used to derive the damping ratios for the Lisbon Formation using PI = 25 percent. The damping ratios are provided in Table 2.5.4-12 and Figure 2.5.4-11.

The damping ratio values developed for the neighboring SRS and contained in Lee (1996) were also used in the analysis. The SRS curves were selected based on their stratigraphic relationship to the Voglte 3 and 4 site. The SRS curve labeled as Blue Bluff Marl in Table 2.5.4-13 and on Figure 2.5.4-12 is based on the Dry Branch Formation and the Santee Formation, the SRS stratigraphic equivalent to the Vogtle Blue Bluff Marl. Degradation curves for the compacted backfill were not developed for SRS. The mean site reduction site amplification factors using EPRI and SRS shear modulus degradation relationships were weighted equally as described in Section 2.5.2.5.1.2.1.

2.5.4.7.2.2.2 COL Analysis

Site-specific damping curves were developed from RCTS test results on samples from the Blue Bluff Marl and Lower Sand strata as well as proposed borrow materials for the compacted backfill, as similarly described in Section 2.5.4.7.2.1.2.

The RCTS damping relationships for the Blue Bluff Marl samples were plotted and overlain on the EPRI curves for clay **(EPRI TR-102293 1993, Figure 7.A-17)**. The site specific data followed trends consistent with the EPRI damping relationships for PI. Site specific curves were derived for low PI material and high PI material based on the similarity of the EPRI PI curves.

The RCTS damping relationships for the Lower Sand Stratum samples were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils **(EPRI TR-102293 1993, Figure 7.A-19)**. The damping relationships for the clayey samples were evaluated against EPRI curves for clay; however, these data disclosed lower damping values at lower shear strains. Instead the RCTS data were more closely aligned with the EPRI relationships with depth for granular soils. The site specific data for both sand and clay samples followed trends consistent with the EPRI relationships for depth for granular soils. Therefore, site specific damping curves were derived for the sand and the clay materials in the Lower Sand stratum based on the similarity of the EPRI curves for depth for granular soils.

The RCTS damping relationships for the proposed borrow sources were plotted against shearing strain and overlain on the EPRI curves for depth for granular soils (EPRI TR-102293 1993, Figure 7.A-19). Test results for samples at low confining pressures disclosed similar trends, as did test results for samples at higher confining pressures. The site specific data followed trends consistent with the EPRI relationships for depth for granular soils. Site specific damping curves for borrow material were developed for samples under low confining pressure (depths less than 25 ft) and for samples under higher confining pressures (greater than 25 ft) based on the similarity of the EPRI curves for depth for granular soils.

Site specific damping curves developed from the RCTS testing of COL samples are provided in Table 2.5.4-12a and Figure 2.5.4-11a. These data were used to evaluate the site response as described in Section 2.5.2.9.3.

After randomization, the damping curves were cut off at 15 percent damping ratio per NUREG-0800, Section 3.7.2 (1996).

2.5.4.7.3 Soil/Rock Column Amplification/Attenuation Analysis

The SHAKE2000 **(Bechtel 2000)** computer program was used to compute the site dynamic responses for the soil/rock profiles described in Section 2.5.4.7.1. The computation was performed in the frequency domain using the complex response method. Section 2.5.2.5 describes in detail the soil/rock column amplification/attenuation analysis based on the ESP soil column.

SHAKE2000 uses an equivalent linear procedure to account for the non-linearity of the soil by employing an iterative procedure to obtain values for shear modulus and damping that are compatible with the equivalent uniform strain induced in each sublayer. At the outset of the analysis, a set of properties (based on the values of shear modulus and damping presented in Section 2.5.4.7.1, and total unit weight) was assigned to each sublayer of the soil profile. The analysis was conducted using these properties, and the shear strain induced in each sublayer was calculated. The shear modulus and damping ratio for each sublayer was then modified based on the shear modulus and damping ratio versus strain relationships presented in Section 2.5.4.7.2. The analysis was repeated until strain-compatible modulus and damping values were achieved.

2.5.4.7.4 Two-Dimensional Effects Site Response Analysis (Bathtub Model)

The model for the site dynamic response analysis as described in Section 2.5.2.5 depicted the backfill above the Blue Bluff Marl as a continuum. The model did not account for the extent of the excavation and backfill and any impacts the Upper Sands have on the site response. These impacts were evaluated by considering the site response with the Upper Sands in place and with these materials replaced with backfill. The average shear wave profile of the Upper Sands as developed from the COL data, as shown on Figure 2.5.4.6a, was used to characterize shear wave velocity of the Upper Sands. A discussion of this analysis and results are presented in Section 2.5.2.9.2.

2.5.4.7.5 Comparison of ESP vs. COL Soil Column

Subsurface data were collected and evaluated at the site during two distinct phases referred to as the ESP investigation and COL investigation (including the Phase 1 test pad program) as presented in Section 2.5.4.3. The ESP investigation was limited in scope and broad in areal coverage; whereas the COL investigation was more focused in coverage (to the power block area) and extensive in scope. Subsurface data, including shear wave velocity, from the ESP investigation were taken from widely spaced borings. One of these boreholes (B-1003) extended through the entire soil column (over 1,000 ft) and into the underlying sedimentary rock of the Triassic Basin. Subsurface data from VEGP Units 1 and 2 and other regional sources were also evaluated. Soil non-linearity curves obtained from EPRI and the nearby Savannah River Site (SRS) were assigned based on soil type and depth. The resulting ESP soil column was used in the amplification/attenuation analysis in Section 2.5.2.5.

The COL investigation provided numerous additional subsurface data specific to the powerblock areas of Units 3 and 4. The COL investigation was taken to exploration depths of 420 ft. ESP data taken within the powerblock areas were compiled with the COL data to develop the COL soil column. These data included averaged shear wave velocities, averaged strata thicknesses and densities. A thick clay layer (approximately 70 ft) encountered in the Lower Sands, as discussed in Section 2.5.4.2.2.3, was incorporated into the COL soil column as shown on Figure 2.5.4-7a. Site specific soil non-linearity curves for the various strata, including the clay soils in the Lower Sands, were developed from RCTS testing of representative COL samples and are included in the COL soil column. These data were discussed in Section 2.5.4.7.2 and are presented in Figures 2.5.4-9a (G/Gmax curves) and 2.5.4-11a (damping ratio curves). Site specific dynamic properties of the compacted backfill were developed during the COL laboratory testing program and the Phase 1 test pad program and are included in the COL soil column.

The stratification and shear wave velocity profiles for the ESP and COL soil columns are presented in Figure 2.5.4-7a. The offset in soil stratification between the soil columns reflects refinements due to the additional data collected during the COL investigation. The stratification of the ESP soil column is based on the deep boring, B-1003. The stratification of the COL soil column is based on numerous additional borings in the power block areas. The data disclosed thicker near surface strata as compared with boring B-1003. No additional stratification or shear wave velocity data below the top of the Snapp Formation in the Lower Sands were collected during the COL investigation; therefore, the COL soil column stratification and shear wave velocity profiles between the Snapp Formation and the top of the Triassic Basin bedrock were carried over from the ESP soil column with the same strata thicknesses but slightly shifted in depth to match the thicker near surface strata. Comparison of the two shear wave velocity profiles indicates good agreement between the data sets. Trends within the strata are consistent.

Comparisons of the soil non-linearity curves used for ESP and COL are presented in the attached Figures 2.5.4-19a through 2.5.4-20c. Figures 2.5.4-19a, 19b, and 19c illustrate the normalized shear modulus vs. shear strain curves for compacted backfill, Blue Bluff Marl, and Lower Sands, respectively. Figures 2.5.4-20a, 20b, and 20c illustrate the soil damping vs. shear strain curves for the same strata. The figures include both the site specific curves developed during the COL investigation and the EPRI and SRS model curves assigned during the ESP investigation. The COL site specific data for the Lower Sands includes non-linearity curves for both sand and clay materials in this stratum. Generally the figures suggest that the subsurface soils behave more linearly (provide a smaller reduction in shear modulus and less damping) than the models used for the ESP investigation.

The COL soil column, including shear wave velocity and site specific non-linearity relationships as described here, was used in the site response sensitivity analysis to evaluate the effects of the COL data with the ESP data as described in Section 2.5.2.9.3.

2.5.4.7.6 MSE Backfill Shear Wave Velocity Profile

As discussed in Section 2.5.4.5.7, an MSE wall is planned to facilitate construction of the Nuclear Island. This wall, as shown on Figure 2.5.4-17, will be founded at the NI foundation level and will consist of wall facing panels and tensile elements embedded in the backfill behind the wall face.

During construction, the backfill immediately behind the wall face, for a distance of about 5 feet, will likely be compacted with smaller, potentially hand-operated, compactors and in thinner lifts to achieve the compaction criteria of at least 95 percent of the maximum dry density modified Proctor value (ASTM D 1557). Beyond this wall face zone, the backfill will be compacted as part of the mass earthwork operation utilizing larger self-propelled compaction equipment. Owing to the likely different compaction procedures and the presence of the MSE wall face, the shear wave velocity profile of the backfill in the 5 ft wall face zone may be reduced. To investigate the effect of this possibility, a reduced velocity profile for the full height of the wall (MSE best estimate) was used in a soil structure interaction analysis, as presented in Appendix 2.5.E. The results show that there are no differences in the seismic structural responses from the potentially reduced shear wave velocity profile behind the MSE wall.

2.5.4.8 Liquefaction Potential

Soil liquefaction is a process by which loose, saturated, granular deposits lose a significant portion of their shear strength due to pore pressure buildup resulting from cyclic loading, such as that caused by an earthquake. Soil liquefaction occurrence (or lack thereof) depends on geologic age, state of soil saturation, density, gradation, plasticity, and earthquake intensity and duration. Soil liquefaction can occur, leading to foundation bearing failures and excessive settlements, when all of the following 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 naturally occurring Upper Sand Stratum soils at the VEGP site meet these three criteria. These soils consist of sands with varying fines content. An approximate 30-ft depth of the Upper Sand Stratum occurs beneath the groundwater table at a depth of 60 ft beneath the ground surface. The average corrected SPT N-value within the Upper Sand Stratum was 25 bpf, indicating a medium dense condition. The underlying Blue Bluff Marl soils are significantly cohesive; although some seams of coarse-grained materials are present. The Lower Sand Stratum is sufficiently dense and deep. Liquefaction is not a concern within these strata; although the liquefaction potential of the coarse-grained materials in the Blue Bluff Marl will be discussed. The liquefaction potential of the Upper Sand Stratum will also be discussed.

During construction of VEGP Units 1 and 2, the entire portion of the Upper Sand Stratum was removed and replaced with engineered fills due to susceptibility to liquefaction. A similar excavation will be executed for VEGP Units 3 and 4.

Regulatory Guide 1.198, *Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites*, US Nuclear Regulatory Commission, November 2003 (RG 1.198) is used as a guide for liquefaction analysis presented herein.

2.5.4.8.1 Acceptable Factor of Safety Against Liquefaction

RG 1.198 states that factors of safety (FS) ≤ 1.1 against liquefaction are considered low, FS ≈ 1.1 to 1.4 are considered moderate, and FS ≥ 1.4 are considered high. The Committee of Earthquake Engineering of the National Research Council **(NRC/NAP 1985)** states:

> *There is no general agreement on the appropriate margin (factor) of safety, primarily because the degree of conservatism thought desirable at this point depends upon the extent of the conservatism already introduced in assigning the design earthquake. If the design earthquake ground motion is regarded as reasonable, a safety factor of 1.33 to 1.35...is suggested as adequate. However, when the design ground motion is excessively conservative, engineers are content with a safety factor only slightly in excess of unity.*

2.5.4.8.2 Previous Liquefaction Analyses

The liquefaction potential of the Upper Sand Stratum was previously evaluated using the standard penetration test blow counts obtained during the investigations for VEGP Units 1 and 2 and the simplified procedure of Seed and Idriss. This evaluation indicated that the Upper Sand Stratum below the groundwater table was susceptible to liquefaction when subjected to the maximum SSE acceleration of 0.2g developed for VEGP Units 1 and 2. Based on this evaluation, the Upper Sand Stratum was removed to an approximate elevation of 130 to 135 ft in the VEGP Units 1 and 2 power block area. Select sand and silty sand compacted to 97 percent of the maximum density determined by ASTM D 1557 was placed from the top of the Blue Bluff Marl stratum to the design elevation of the various power block structures with the exception of an area north of the turbine building. The liquefaction potential of compacted backfill in the power block area was evaluated, and the analysis indicated a factor of safety against liquefaction on the order of 1.9 to 2.0. The analysis was done utilizing cyclic strength data (PSAR data) obtained from tests on specimens of compacted backfill.

During the investigations for borrow sources for VEGP Units 1 and 2, additional dynamic data (borrow source data) were obtained to supplement the cyclic strength data for the compacted fill. Cyclic triaxial tests were performed on compacted specimens of sands obtained from stockpiles and borrow areas. The cyclic stress ratios versus the number of cycles to 2.5 percent total strain (initial liquefaction) showed that the stress ratios for the cleaner sands were substantially lower than for silty sands. In the liquefaction analysis performed using the PSAR data, stress ratios for the cleaner sands were used to obtain the safety factor against liquefaction. Therefore, the cyclic stress ratios for the cleaner sands obtained during investigations for borrow material were compared with values obtained during the PSAR investigations. A comparison of the two test data (PSAR data versus borrow source data) indicates that the PSAR data represent a lower bound of test values. If the liquefaction analysis were performed using the upper bound values (borrow source data), a factor of safety higher than 1.9 to 2.0 would have been obtained for the design SSE conditions.

From the discussion presented above for the VEGP Units 1 and 2, it is concluded that there exists an adequate factor of safety against liquefaction for the compacted backfill.

2.5.4.8.3 Liquefaction Analyses Performed for the ESP Application

2.5.4.8.3.1 Liquefaction Analyses of the Upper Sands

Based on previous investigations and excavation completed for the existing VEGP Units 1 and 2 and their proximity to proposed VEGP Units 3 and 4, the Upper Sand Stratum will be completely removed and replaced with select compacted non-liquefiable fills back to the plant grade within the footprint of the planned power block.

Because select compacted non-liquefiable fills will be used to replace the Upper Sand Stratum in the power block area of proposed VEGP Units 3 and 4, no liquefaction study was performed for this ESP investigation.

2.5.4.8.3.2 Liquefaction Analyses of the Blue Bluff Marl

The Blue Bluff Marl is identified as a cemented, overconsolidated, calcareous fine-grained material (silt and clay), and thus exhibits high factor of safety against liquefaction. However, some lenses of silty fine sand were encountered during the COL investigation. Due to the presence of these materials, a review of the liquefaction potential of the Blue Bluff Marl is presented in the following paragraphs.

The present state-of-the-art considers an evaluation of data from SPT, CPT, and shear wave velocity (Vs) measurements, with the method employing SPT measurements being the most well-developed and well-recognized. Initially, a measure of the stress imparted to the soils by the ground motion is calculated, referred to as the cyclic stress ratio (CSR). Then, a measure of the resistance of soils to the ground motion is calculated, referred to as the cyclic resistance ratio (CRR). And finally, a factor of safety (FOS) against liquefaction is calculated as the ratio of the resisting stress, CRR, to the driving stress, CSR. Details of the liquefaction methodology and the relationships for calculating CSR, CRR, FOS, and other intermediate parameters such as the stress reduction coefficient (rd), the magnitude scaling factor (MSF), the $K\sigma$ correction factor accounting for liquefaction resistance with increasing confining pressure, and a host of other correction factors, can be found in **Youd et al. 2001**. A MSF of 1.11 was used in the analyses, based on the selected earthquake magnitude. A review of the results of liquefaction potential analyses using the available SPT and Vs data (CPT data was unavailable) for the Blue Bluff Marl in the power block are of Units 3 and 4 follows.

2.5.4.8.3.2.1 Liquefaction Potential Based on SPT Data

SPT N_{60}-values versus elevation are presented on Figure 2.5.4-18 for the 70 borings taken in the power block area of Units 3 and 4 for the COL investigation. With the assumption of clean sand (i.e., fines content, FS = 5%), the results show that most of the coarse-grained soil samples have corrected SPT blow counts, N1 or $(N1)_{60}$, greater than 30, indicating non-liquefiable. Among eight of the soil soils that are analyzed, only three of them are potentially liquefiable, with calculated factors of safety (FS) against liquefaction 1.43, 1.75, and 2.19. In all cases, the FS against liquefaction in the Blue Bluff Marl was greater than 1.1.

2.5.4.8.3.2.2 Liquefaction Potential Based on Shear Wave Velocity Data

Shear wave velocity (Vs) data measured in the Blue Bluff Marl by P-S logging in six borings taken in the power block for the COL investigation were evaluated for liquefaction potential. The The shear wave velocity values were corrected for overburden (Vs1) following recommendations in **Youd et al**. The calculated Vs1 values ranged from 253 meters/second to 508 m/s. The relationship between Vs1, CRR, and liquefaction potential presented by **Youd et al**. suggests that the Blue Bluff Marl is non-liquefiable based these calculated values.

2.5.4.8.3.3 Liquefaction Analyses of the Compacted Backfill

The backfill will be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D 1557. The Phase I test pad program (Appendix 2.5D) was conducted to evaluate backfill properties. Borrow sources and quantities have been identified as summarized in Section 2.5.4.5.3. Field and laboratory testing was conducted on these materials. Results from the testing of borrow sources and the test pad program including measured N-values and shear wave velocities are consistent with the results from Units 1 and 2. Figure 16 of Appendix 2.5D provides a plot of $N1_{(60)}$ vs depth and demonstrates an average $N1_{(60)}$ value (below a depth of 2 feet) equal to or greater than 40 bpf. In addition, the shear wave velocity profile for the backfill, as presented in Table 2.5.4-10a, demonstrates a shear wave velocity greater than 1,000 fps below a depth of 30 feet. Therefore, as determined for Units 1 and 2 and presented in Section 2.5.4.8.2 for the design basis earthquake, liquefaction is not a concern.

2.5.4.8.4 Liquefaction Conclusions

Based on the foregoing sections on the analysis of liquefaction potential, the following conclusions are made:

- Only the Upper Sand Stratum below the groundwater table falls into the gradation and relative density categories where liquefaction would be considered possible.
- The Upper Sand Stratum was completely removed and replaced with compacted structural fill before construction of the existing VEGP Units 1 and 2. The same approach will be used before construction of the proposed VEGP Units 3 and 4.
- The liquefaction potential of the compacted structural fill, consisting of materials and methods similar to VEGP Units 1 and 2 is not a concern.

- The Blue Bluff Marl is primarily cohesive but has some lenses of coarse-grained materials. These materials were found to have an adequate factor of safety against liquefaction (greater than 1.1).

2.5.4.9 Earthquake Design Basis

The site ground motion response spectra (GMRS) is derived and discussed in detail in Section 2.5.2.6. The Operating Basis Earthquake (OBE) is discussed in Section 2.5.2.8.

2.5.4.10 Static Stability

All safety-related structures will be founded on the structural backfill that will be placed on top of the Blue Bluff Marl after complete removal of the Upper Sand Stratum. The base of the Containment and Auxiliary Building foundations for VEGP Units 3 and 4 will be about El. 180 ft msl. This level corresponds to a depth of 40 ft below final grade (below El. 220 ft msl), or 50 to 60 ft above the top of the Blue Bluff Marl bearing stratum based on the borings completed during the ESP and COL subsurface investigations. Other foundations in the power block area will be placed at depths of about 4 ft below final grade. The following sections on bearing capacity and settlement focus on these two scenarios.

Based on the results of the ESP and COL investigations and the Phase I test pad program, the soils supporting the NIs do not exhibit extreme variations in subgrade stiffness and the site can be considered uniform according to **WEC (2008)**. As presented in Section 2.5.4.2.2.2, the subsurface data disclosed a nearly even top of Blue Bluff Marl (varying from El. 122 ft to El. 140 ft over the length of the excavation footprints) with relatively uniform thickness and consistent engineering properties. The earthwork specification for the compacted backfill will be developed after the completion of the Phase II test pad program which is expected to finalize the placement procedures and material types. The Phase I test pad program considered materials and placement procedures consistent with Units 1 and 2. Test results disclosed consistent engineering properties including density, shear wave velocity, and N-values as presented in Section 2.5.4.5.3.

A coefficient of friction of 0.45 against concrete can be expected for the sand and silty sand compacted backfill material. A site-specific stability evaluation was conducted by Westinghouse. Results are presented in Appendix 2.5E.

2.5.4.10.1 Bearing Capacity

All structures in the power block footprint will be founded on the structural backfill compacted to a minimum of 95% (ASTM D 1557) as presented in Section 2.5.4.5. The structural backfill will be about 90 ft thick in the power block area. The Nuclear Island will be founded at a depth of about 40 ft below grade (about 50 ft of structural backfill beneath the foundations). Other structures will be founded at an approximate depth of 4 ft below grade. The allowable static bearing capacity values are calculated with Terzaghi's bearing capacity equations. An internal angle of friction of 36° was used for the compacted backfill as developed from field and laboratory testing of borrow materials during the Phase I test pad program (Appendix 2.5D) and the COL investigation (Appendix 2.5C). The influence of the Blue Bluff Marl on the allowable bearing pressure was evaluated using procedures outlined by Vesic (1975). With a factor of safety of 3.0 (ASCE 1994), site conditions provide an allowable bearing pressure of 34 ksf under static loading conditions for the Nuclear Island, which is greater than the required 8.6 ksf (WEC CCC-004). An internal friction angle of 34° was used to calculate the allowable bearing capacity values for foundations placed on compacted fills at depths of about 4 ft below finished grade as provided in Figure 2.5.4-13.

The allowable bearing capacity of the structural backfill under the Nuclear Island for dynamic loading conditions was also evaluated using Terzaghi's bearing capacity equation for local shear (Peck et al. 1974) and Soubra's method with seismic bearing capacity factors **(Soubra 1999)** using Terzaghi's bearing capacity equation for general shear with an internal friction angle of 36°. To simulate the potential for higher edge pressures during dynamic loading, three foundation widths were considered (10, 25, and 50 ft) corresponding to 10, 25, and 50 percent of the width of the Nuclear Island basemat. The results from these two methods compared well, with Terzaghi's approach for local shear providing more conservative values. The computed average ultimate capacities of the three widths (10, 25, and 50 ft) were 89, 100, and 119 ksf, respectively. A width of 25 ft and a factor of safety of 2.25 (ASCE 1994) were used for site specific conditions providing an allowable bearing pressure greater than 42 ksf under dynamic loading conditions for the Nuclear Island. This value is greater than the required 35 ksf for dynamic bearing (WEC SC2-065) as well as the Vogtle site specific maximum dynamic demand (for the ESP soil profile as described in Appendix 2.5E) of 18 ksf.

The bearing capacity of the structural backfill was also evaluated in terms of the ratio of the ultimate bearing capacity to the structure demand. This capacity over demand (C/D) ratio provides an alternative measure of the margin of safety against bearing failure. These C/D ratios were evaluated for the static and dynamic demand conditions as provided by Westinghouse (WEC CCC-004 and WEC SCE-065), as well as the maximum dynamic demand from the Vogtle site specific seismic evaluation (Appendix 2.5E). The results are given below:

Condition	Static	Dynamic	Site-Specific Dynamic
Ultimate Capacity (C), ksf	102	100[a]	100[a]
Demand (D), ksf	8.6[b]	35[c]	18[d]
C/D	11.9	2.9	5.6

a. Based on a reduced foundation width of 25 feet to account for higher edge pressures during a seismic event.
b. APP-1000-CCC-004, Rev. 0, *Nuclear Island - Stability Evaluation*.
c. APP-1000-S2C-065, Rev. 0, *Nuclear Island Stick Model Analysis at Soil Sites*.
d. Based on analysis using ESP profile in Appendix 2.5E.

The C/D ratios are higher than those typically used for standard practice. While these results do not take into account settlement of the structures, the significant margin suggests that settlements will be minimal and within the design requirements (WEC SC2-065). A further discussion of settlement is provided in Subsection 2.5.4.10.2.

The results of settlement analyses are presented in Section 2.5.4.10.2.

2.5.4.10.2 Settlement Analysis

For the large mat foundations that support the major power plant structures, general considerations based on previous site experience **(Bechtel 1986)** indicate that the total settlement can exceed the suggested limit of 2 in. encountered in the geotechnical literature (Peck et al. 1974). Settlement monitoring of VEGP Units 1 and 2 **(Bechtel 1986)** disclosed foundation settlements ranging from 2.7 to 3.2 in. for the containment buildings, versus calculated/design values of 4.0 to 4.3 in. Similar results were obtained for the control building (measured settlements ranging from 1.1 to 1.9 inches versus calculated/design values of 3.2 to 3.4 in.), auxiliary building (measured settlements ranging from 2.9 to 3.3 in. versus calculated/design values of 4.4 to 4.6 in.), and the NSCW towers (measured settlements ranging from 2.5 to 3.6 in. versus calculated/design values of 4.5 to 4.8 in.). The ratio of measured to predicted settlement for these structures ranged from less than 0.5 to about 0.75, indicating that the subsurface soils were generally stiffer than anticipated.

Similarly, the measured differential settlements between mats of Units 1 and 2 **(Bechtel 1986)**, which can affect pipe connections, was generally within the suggested limit of ¾ in. encountered in the geotechnical literature **(Peck et al. 1974)**. The measured differential settlements within structures of Units 1 and 2 were smaller than the design limit of 1/670 **(Bechtel 1986)**.

It is noted that settlements reported for Units 1 and 2 **(Bechtel 1986)** were essentially elastic, i.e., they took place during construction. This reflects the elastic nature of the compacted backfill, the heavily overconsolidated Blue Bluff Marl, and the underlying Lower Sand Stratum.

For footings that support smaller plant components, the total settlement can be limited to 1 inch, while the differential settlement between footings can be limited to ½ in. **(Peck et al. 1974)**.

The general approach used for Units 1 and 2 consisted of estimating total and differential settlements for powerblock structures and using them as design values. A detailed settlement monitoring program was established, and measured settlements were compared with the design values. Re-analysis and/or corrective measures were employed if measured settlements exceeded design or trigger values. An additional strategy consisted of installing pipes as late in the construction schedule as practicable and installing pipe supports only when construction of the structure that the pipe was connected to was essentially complete.

Laboratory consolidation tests were conducted on relatively undisturbed samples from the Blue Bluff Marl and the Lower Sand strata collected during the COL investigation. These results are included in Appendix 2.5C, and they confirm the elastic behavior and very stiff and dense nature of the Blue Bluff Marl and Lower Sand strata. A test fill program has been performed (Section 2.5.4.5.8) to assess properties of compacted backfill. The results confirm the very dense nature, similar to Units 1 and 2, of the fill and the expected performance under load.

A detailed settlement analysis has been carried out for Units 3 and 4 utilizing similar elastic properties used for the analysis of Units 1 and 2. The analysis incorporated excavation, dewatering, and a timeline of construction to estimate, as much as practical, mat displacement time histories. The results show that for the assumed loads, predicted total settlements range from about 2 to 3 inches, with a tilt of approximately ¼ inch in 50 feet, and a differential settlement between structures of less than 1 inch **(Rizzo 2008)**. In addition, predicted heave due to foundation excavation ranges from about 1 to 2 ½ inches **(Rizzo 2008)**. As expected, the results are similar to movements measured for Units 1 and 2.

2.5.4.10.2.1 Displacement Monitoring

An instrumentation plan will be developed to monitor heave in the subsurface soils due to excavation, change in pore pressures due to excavation and dewatering, and settlement due to construction of the structures. The detailed plan to be developed will include displacement monitoring at depth in order to estimate and confirm moduli of the subsurface soils. Estimates of unloading (excavation) and loading will be made to correlate movement with load and to update the movement time histories discussed in 2.5.4.10.2.

The instrumentation will be monitored on a regular basis and will include conventional survey, electronic instrumentation, and where practical, remote telemetry. Particular emphasis will be placed on differential movement and tilt of the structures.

2.5.4.11 Design Criteria

Applicable geotechnical-related design criteria are discussed in various sections of the SSARand are summarized below.

Section 2.5.4.8 specifies that the acceptable factor of safety against liquefaction of site soils should be ≥ 1.1 in accordance with Regulatory Guide 1.198.

Bearing capacity criteria are presented in Section 2.5.4.10. A minimum factor of safety of 3 is used when applying bearing capacity equations. This factor of safety is also applied against breakout failure due to uplift forces on buried piping. For soils, this factor of safety can be reduced to 2.25 when dynamic or transient loading conditions apply **(ASCE 1994)**.

Section 2.5.5.2 specifies that the minimum acceptable long-term static factor of safety against slope stability failure is 1.5 and that the minimum acceptable long-term seismic factor of safety against slope stability failure is 1.1 **(USACE 2003)**.

Appendix 2.5E describes the site-specific analyses that have been performed to show the acceptability of the AP1000 plant at the Vogtle site.

2.5.4.12 Techniques to Improve Subsurface Conditions

For the ESP investigation, ground improvement techniques were not considered beyond the removal and replacement of the Upper Sand Stratum. Likewise, no additional ground improvement methods are being considered based on the COL investigation. The Phase I test pad program (Appendix 2.5D) presents the field and test results of the materials and methods that are currently planned for the backfill to replace the Upper Sand Stratum. For areas outside the power block excavation, surficial ground can be improved through densification with heavy vibratory rollers. Other ground improvement methods and the use of piles will be considered as warranted.

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

Parameter[1]	Stratum: Upper Sand	Compacted Structural Fill	Blue Bluff Marl	Lower Sand
Range of Thickness, 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 N60-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	3,088	3,820	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_0)	N/A	0.5	N/A	N/A
Coefficient of Sliding	N/A	0.45	N/A	N/A
Poisson's Ratio	0.09–0.49[5]		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 depths of 0 to 100 ft below the bottom of the Blue Bluff Marl.

[4]This value applies between depths of 100 to 300 ft below the bottom of the Blue Bluff Marl.

[5]Values not determined during COL investigation, retain ESP values.

Table 2.5.4-1a Static Engineering Properties of Subsurface Materials (COL)

Parameter[1]	Stratum			
	Upper Sand	Compacted Structural Fill	Blue Bluff Marl	Lower Sand(5)
Range of Thickness, feet	82 to 94	82 to 94	60 to 77	900
Average thickness, feet	88	88	68	900
USCS symbol	SP/SM/SC/ML	SP/SM	CL/CH/ML/MH	SP/SM/ML/CL
Natural moisture content (ω), %	N/A	N/A	32	N/A
Unit weight (pcf)	113	123 (moist) 133 (saturated)	115	120
Atterberg Limits				
Liquid limit (LL), %	N/A[2]	N/A	63	N/A
Plastic limit (PL), %	N/A	N/A	33	N/A
Plasticity index (PI), %	N/A	N/A	30	N/A
Measured SPT N-value, bpf	18	32	71	50
Adjusted SPT N_{60}-value, bpf	25	43	95	62
Strength properties				
Undrained shear strength (c_u), ksf	—	-	10	—
Internal friction angle (Ø'), degrees	34	36	0	34
Elastic modulus (high strain) (E_s), ksf	900	1,500	9,000	10,800[3] 13,500[4]
Shear modulus (high strain) (G_s), ksf	350	520	3,000	4,200[3] 5,200[4]
Shear modulus (low strain) (G_{max}), ksf	3,100	3,800	15,000	20,538
Coefficient of Subgrade Reaction (k_1), tcf	N/A	1,000	N/A	N/A
Earth Pressure Coefficients				
Active (K_a)	N/A	0.26	N/A	N/A
Passive (K_p)	N/A	3.9	N/A	N/A
At Rest (K_0)	N/A	0.4	N/A	N/A
Coefficient of Sliding	N/A	0.45	N/A	N/A
Poisson's Ratio	0.09–0.49[5]	0.24–0.45	0.43	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 depths of 0 to 100 ft below the bottom of the Blue Bluff Marl.

[4]This value applies between depths of 100 to 300 ft below the bottom of the Blue Bluff Marl.

[5]Values not fully evaluated during COL investigation, ESP values retained.

Table 2.5.4-2 Design Dynamic Shear Modulus (ESP)

Geologic Formation	Depth (ft)	Elevation (ft)	G_{max} (ksf)
Upper Sand Stratum	0 to 16	223 to 207	7,000
(Barnwell Group)	16 to 41	207 to 182	2,286
	41 to 58	182 to 165	2,580
	58 to 86	165 to 137	2,893
Blue Bluff Marl	86 to 92	137 to 131	6,978
(Lisbon Formation)	92 to 97	131 to 126	10,321
	97 to 102	126 to 121	15,750
	102 to 105	121 to 118	10,321
	105 to 111	118 to 112	17,286
	111 to 123	112 to 100	19,723
	123 to 149	100 to 74	25,080
Lower Sand Stratum	149 to 156	74 to 67	14,286
(Still Branch)	156 to 216	67 to 7	9,723
(Congaree)	216 to 331	7 to -108	13,580
(Snapp)	331 to 438	–108 to -215	15,009
(Black Mingo)	438 to 477	–215 to -254	19,723
(Steel Creek)	477 to 587	–254 to -364	25,080
(Gaillard/Black Creek)	587 to 798	–364 to -575	29,009
(Pio Nono)	798 to 858	–575 to -635	29,418
(Cape Fear)	858 to 1,049	–635 to -826	26,229
Dunbarton Triassic Basin	1,049		

Note: Gmax was calculated using g from Table 2.5.4-1, and the shear wave velocity values from Table 2.5.4-6.

Table 2.5.4-3 Types and Numbers of Laboratory Tests Completed for the ESP Application

Type of Test	Number of Tests Performed
Grain size	61
Unit Weight	31
Natural Moisture Content	75
Atterberg Limits	27
UU Triaxial (1-point)	15

Table 2.5.4-3a Types and Numbers of Completed Laboratory Tests in the Powerblock Footprint for the COL Investigation

Type of Test	Number of Tests Performed
Moisture Content	113
Wash #200	272
Unit Weight	100
Atterberg Limits	109
Chemical Analysis	5
Unconfined Compression	27
Unconsolidated Undrained Triaxial	11
Consolidated Undrained Triaxial	27
1-D Consolidation	18
Resonant Column Torsional Shear	19

Table 2.5.4-4 Summary of Laboratory Tests Performed on Selected Soils Samples from ESP Borings

SAMPLE DETAILS						SOIL TESTING							
Boring No.	Top Depth (ft)	Length (ft)	Type	Formation	SPT N-value (bpf)	% Fines	γ (pcf)	ω_N (%)	PL (%)	LL (%)	PI (%)	USCS Classification	UU s_u (ksf)
B-1002	7.5	1.5	SS	Fill	20	9.4		6.2					
	18.5	1.5	SS	Barnwell	19	37.1		24.4					
	28.5	1.5	SS	Barnwell	8	24.9		31.8					
	33.5	1.5	SS	Barnwell	6	31.6		58.8					
	38.5	1.5	SS	Barnwell	7			92.8	27	48	21		
	53.5	1.5	SS	Barnwell	8	10.5		42.9					
	63.5	1.5	SS	Barnwell	13	7.2		29.3					
	73.5	1.5	SS	Barnwell	12	10		24.5					
	83.5	1.5	SS	Barnwell	9	6.1		27.6					
	92.0	2.5	UD-Upper	Lisbon	N/A	67.2	103.6	52.1	50	83	33	MH	1.15
			UD-Middle				102.4						3.35
	103.5	2.5	UD	Lisbon	N/A	35.9	114.3	56.6	22	34	12	CL	
							114.5	26.5					2.4
	113.5	2.5	UD	Lisbon	N/A	33.8	132.8	25.5	19	29	10	SC	
							132.9	16.3					2.15
	123.5	2.5	UD	Lisbon	N/A	24.5	140.2	13.5	17	22	5	GC-GM	
	133.5	2.0	UD	Lisbon	N/A	96.6	118.0	28.6	26	40	14	CL	
							118.1	29.8					2.4
	153.5	1.5	SS	Lisbon	27	39.4		23.3	21	34	13	ML	
	188.5	1.5	SS	Still Branch	9	6.6		40.7	NP	NP	NP	SM	
	238.5	1.5	SS	Congaree	77	12.3		18.5					
B-1003	15	5	C	Barnwell	N/A	20.9		13.4					
	35	5	C	Barnwell	N/A	29.8		42.1					
	55	5	C	Barnwell	N/A	13.4		17.5					
	75	5	C	Barnwell	N/A	8.2		32.3					

Table 2.5.4-4 (cont.) Summary of Laboratory Tests Performed on Selected Soils Samples from ESP Borings

SAMPLE DETAILS						SOIL TESTING							
Boring No.	Top Depth (ft)	Length (ft)	Type	Formation	SPT N-value (bpf)	% Fines	γ (pcf)	ω_N (%)	PL (%)	LL (%)	PI (%)	USCS Classification	UU s_u (ksf)
B-1003	88	5	C	Lisbon	N/A	33.4		67.4	42	93	51	SM	
	93	2.5	UD-1	Lisbon	N/A	40.6	115.7	30.6	32	54	22	SM	
							115.8	29.5					4.3
	104.7	2	C	Lisbon	N/A	31.7	111.5	40.6	51	83	32	SM	
	121.7	5	C	Lisbon	N/A	42.5	122.5	28.0	NP	NP	NP	SM	
	141.7	5	C	Lisbon	N/A	34.2	126.1	25.9	28	46	18	SM	
B-1003	165.7	5	C	Still Branch	N/A	5.4	121.7	23.6	NP	NP	NP	SP-SM	
	185.7	5	C	Still Branch	N/A	16.4		32.3					
	205.7	5	C	Still Branch	N/A	21.4		39.3					
	240.7	5	C	Congaree	N/A	10.9		23.2					
	280.7	5.0	C	Congaree	N/A	14.2		23.2					
	315.7	5.0	C	Congaree	N/A	79.8		32.7	24	43	19	CL	
							119.4	31.0					
	350.7	5.0	C	Snapp	N/A	78.5	128.3	21.3	22	41	19	ML	
	400.7	5.0	C	Snapp	N/A	15.8		18.9					
	450.7	5.0	C	Black Mingo	N/A	15.9		28.6					
	496.7	5.0	C	Steel Creek	N/A	13.2		26.4					
B-1004	9.0	1.5	SS	Barnwell	13	24.4		13.8					
	12.0	1.5	SS	Barnwell	12	23.1		14.5					
	23.5	1.5	SS	Barnwell	8	14.9		18.5					
	43.5	1.5	SS	Barnwell	4	60.0		46.2	24	58	34	ML	
	53.5	1.5	SS	Barnwell	7	41.0		62.9					
	68.5	1.5	SS	Barnwell	6	19.9		24.1					
	83.5	1.5	SS	Barnwell	6	11.5		28.8					
	123.5	1.5	SS	Barnwell	5	19.2		19.7	19	43	24	GM	

Table 2.5.4-4 (cont.) Summary of Laboratory Tests Performed on Selected Soils Samples from ESP Borings

SAMPLE DETAILS						SOIL TESTING							
Boring No.	Top Depth (ft)	Length (ft)	Type	Formation	SPT N-value (bpf)	% Fines	γ (pcf)	ω_N (%)	PL (%)	LL (%)	PI (%)	USCS Classification	UU s_u (ksf)
B-1004	144.0	1.5	UD-Upper	Lisbon	N/A	46.3	105.1	44.6	38	59	21	SM	
							105.2	52.0					0.15
			UD-Middle				114.2	29.8					0.8
	153.5	1.5	UD	Lisbon	N/A	41.7		30.1	27	43	16	SM	
							117.4	25.2					
							119.3	28.7					3.75
	163.5	2.5	UD-Upper	Lisbon	N/A	58.3		25.1	37	55	18	MH	
							117.4	30.2					1.05
			UD-Middle				125.6	24.5					1.2
	177.0	2.5	UD-Upper	Lisbon	N/A	41.7	124.7	20.8	22	31	9	SM	
							124.6	22.4					0.8
			UD-Middle				131.8	39.2					1.9
B-1004	188.5	2.0	UD	Lisbon	N/A	75.2	120.4	29.0	24	41	17	CL	
							120.6	28.4					4.0
	198.5	2.0	UD	Lisbon	N/A	34.5	128.1	26.2	21	31	10	SM	
							128.2	21.7					3.0
B-1006	7.5	1.5	SS	Barnwell	3	7.3		3.8					
	33.5	1.5	SS	Barnwell	13	26.1		19.7					
	58.5	1.5	SS	Barnwell	W HAMM	58.3		92.8	30	97	67	CH	
	68.5	1.5	SS	Barnwell	W HAMM	3.1		25.4					
	88.5	1.5	SS	Barnwell	W HAMM	15.7		51.9					
	108.5	1.5	SS	Barnwell	42	21.5		22.0					
	123.5	1.5	SS	Lisbon	50/2"	64.1		53.7	43	99	56	MH	

Table 2.5.4-4 (cont.) Summary of Laboratory Tests Performed on Selected Soils Samples from ESP Borings

SAMPLE DETAILS						SOIL TESTING							
Boring No.	Top Depth (ft)	Length (ft)	Type	Formation	SPT N-value (bpf)	% Fines	γ (pcf)	ω_N (%)	PL (%)	LL (%)	PI (%)	USCS Classification	UU s_u (ksf)
B-1010	7.5	1.5	SS	Barnwell	27	7.8		5.7					
	33.5	1.5	SS	Barnwell	23	17.0		18.9					
	58.5	1.5	SS	Barnwell	19	13.3		27.3					
	73.5	1.5	SS	Barnwell	6	23.9		30.8					
	98.5	1.5	SS	Lisbon	77	91.3		49.9	47	94	58	CH	

Legend:
NP = non-plastic
ωN = natural moisture content
γ = unit weight
% Finer = % finer than the #200 sieve
PL = plastic limit
LL = liquid limit
PI = plasticity index
UU s_u = undrained strength from UU triaxial test
SS = split spoon or split barrel sample
UD = undisturbed sample
UD-Upper = test specimen taken from top of UD sample
UD-Middle = test specimen taken from middle of UD sample
C = soil core
W HAMM = weight of hammer (sampler penetrated at least 18" under the weight of the hammer, no blows applied by the hammer)

Table 2.5.4-5 Summary of SPT N-Values Measured at the ESP Borings

Boring Number	Measured SPT N-value (blows/ft) for Different Formations		
	Upper Sand Stratum (Barnwell Group)	Blue Bluff Marl (Lisbon Formation)	Lower Sand Stratum
B-1001	47, 32, 22, 22, 22, 23, 21, 23, 23, 37, 13, 10, 7, 5, 6, 12, 13, 30, 11, 37, 36, 47, WOR, 50/5"	50/5", 50/4", 51, 50/4", 50/6", 50/4", 50/5"	Not measured
B-1002	30, 67, 28, 33, 19, 10, 8, 6, 7, 12, 22, 8, 11, 13, 18, 12, 10, 9	77/11", 68/7", 54, 72, 50/2", 78/8", 65, 40, 27	46, 26, 50/4", 40, 9, 43, 32, 41, 50, 77
B-1004	21, 24, 25, 16, 16, 13, 19, 12, 14, 10, 8, 17, 13, 14, 4, 5, 7, 7, 18, 6, 5, 9, 5, 5, 17, 11, 16, 20,18, 34, 5, 9, 50/5"	77, 50/4", 50/0", 50/3", 50/3", 77, 79, 50/5", 50/4", 70/10", 81, 78, 58	79/10", 35, 50/5", 95, 47, 104
B-1005	27, 29, 26, 15, 11, 11, 10, 17, 13, 19, 17, 19, 11, 7, WOH, 37, 17, 34, 28, 25, 50/1", 56, 37,69, 46, 54, 57, 33, 31, 37, 95, 30, 32, 50/4", 80/9", 39	50/5", 50/4"	Not measured
B-1006	19, 20, 15, 9, 2, 3, 4, 8, 10, 11, 30, 24, 17, 13, 10, 2, 8, 7, WOH, 9, WOH, WOH, 13, 7, WOH,14, 19, 28, 42, 50	50/5", 50/2"	Not measured
B-1007	30, 32, 10, 10, 8, 14, 23, 20, 27, 26, 31, 25, 23, 15, 15, 24, 21, 26, 36, 37, 27, 36, 18, 13	50/2", 50/3", 45, 50/2", 50/5", 50/4", 74	Not measured
B-1008	19, 30, 53, 67, 34, 31, 19, 24, 30, 36, 30, 20, 17, 17, 25, 18, 22, 33, 39, 22, 25, 50/5", 50/4", 50/5"	46, 65, 53, 71/9", 50/3", 50/3", 50/4"	Not measured
B-1009	19, 37, 42, 44, 20, 21, 27, 21, 20, 30, 29, 35, 19, 31, 37, 42, 23, 13, 27, 32, 20, 8, 10, 40, 24	51, 50/5"	Not measured
B-1010	13, 18, 29, 24, 20, 27, 9, 13, 18, 29, 72, 23, 27, 23, 30, 26, 15, 34, 19, 6, 28, 6, 20, 10, 15, 21	67, 50/4"	Not measured
B-1011	8, 7, 11, 10, 14, 15, 15, 20, 13, 44, 42, 12, 25, 48, 28, 41, 37, 49, 60, 40, 50/0", 50/4"	69, 74, 50/3", 50/1", 36	Not measured
B-1013	9, 14, 26, 26, 12, 26, 26, 33, 9, 22, 16, 41, 16, 34, 22, 25, 21, 28, 12, 26, 15, 8, 18, 36, 13, 26	50/2", 76	Not measured
Range:	WOR-50/0"	27-50/1"	9-50/4"
Average:	25	83	59
Median	21	100	47

NOTES:

[a]SPT blow counts will be adjusted to reflect the measured hammer efficiencies.

[b]WOR means that the sampler penetrated 18" or more under weight of the rods, and WOH means that the sampler penetrated 18" or more under weight of the rods and hammer. These values were taken as zero when calculating the average.

[c]SPT blow counts linearly extrapolated to more than 100 bpf were truncated at 100 bpf when calculating the average.

[d]SPT N-values shown for the Barnwell Group exclude measurements in the fill layers encountered at borings B-1001, B-1002, B-1004, and B-1005.

Table 2.5.4-6 Typical Shear Wave Velocity Values for Existing Strata (ESP)

Geologic Formation	Depth (ft)	Elevation (ft)	V_s (fps)
Upper Sand Stratum	0 to 16	223 to 207	1,400
(Barnwell Group)	16 to 41	207 to 182	800
	41 to 58	182 to 165	850
	58 to 86	165 to 137	900
Blue Bluff Marl	86 to 92	137 to 131	1,400
(Lisbon Formation)	92 to 97	131 to 126	1,700
	97 to 102	126 to 121	2,100
	102 to 105	121 to 118	1,700
	105 to 111	118 to 112	2,200
	111 to 123	112 to 100	2,350
	123 to 149	100 to 74	2,650
Lower Sand Stratum	149 to 156	74 to 67	2,000
(Still Branch)	156 to 216	67 to 7	1,650
(Congaree)	216 to 331	7 to -108	1,950
(Snapp)	331 to 438	-108 to -215	2,050
(Black Mingo)	438 to 477	-215 to -254	2,350
(Steel Creek)	477 to 587	-254 to -364	2,650
(Gaillard/Black Creek)	587 to 798	-364 to -575	2,850
(Pio Nono)	798 to 858	-575 to -635	2,870
(Cape Fear)	858 to 1,049	-635 to -826	2,710
Dunbarton Triassic Basin	1,049	-826	2,710
	1,093	-870	5,300
	1,323	-1,100	7,800

Table 2.5.4-7 Summary of ESP Borings and CPTs

Boring Number	Plant Coordinates Northing (ft)	Plant Coordinates Easting (ft)	State Coordinates Northing (ft)	State Coordinates Easting (ft)	Elevation (ft msl)	Depth (ft)
B-1001	7,662	6,220	1,142,662	620,220	221.64	123.9
B-1002[a,b]	7,999	6,985	1,142,999	620,985	221.98	260
B-1002A[a,d]	7,986	6,986	1,142,986	620,986	222.27	105
B-1003[a,b,c]	7,974	7,890	1,142,974	621,890	223.21	1338
B-1004[a,b]	7,985	6,131	1,142,985	620,131	249.78	304
B-1005	8,992	6,155	1,143,992	620,155	253.14	164.3
B-1006	8,810	7,343	1,143,810	621,343	255.95	124.2
B-1007	7,662	7,120	1,142,662	621,120	221.02	125
B-1008	7,671	7,996	1,142,671	621,996	219.51	124.3
B-1009	6,001	6,361	1,141,001	620,361	220.39	98.9
B-1010	6,000	7,280	1,141,000	621,280	218.60	104.3
B-1011	8,741	8,378	1,143,741	622,378	219.38	100
B-1013	5,976	8,272	1,140,976	622,272	218.62	105
C-1005A[a,d]	7,990	8,179	1,142,990	622,179	223.66	90
CPT Number	**Plant Coordinates Northing (ft)**	**Plant Coordinates Easting (ft)**	**State Coordinates Northing (ft)**	**State Coordinates Easting (ft)**	**Elevation (ft msl)**	**Depth (ft)**
C-1001A	8,028	6,356	1,143,028	620,356	248.57	116.7
C-1002	7,668	6,575	1,142,668	620,575	222.13	78.5
C-1003[e,f]	7,669	7,478	1,142,669	621,478	219.80	80
C-1004[f]	7,646	8,362	1,142,646	622,362	220.82	77
C-1005[e,f]	7,995	8,175	1,142,995	622,175	223.81	82
C-1006	8,001	7,262	1,143,001	621,262	222.80	74
C-1007	8,271	8,055	1,143,271	622,055	222.81	81.7
C-1008	8,268	6,931	1,143,268	620,931	221.30	76
C-1009A[e,f]	5,980	6,798	1,140,980	620,798	218.93	99
C-1010	6,008	7,754	1,141,008	621,754	219.06	96

[a] Location of suspension P-S velocity logging.
[b] Location of caliper, natural gamma, resistivity, and spontaneous potential measurements.
[c] Location of borehole deviation survey.
[d] Boreholes drilled without sampling to allow the performance of suspension P-S velocity logging above the zone of drilling fluid loss.
[e] Location of seismic CPT.
[f] Location of pore pressure dissipation tests.

Note: State Plane Coordinates are from NAD27 Georgia East state grid system. Plant coordinates are converted from the following formula:

Plant North + 1,135,000 = State North

Plant East + 614,000 = State East

Table 2.5.4-7a Summary of COL Borings, CPTs, and Test Pits

	Plant Coordinates		State Coordinates			
Boring Number	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)	Elevation (ft, msl)	Depth (ft)
B-1105	9,168	6,003	1,144,168	620,003	257.89	148.8
B-1107	9,154	6,916	1,144,154	620,916	266.66	150.0
B-1108	9,214	7,273	1,144,214	621,273	273.56	149.8
B-1109	9,180	7,581	1,144,180	621,581	276.48	150.0
B-1110	9,171	8,011	1,144,171	622,011	265.14	150.0
B-1111	9,213	8,334	1,144,213	622,334	224.90	150.0
B-1112	9,223	8,691	1,144,223	622,691	213.74	23.0
B-1112A	9,219	8,561	1,144,219	622,561	227.14	150.0
B-1113	8,901	6,217	1,143,901	620,217	249.99	170.0
B-1116	8,894	7,265	1,143,894	621,265	261.82	138.5
B-1117	8,891	7,628	1,143,891	621,628	263.89	149.3
B-1118	8,886	8,008	1,143,886	622,008	257.91	149.4
B-1119	8,888	8,334	1,143,888	622,334	223.57	150.0
B-1120	8,893	8,558	1,143,893	622,558	227.18	149.8
B-1121	8,576	6,216	1,143,576	620,216	241.33	150.0
B-1123	8,575	6,922	1,143,575	620,922	241.27	150.0
B-1124	8,628	7,422	1,143,628	621,422	241.21	150.0
B-1125	8,587	7,628	1,143,587	621,628	240.97	150.0
B-1126	8,568	7,980	1,143,568	621,980	219.88	150.0
B-1127	8,573	8,332	1,143,573	622,332	219.67	150.0
B-1128	8,573	8,682	1,143,573	622,682	218.26	73.0
B-1128A	8,574	8,685	1,143,574	622,685	217.92	148.8
B-1129	8,278	7,894	1,143,278	621,894	221.84	100.0
B-1130	7,483	8,250	1,142,483	622,250	217.46	99.2
B-1131	8,173	7,823	1,143,173	621,823	222.18	98.6
B-1132	7,614	7,450	1,142,614	621,450	218.73	100.0
B-1133	7,969	7,451	1,142,969	621,451	221.20	100.0
B-1134	8,283	7,104	1,143,283	621,104	222.04	100.0
B-1136	8,178	7,023	1,143,178	621,023	221.65	100.0
B-1138	8,470	5,193	1,143,470	619,193	215.82	100.0
B-1139	7,290	7,027	1,142,290	621,027	216.68	150.0
B-1140	7,290	7,824	1,142,290	621,824	216.58	150.0
B-1142	9,417	6,650	1,144,417	620,650	224.69	100.0
B-1146	10,428	8,272	1,145,428	622,272	240.04	98.6
B-1148	10,538	9,237	1,145,538	623,237	218.94	100.0
B-1150	10,467	10,235	1,145,467	624,235	170.69	100.0
B-1152	10,582	11,227	1,145,582	625,227	117.05	100.0
B-1153	10,569	11,673	1,145,569	625,673	103.58	100.0
B-1154	10,664	12,216	1,145,664	626,216	95.08	98.8

Table 2.5.4-7a (cont.) Summary of COL Borings, CPTs, and Test Pits

	Plant Coordinates		State Coordinates			
Boring Number	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)	Elevation (ft, msl)	Depth (ft)
B-1155	12,390	10,936	1,147,390	624,936	84.95	150.0
B-1156	12,302	10,572	1,147,302	624,572	85.70	99.2
B-1157	12,210	11,062	1,147,210	625,062	86.77	150.0
B-1158	10,195	12,669	1,145,195	626,669	88.74	149.5
B-1159	12,286	10,955	1,147,286	624,955	88.70	150.0
B-1161	12,363	10,862	1,147,363	624,862	86.10	150.0
B-1162	12,235	10,815	1,147,235	624,815	85.55	200.0
B-1163	12,171	10,939	1,147,171	624,939	85.95	150.0
B-1164	11,995	10,519	1,146,995	624,519	220.50	150.0
B-1166	12,453	9,962	1,147,453	623,962	203.40	100.0
B-1168	12,688	9,468	1,147,688	623,468	202.20	100.0
B-1170	12,424	8,954	1,147,424	622,954	223.29	98.9
B-1172	11,983	8,539	1,146,983	622,539	249.49	100.0
B-1174	11,476	8,228	1,146,476	622,228	225.81	100.0
B-1176	10,876	8,195	1,145,876	622,195	221.48	35.0
B-1176A	10,879	8,197	1,145,879	622,197	221.51	100.0
B-1185	9,717	8,232	1,144,717	622,232	226.78	148.9
B-1186	9,712	4,819	1,144,712	618,819	277.51	178.8
B-1187	9,710	5,260	1,144,710	619,260	277.68	150.0
B-1189	9,460	4,997	1,144,460	618,997	279.98	150.0
B-1191	9,302	5,491	1,144,302	619,491	260.30	150.0
B-1192	9,217	4,841	1,144,217	618,841	243.17	179.5
B-1193	9,091	5,278	1,144,091	619,278	254.11	178.8
B-1194	12,505	7,630	1,147,505	621,630	199.35	50.0
B-1195	12,575	8,478	1,147,575	622,478	220.60	50.0
B-1196	12,287	8,018	1,147,287	622,018	217.52	50.0
B-1197	11,875	8,004	1,146,875	622,004	245.60	50.0
B-3001(DH)	7,600	7,800	1,142,600	621,800	218.40	420.0
B-3002(DH)	7,600	7,872	1,142,600	621,872	218.89	249.9
B-3002A	7,598	7,879	1,142,598	621,879	218.83	21.5
B-3003(DH)	7,600	7,727	1,142,600	621,727	218.29	250.0
B-3004	7,447	7,867	1,142,447	621,867	218.51	160.0
B-3005	7,718	7,749	1,142,718	621,749	219.20	155.0
B-3006	7,426	7,925	1,142,426	621,925	217.59	155.0
B-3007	7,719	7,877	1,142,719	621,877	220.78	159.8
B-3008	7,425	7,773	1,142,425	621,773	217.86	155.0
B-3009	7,484	7,957	1,142,484	621,957	217.85	153.9
B-3010	7,635	8,025	1,142,635	622,025	219.69	160.0
B-3011	7,777	8,025	1,142,777	622,025	220.57	165.0
B-3012	7,773	7,912	1,142,773	621,912	220.40	159.3
B-3013(C)	7,843	7,825	1,142,843	621,825	220.51	155.0

Table 2.5.4-7a (cont.) Summary of COL Borings, CPTs, and Test Pits

	Plant Coordinates		State Coordinates			
Boring Number	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)	Elevation (ft, msl)	Depth (ft)
B-3014	7,799	7,749	1,142,799	621,749	220.26	158.7
B-3015	7,957	7,824	1,142,957	621,824	221.78	150.0
B-3016	7,978	7,913	1,142,978	621,913	222.48	150.0
B-3017	8,034	7,750	1,143,034	621,750	222.10	150.0
B-3018	7,738	8,116	1,142,738	622,116	219.80	155.0
B-3019	7,977	8,167	1,142,977	622,167	222.42	153.8
B-3020	7,978	8,075	1,142,978	622,075	222.44	149.4
B-3021	8,070	8,033	1,143,070	622,033	223.19	154.5
B-3022	8,070	7,873	1,143,070	621,873	223.86	150.0
B-3023	8,061	7,680	1,143,061	621,680	222.81	150.5
B-3024	7,906	7,400	1,142,906	621,400	220.16	150.0
B-3025	7,460	7,425	1,142,460	621,425	218.21	150.0
B-3026	7,290	7,404	1,142,290	621,404	215.76	149.2
B-3027	7,059	7,423	1,142,059	621,423	218.80	150.0
B-3028	6,867	7,409	1,141,867	621,409	220.12	150.0
B-3029	6,882	7,804	1,141,882	621,804	220.13	149.9
B-3030	6,700	7,800	1,141,700	621,800	221.99	150.0
B-3031	6,399	8,042	1,141,399	622,042	222.70	150.0
B-3032	6,158	7,710	1,141,158	621,710	220.05	149.5
B-3033	6,405	7,715	1,141,405	621,715	222.26	149.3
B-3034	6,400	7,915	1,141,400	621,915	224.67	149.2
B-3035	7,729	7,675	1,142,729	621,675	219.34	150.5
B-3036	7,442	7,676	1,142,442	621,676	217.87	155.0
B-3037	8,057	7,769	1,143,057	621,769	222.94	150.0
B-3038	6,883	7,543	1,141,883	621,543	220.76	98.9
B-3039	7,918	7,754	1,142,918	621,754	219.17	150.0
B-4001(DH)	7,600	7,000	1,142,600	621,000	218.88	399.9
B-4002(DH)	7,600	7,072	1,142,600	621,072	219.06	250.0
B-4003(DH)	7,600	6,927	1,142,600	620,927	218.99	249.8
B-4004	7,460	7,047	1,142,460	621,047	218.45	150.0
B-4005	7,715	6,949	1,142,715	620,949	221.13	164.9
B-4006	7,720	7,076	1,142,720	621,076	220.98	165.0
B-4007	7,426	7,125	1,142,426	621,125	217.90	170.0
B-4008	7,424	6,974	1,142,424	620,974	218.08	169.4
B-4009	7,486	7,157	1,142,486	621,157	217.91	164.9
B-4010	7,668	7,249	1,142,668	621,249	219.09	160.0
B-4011	7,773	7,236	1,142,773	621,236	219.08	150.0
B-4013(C)	7,843	7,020	1,142,843	621,020	222.24	165.0
B-4014	7,832	6,950	1,142,832	620,950	220.74	158.6
B-4015	7,773	7,115	1,142,773	621,115	220.11	155.0
B-4016	7,996	7,113	1,142,996	621,113	221.23	149.6

Table 2.5.4-7a (cont.) Summary of COL Borings, CPTs, and Test Pits

Boring Number	Plant Coordinates		State Coordinates		Elevation (ft, msl)	Depth (ft)
	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)		
B-4017	8,035	6,950	1,143,035	620,950	220.94	150.0
B-4018	7,735	7,316	1,142,735	621,316	220.30	160.0
B-4019	7,976	7,371	1,142,976	621,371	221.76	160.0
B-4020	7,969	7,280	1,142,969	621,280	222.79	89.4
B-4020A	7,974	7,280	1,142,974	621,280	222.56	165.0
B-4021	8,093	7,247	1,143,093	621,247	224.55	150.0
B-4022	8,081	7,074	1,143,081	621,074	220.71	148.7
B-4023	8,062	6,880	1,143,062	620,880	220.71	150.0
B-4024	7,905	6,602	1,142,905	620,602	223.80	150.0
B-4025	7,510	6,625	1,142,510	620,625	220.80	150.0
B-4026	7,330	6,598	1,142,330	620,598	221.54	150.0
B-4027	7,180	6,633	1,142,180	620,633	217.73	150.0
B-4028	6,984	6,588	1,141,984	620,588	219.57	150.0
B-4029	6,875	6,700	1,141,875	620,700	220.28	150.0
B-4030	6,677	6,698	1,141,677	620,698	222.35	150.3
B-4031	6,400	6,975	1,141,400	620,975	222.13	150.0
B-4032	6,118	6,795	1,141,118	620,795	220.24	38.5
B-4032A	6,124	6,795	1,141,124	620,795	220.22	150.0
B-4033	6,398	6,349	1,141,398	620,349	219.93	149.4
B-4034	6,376	6,795	1,141,376	620,795	222.79	150.0
B-4035	7,729	6,876	1,142,729	620,876	220.52	164.8
B-4036	7,457	6,876	1,142,457	620,876	218.05	170.0
B-5001	11,177	7,808	1,146,177	621,808	218.99	150.0
B-5002	11,340	7,808	1,146,340	621,808	241.53	150.0
B-5003	11,387	7,575	1,146,387	621,575	227.94	148.7
B-5004	11,548	7,568	1,146,548	621,568	236.61	149.8
B-6002	9,134	5,627	1,144,134	619,627	247.90	150.0
B-6003	8,925	5,423	1,143,925	619,423	229.76	179.4
B-6004	8,718	5,473	1,143,718	619,473	231.59	150.0
B-6005	8,718	5,874	1,143,718	619,874	242.59	178.8
B-6006	8,070	6,302	1,143,070	620,302	248.22	50.0
B-6007	7,731	6,302	1,142,731	620,302	222.28	50.0
B-6008	10,444	8,676	1,145,444	622,676	240.11	150.0
B-6009	9,774	7,748	1,144,774	621,748	246.04	100.0
B-6010	8,893	7,059	1,143,893	621,059	263.39	169.3
B-6011	9,558	7,262	1,144,558	621,262	244.00	120.0
B-6012	9,257	6,481	1,144,257	620,481	194.20	120.0
B-6013	8,170	3,235	1,143,170	617,235	251.14	50.0
B-6014	8,168	4,281	1,143,168	618,281	209.79	50.0
B-6015	8,166	5,318	1,143,166	619,318	221.52	50.0
B-6018	7,909	4,367	1,142,909	618,367	204.66	50.0

Table 2.5.4-7a (cont.) Summary of COL Borings, CPTs, and Test Pits

	Plant Coordinates		State Coordinates			
Boring Number	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)	Elevation (ft, msl)	Depth (ft)
B-6019	7,133	4,344	1,142,133	618,344	163.94	50.0
B-6020	7,634	5,556	1,142,634	619,556	221.48	130.0
B-6021	7,186	5,103	1,142,186	619,103	209.80	120.0
B-6022	7,225	6,040	1,142,225	620,040	216.23	90.0
B-6023	6,553	5,178	1,141,553	619,178	202.77	50.0
B-6024	6,546	5,998	1,141,546	619,998	216.07	50.0
B-6025	5,519	5,190	1,140,519	619,190	172.69	50.0
B-6026	5,538	5,900	1,140,538	619,900	215.46	50.0
B-6027	10,779	12,145	1,145,779	626,145	96.65	75.0
B-6028	10,611	12,062	1,145,611	626,062	95.70	50.0
B-6029	12,772	9,967	1,147,772	623,967	85.41	50.0
B-6030	12,588	10,223	1,147,588	624,223	88.37	50.0

	Plant Coordinates		State Coordinates			
CPT Number	Northing (ft)	Easting (ft)	Northing (ft)	Easting (ft)	Elevation (ft, msl)	Depth (ft)
C-1101	9,357	6,185	1,144,357	620,185	265.76	71.4
C-1102	9,424	7,333	1,144,424	621,333	267.61	51.4
C-1103	10,012	8,037	1,145,012	622,037	236.52	27.4
C-1104	10,602	8,747	1,145,602	622,747	230.19	77.1
C-1105	10,483	9,734	1,145,483	623,734	200.57	50.2
C-1106	10,534	10,748	1,145,534	624,748	138.02	20.0
C-1107	12,234	10,202	1,147,234	624,202	211.92	71.0
C-1108	12,628	9,753	1,147,628	623,753	200.89	59.6
C-1109	12,622	9,172	1,147,622	623,172	209.79	72.5
C-1110	12,199	8,740	1,147,199	622,740	242.39	72.3
C-1111	11,753	8,346	1,146,753	622,346	250.69	32.2
C-3001(S)	7,611	7,727	1,142,611	621,727	218.37	70.1
C-3002(S)	7,607	7,873	1,142,607	621,873	218.89	67.9
C-3003(S)	6,772	7,802	1,141,772	621,802	221.38	82.0
C-3004	6,542	7,807	1,141,542	621,807	223.25	72.7
C-3005(S)	6,267	7,792	1,141,267	621,792	221.27	101.1
C-4001(S)	7,600	6,919	1,142,600	620,919	218.87	74.2
C-4002(S)	7,600	7,064	1,142,600	621,064	219.08	82.2
C-4003(S)	6,785	6,708	1,141,785	620,708	221.16	82.5
C-4004	6,543	6,598	1,141,543	620,598	219.99	77.1
C-4005(S)	6,250	6,594	1,141,250	620,594	220.01	90.2

Table 2.5.4-7a (cont.) Summary of COL Borings, CPTs, and Test Pits

Test Pit Number	Plant Coordinates Northing (ft)	Plant Coordinates Easting (ft)	State Coordinates Northing (ft)	State Coordinates Easting (ft)	Elevation (ft, msl)	Depth (ft)
TP-B-1108	9,312	7,146	1,144,312	621,146	264.14	12.2
TP-B-1117	8,967	7,628	1,143,967	621,628	269.50	9.0
TP-B-1121	8,592	6,402	1,143,592	620,402	241.17	14.0
TP-B-1125	8,604	7,686	1,143,604	621,686	240.61	11.0
TP-B-1185	9,634	8,242	1,144,634	622,242	225.17	11.0
TP-B-1194	12,501	7,708	1,147,501	621,708	202.73	11.5
TP-B-1195	12,648	8,363	1,147,648	622,363	212.15	8.0
TP-B-1197	11,874	8,075	1,146,874	622,075	245.94	11.0

(DH) - Location of suspension P-S velocity logging and/or geophysical measurements.

(S) - Location of seismic CPT.

(C) – Borings with continuous sampling

Note: State Plane Coordinates are from NAD27 Georgia East state grid system. Plant coordinates are converted from the following formula:

Plant North + 1,135,000 = State North

Plant East + 614,000 = State East

Plant vertical datum is NGVD29, for this study msl = NGVD29

Table 2.5.4-8 Summary of Undisturbed Samples of the Blue Bluff Marl (ESP)

Boring Number	Sample Number	Depth at Top of Sample (ft)	Length of Sample (in.)
B-1002	UD-1	92.0	30
B-1002	UD-2	103.5	30
B-1002	UD-3	113.5	30
B-1002	UD-4	123.5	30
B-1002	UD-5	133.4	30
B-1003	UD-1	92.0	30
B-1004	UD-1	144.0	18
B-1004	UD-2	148.5	18
B-1004	UD-3	163.5	30
B-1004	UD-4	177.0	30
B-1004	UD-5	188.5	30
B-1004	UD-6	198.5	30

Table 2.5.4-9 Summary of SPT Hammer Energy Transfer Efficiency from ESP Investigation

Borehole and Sample Number	Energy Transfer Efficiency (%)
B1013-SS5	65
B1013-SS8	70
B1013-SS10	68
B1013-SS13	71
B1013-SS14	72
B1013-SS15	73
B1008-SS26	79
B1008-SS27	75
B1008-SS28	75
B1006-SS7	71
B1006-SS8	74
B1006-SS10	77
B1006-SS15	85
B1006-SS16	86
B1006-SS17	87
B1006-SS26	83
B1006-SS27	80
B1006-SS28	82
Range:	65-87
Average:	76
Median:	75

Table 2.5.4-9a Summary of SPT Hammer Energy Transfer Efficiency from COL Investigation

Hammer Serial No.	Rig Type	Number of Measurements	Min. ETR (%)	Max. ETR (%)	Avg. ETR (%)	Hammer Correction (Ce)
100	Diedrich D-50 ATV	6	69.1	75.1	72.4	1.21
165952	CME 850 ATV	7	78.9	90.0	83.4	1.39
200587	CME 75 Truck	5	83.7	86.6	84.2	1.40
211797	CME 75 Truck	3	75.1	80.3	77.6	1.29
219505	CME 55 Truck	3	67.1	80.6	70.1	1.17
219907	CME 75 Truck	3	76.6	84.6	80.2	1.34
270256	CME 85 Truck	5	77.7	88.0	82.5	1.38
311025	CME 55 Truck	4	88.3	92.6	90.2	1.50
328848	CME 750 ATV	3	83.1	85.1	84.0	1.40
331145	CME 55LC Truck	5	85.7	90.0	88.4	1.47
337153	CME 550 ATV	4	76.0	87.7	82.0	1.37
XO2958	CME 850 ATV	3	78.0	79.4	78.9	1.32

Table 2.5.4-10 Estimated Shear Wave Velocity and Dynamic Shear Modulus Values for the Compacted Backfill (ESP)

Depth (ft)	Vs(1) (fps)	Gmax(2) (ksf)
0 to 6	573	1,255
6 to 10	732	2,049
10 to 14	811	2,510
14 to 18	871	2,898
18 to 23	927	3,280
23 to 29	983	3,694
29 to 36	1040	4,130
36 to 43	1092	4,553
43 to 50	1137	4,940
50 to 56	1175	5,274
56 to 63	1209	5,588
63 to 71	1232	5,796
71 to 79	1253	6,001
79 to 86	1273	6,186

(1) From Figure 6-1 of **Bechtel (1984)**.

(2) Gmax were calculated using γ from Table 2.5.4-1.

Table 2.5.4-10a Shear Wave Velocity Values for the Compacted Backfill (COL)

Depth (ft)	Vs (fps)
0	550
5	724
10	832
20	975
30	1064
40	1130
50	1183
60	1228
70	1267
80	1302
85	1318
86.5	1327
88	1327

Table 2.5.4-11 Shear Wave Velocity Values for Site Amplification Analysis Part A: Soil Shear-Wave Velocities (ESP)

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

Table 2.5.4-11 Shear Wave Velocity Values for Site Amplification Analysis Part B: Rock Shear-Wave Velocities - Six Alternate Profiles

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

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

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

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

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

Table 2.5.4-11a Shear Wave Velocity Values for Site Amplification Analysis Part A: Soil Shear-Wave Velocities (COL Soil Column)

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

Table 2.5.4-12 Summary of Modulus Reduction and Damping Ratio Values – EPRI-Base

Shear Strain (%)	0-20 ft (Compacted Backfill)		20-50 ft (Compacted Backfill)		50-86 ft (Compacted Backfill)		86-149 ft (Blue Bluff Marl)		149-215.7 ft (Lower Sand Stratum-Still Branch Formation)		Between 215.7 and 500 ft (Lower Sand Stratum below Still Branch)		Soil between 500 ft and top of rock (about 1,000 ft) (Deep Sands)	
	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio
0.0001	1	1.4	1	1.2	1	1	1	1.4	1	0.8	1	0.7	1	0.6
0.00032	1	1.5	1	1.2	1	1	1	1.4	1	0.9	1	0.8	1	0.6
0.001	0.98	1.8	0.99	1.4	1	1.2	0.99	1.5	1	1	1	0.8	1	0.6
0.00316	0.914	2.8	0.946	2.1	0.97	1.64	0.96	2	0.98	1.33	0.988	1.12	0.99	0.81
0.01	0.75	5	0.82	3.6	0.87	2.8	0.84	2.9	0.9	2.2	0.93	1.8	0.95	1.2
0.03162	0.509	9.3	0.608	7	0.68	5.49	0.63	6	0.74	4.36	0.791	3.53	0.852	2.5
0.1	0.27	15.3	0.36	12.4	0.43	10.2	0.36	11.4	0.5	8.6	0.57	7.1	0.65	5.3
0.3162	0.116	21.9	0.165	19.1	0.22	16.5	0.16	17	0.27	14.61	0.321	12.78	0.41	10.27
1	0.04	27	0.06	24.9	0.09	22.9	0.06	19.4	0.12	21.2	0.15	19.3	0.2	16.7

Table 2.5.4-12a Summary of Modulus Reduction and Damping Ratio Values - Site Specific

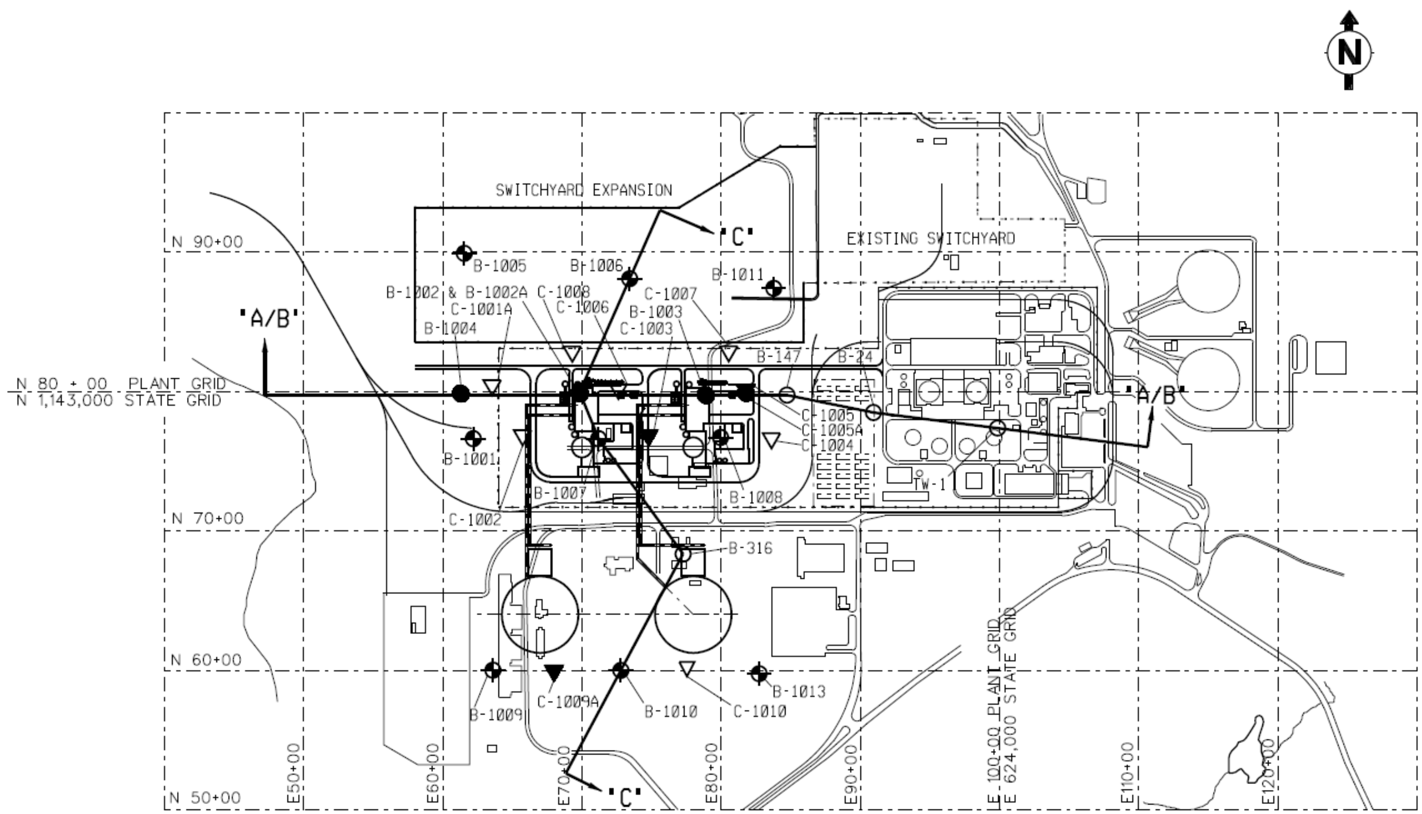
Stratum	Backfill				Blue Bluff Marl				Lower Sands			
Sub strata	<25ft		≥25ft		Low PI		High PI		Sands		Clay (Congaree/ Snapp)	
Shear Strain (%)	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio	G/Gmax	Damping Ratio
0.00010	1	0.97	1	0.62	1	1.44	1	1	1	0.62	1	0.86
0.00032	1	1.05	1	0.62	1	1.56	1	1.05	1	0.62	1	0.87
0.00100	0.998	1.05	1	0.7	1	1.67	1	1.32	1	0.7	1	0.93
0.00359	0.942	1.44	0.975	0.89	0.96	2.34	0.9965	1.71	0.997	0.89	0.99	1.21
0.01019	0.826	2.26	0.902	1.3	0.867	3.23	0.97	2.3	0.954	1.32	0.928	1.8
0.03170	0.603	4.55	0.748	2.6	0.673	5.75	0.88	3.97	0.858	2.6	0.8	3.62
0.10000	0.355	8.97	0.495	5.64	0.395	10.63	0.679	6.715	0.649	5.59	0.56	7.54
0.30690	0.172	14.94	0.269	10.65	0.187	16.39	0.433	11.115	0.411	10.65	0.327	13
0.65313	0.089	19.38	0.158	14.73	0.1	19.08	0.2785	14.545	0.263	14.68	0.198	17.42
1.00000	0.072	22.12	0.117	17.11	0.068	19.12	0.217	15.77	0.209	17.11	0.154	19.87

Table 2.5.4-13 Summary of Modulus Reduction and Damping Ratio Values – SRS-Based

Cyclic Shear Strain (%)	Blue Bluff Marl		Shallow Sand (<300 ft)		Deep Sand (>300 ft)	
	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio	G/G_{max}	Damping Ratio
0.0001	1	0.8	1	0.6	1	0.5
0.0002	1	0.8	1	0.6	1	0.5
0.0003	1	0.8	1	0.7	1	0.5
0.0005	1	0.8	1	0.7	1	0.5
0.001	0.99	0.9	0.99	0.8	0.995	0.6
0.002	0.98	1.1	0.98	1	0.99	0.7
0.003	0.965	1.2	0.96	1.1	0.985	0.8
0.005	0.94	1.5	0.93	1.4	0.96	0.9
0.01	0.89	2.1	0.87	2.2	0.92	1.4
0.02	0.8	3.3	0.77	3.5	0.85	2.2
0.03	0.72	4.3	0.69	4.7	0.78	3
0.05	0.61	6.1	0.57	6.7	0.69	4.5
0.1	0.43	9.6	0.4	10.4	0.53	7.3
0.2	0.28	13.1	0.25	14.8	0.36	11.2
0.3	0.205		0.18		0.27	13.8
0.5	0.13	19	0.12	21	0.18	
0.7	0.1		0.09		0.14	
1	0.08		0.07	27	0.1	23

Table 2.5.4-14 Acceptable Gradation Envelope for Compacted Backfill

Sieve No.	Opening Size (mm)	Percent Passing	
		Minimum	Maximum
4	4.75	97	100
10	2.00	95	100
20	0.85	85	98
40	0.425	50	95
60	0.25	17	80
100	0.15	7	50
140	0.106	4	28
200	0.075	3	25



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 2.5.4-1 ESP Study Boring Location Plan

Figure 2.5.4-1a COL Site Boring Location Plan

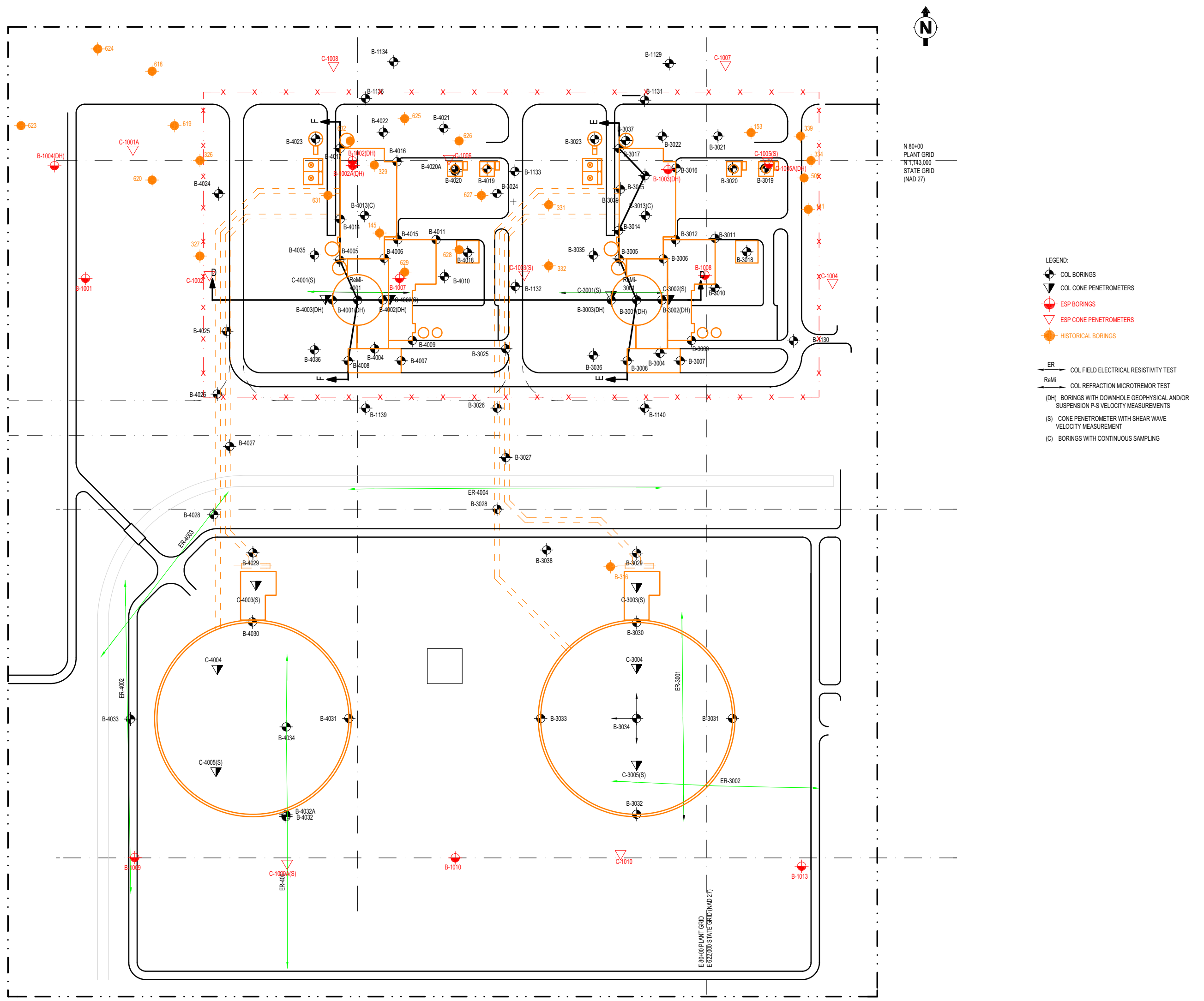
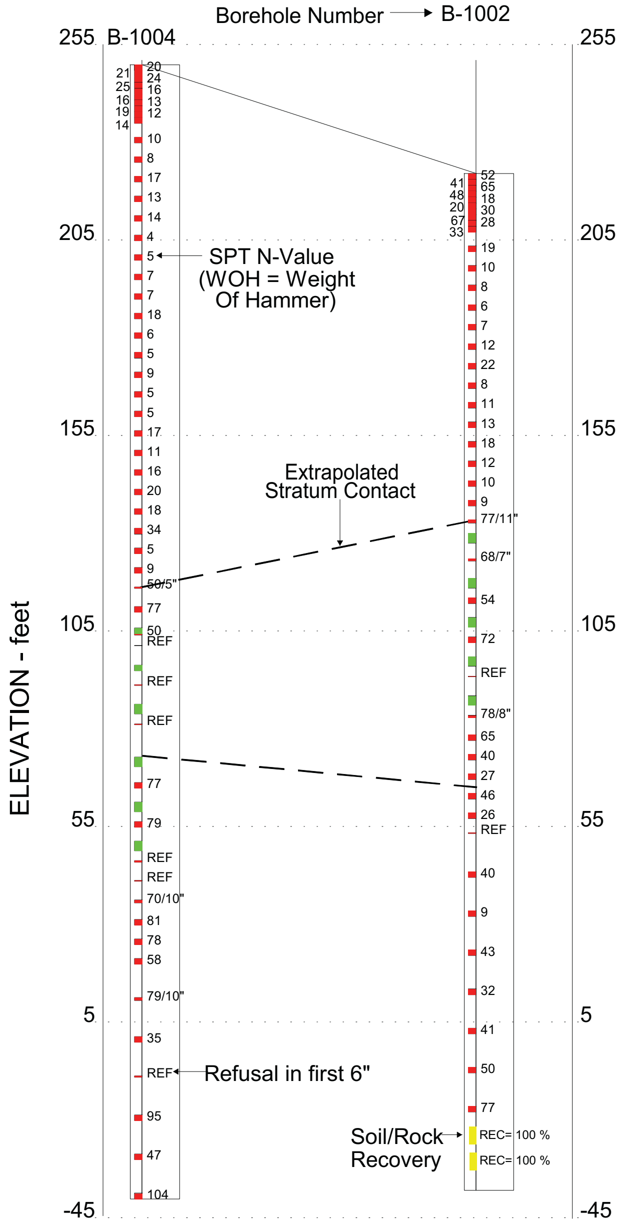


Figure 2.5.4-1b COL Power Block — Cooling Tower Boring Location



Stratum Descriptions Continued

GAILLARD/BLACK CREEK FM. Interbedded SAND, SILT and CLAY

PIO NONO/UNNAMED FM. SAND with silt

CAPE FEAR FM. Silty, clayey SAND with silt and clay layers

TRIASSIC BASIN (BEDROCK). Interbedded CONGLOMERATE, BRECCIA, SANDSTONE and MUDSTONE.

Notes

Subsurface data have been obtained only at the actual boring locations. The stratification shown by the dashed lines between borings is based on extrapolation of the data from the borings. Actual stratification between the borings may differ from that shown.

Stratum Descriptions

BARNWELL GROUP. Silty, clayey SAND with layers of silt and clay. Lower limestone / shell hash (UTLEY LMST FM.)

LISBON FM (BLUE BLUFF MEMBER). Marl with limestone layers

STILL BRANCH FM. Silty, clayey SAND

CONGAREE FM. SAND with silt and clay

SNAPP FM. Interbedded SAND, SILT and CLAY

BLACK MINGO FM. Interbedded SAND, SILT and CLAY

STEEL CREEK FM. SAND with silt and clay

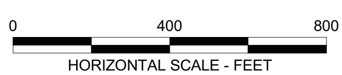


Figure 2.5.4-2 Subsurface Profile Legend

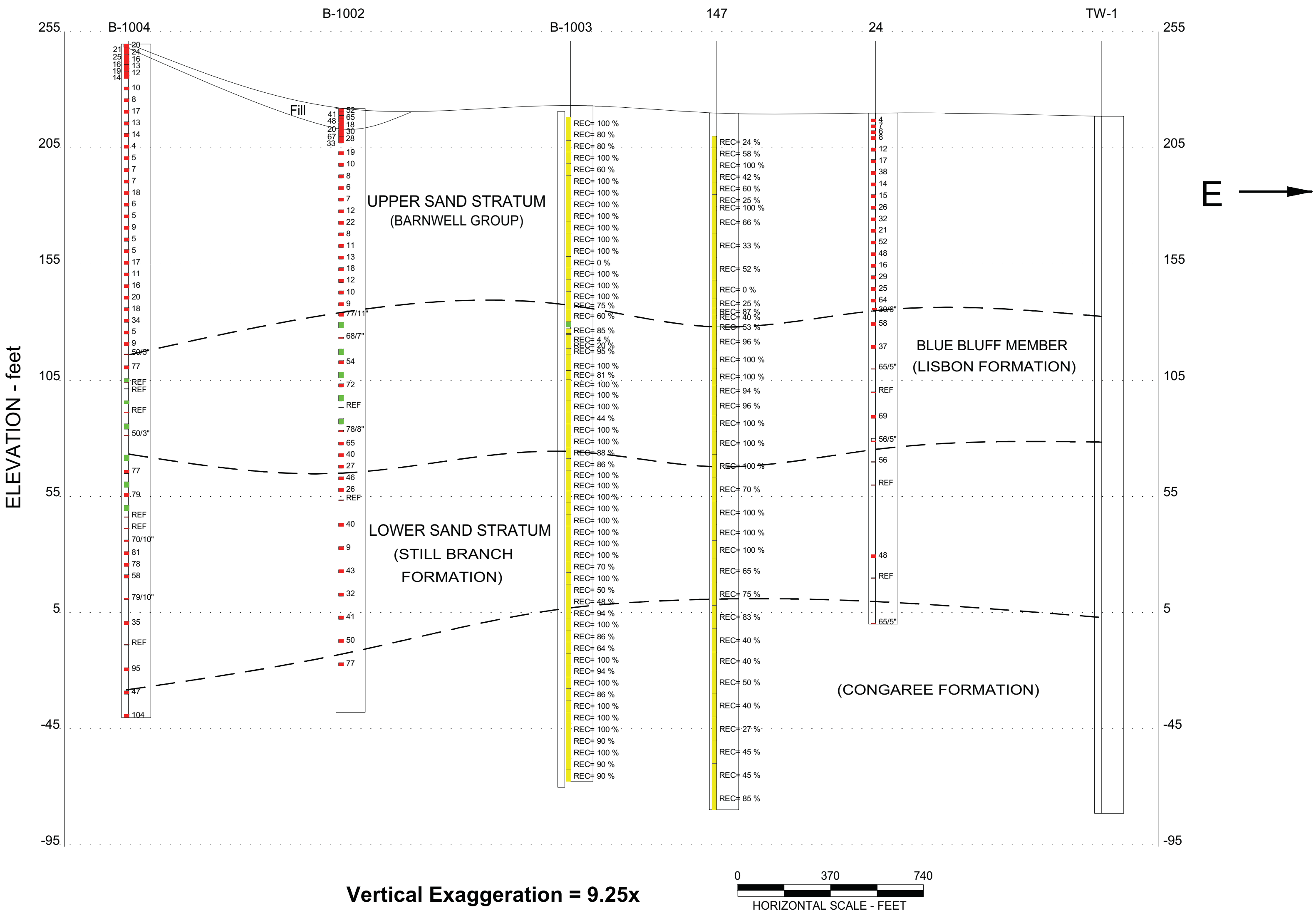


Figure 2.5.4-3 Subsurface Profile A–A'

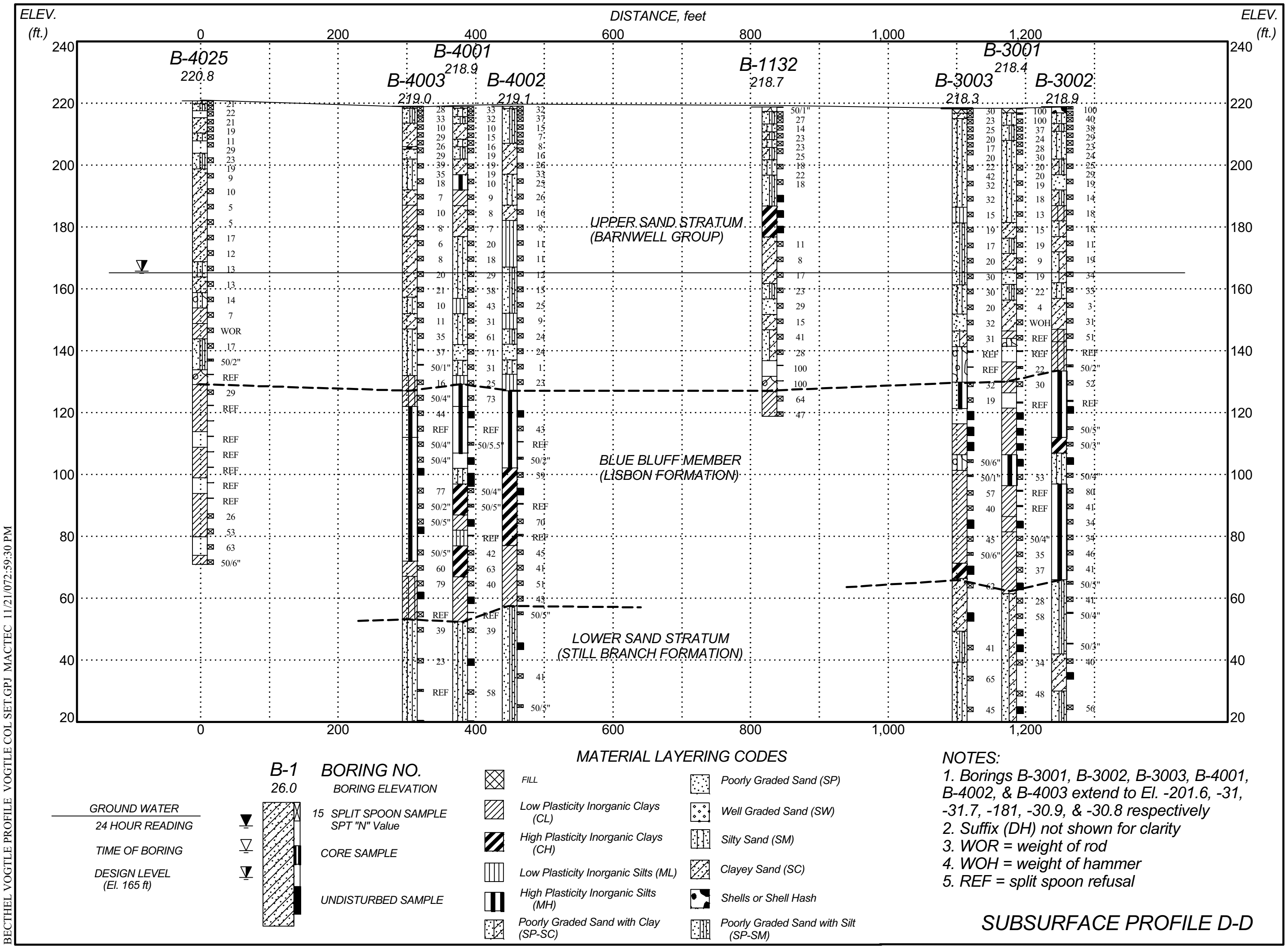


Figure 2.5.4-3a Subsurface Profile D–D’

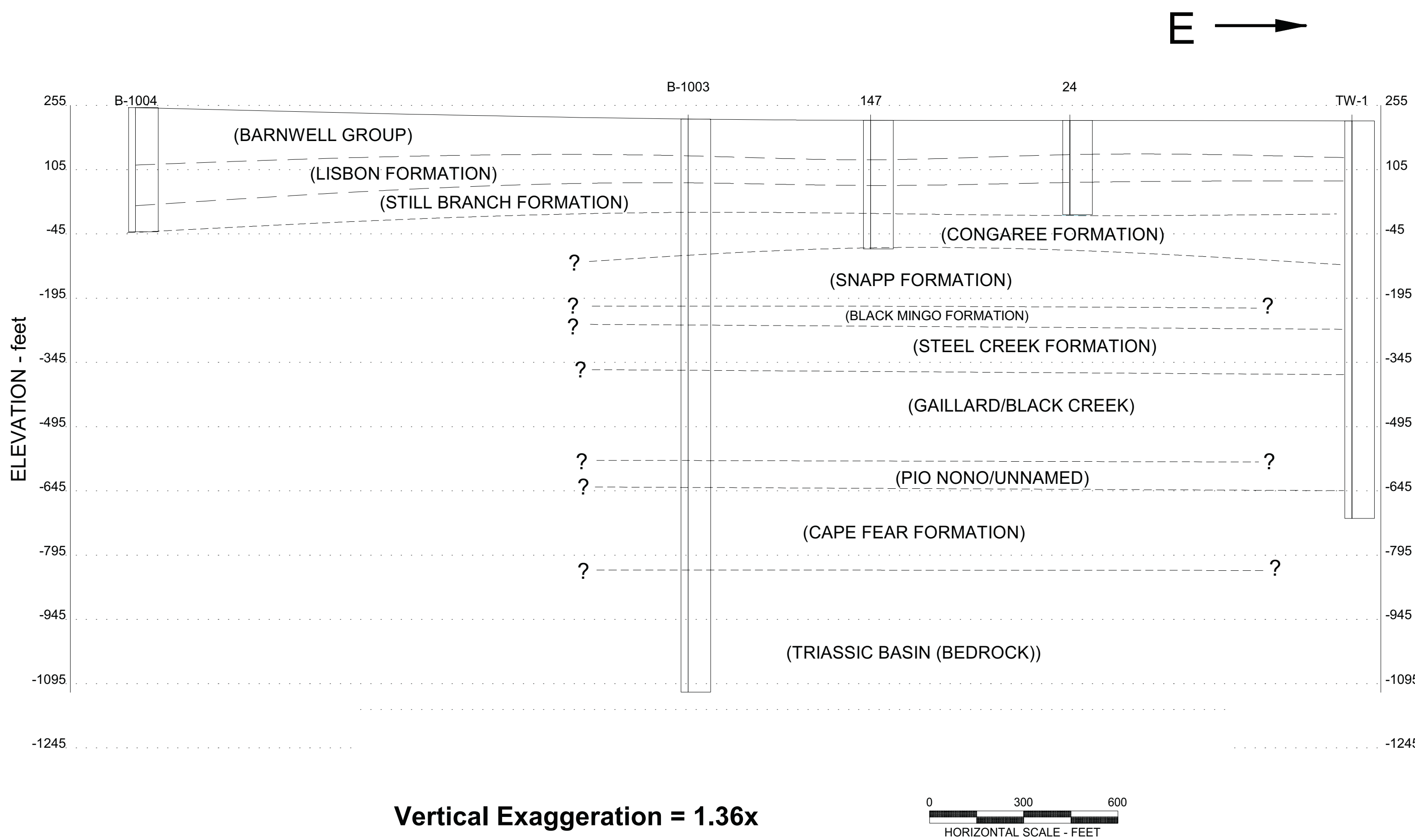


Figure 2.5.4-4 Subsurface Profile B–B’

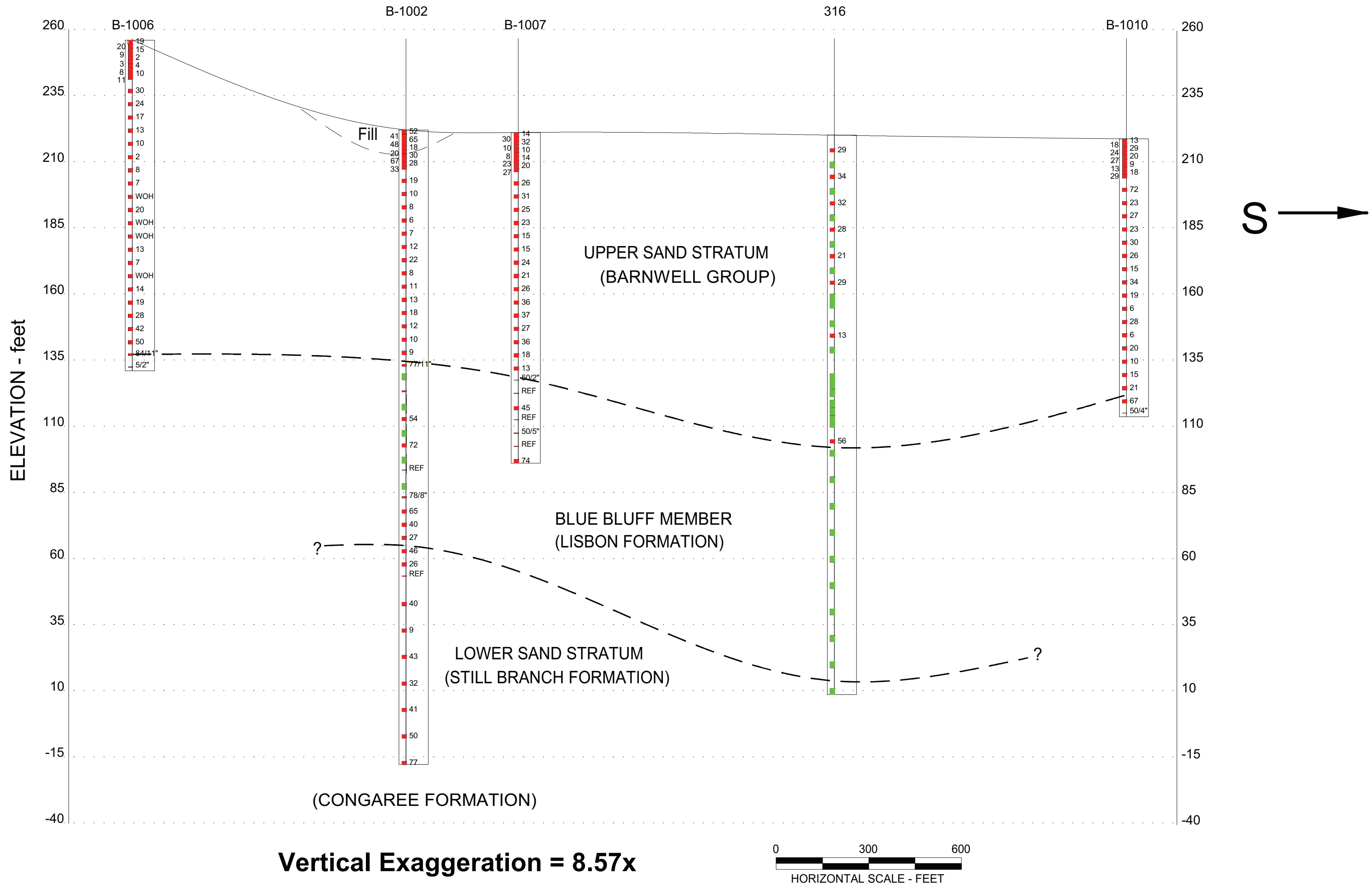


Figure 2.5.4-5 Subsurface Profile C–C'

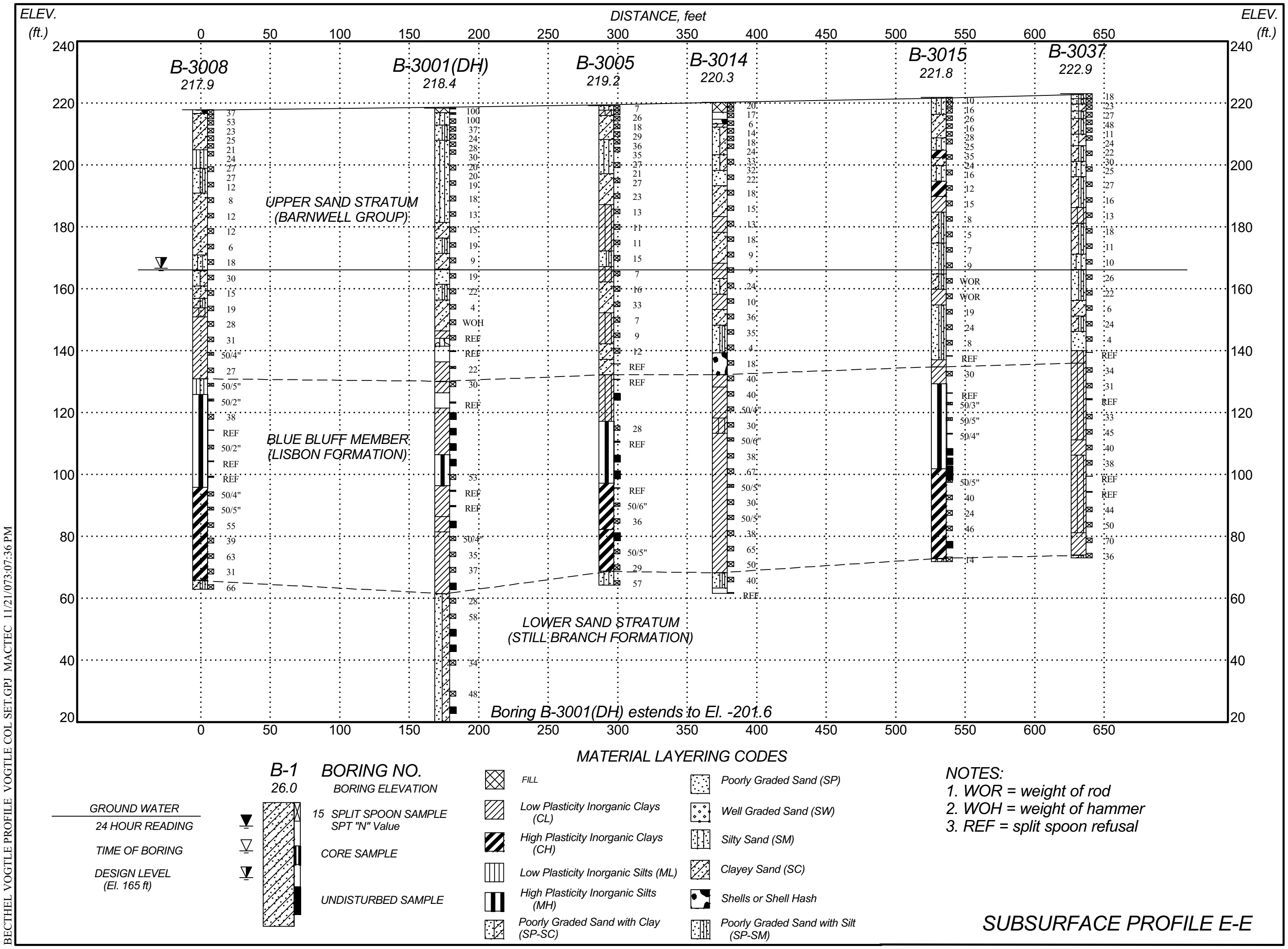


Figure 2.5.4-5a Subsurface Profile E–E'

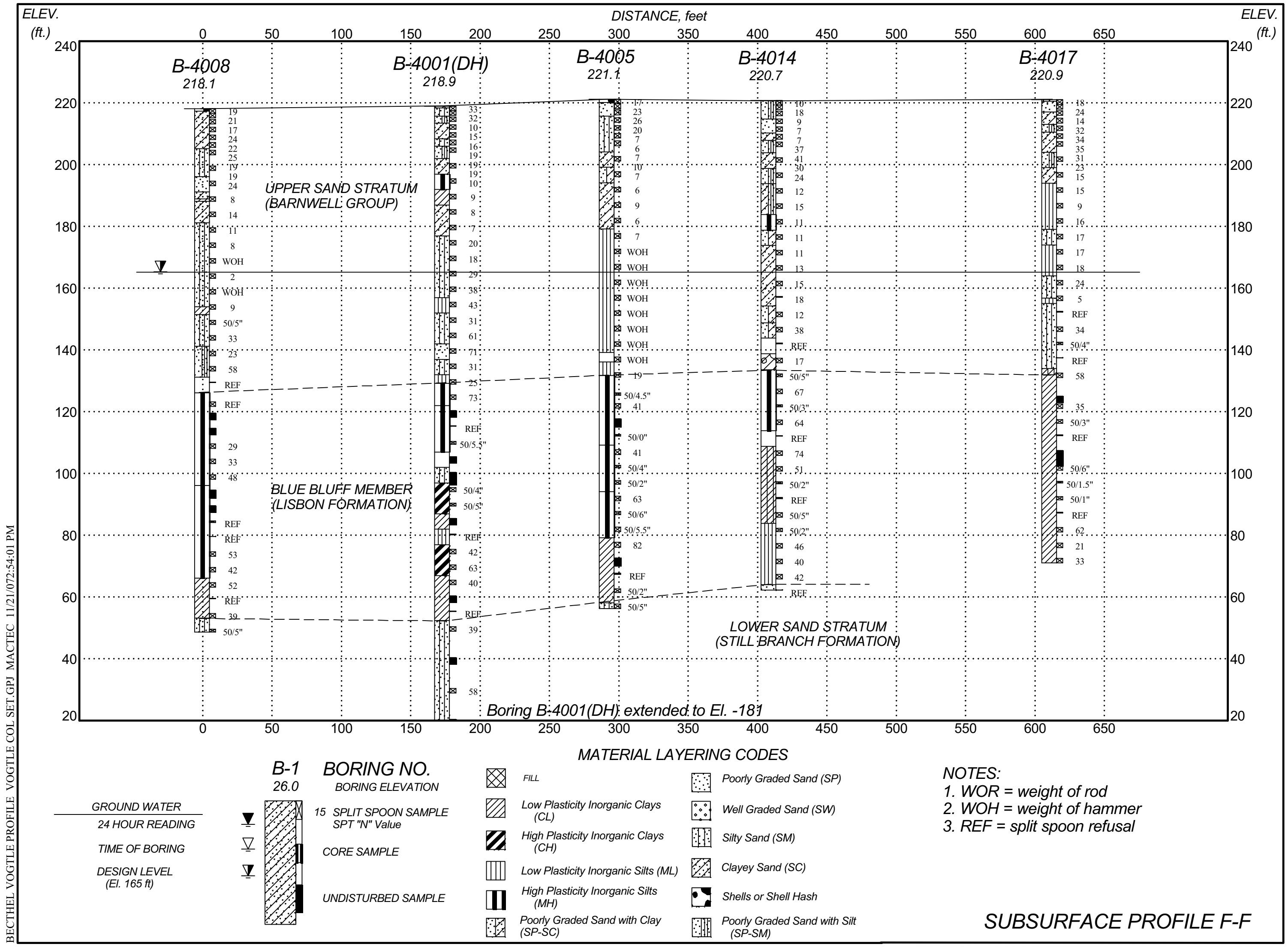


Figure 2.5.4-5b Subsurface Profile F–F’

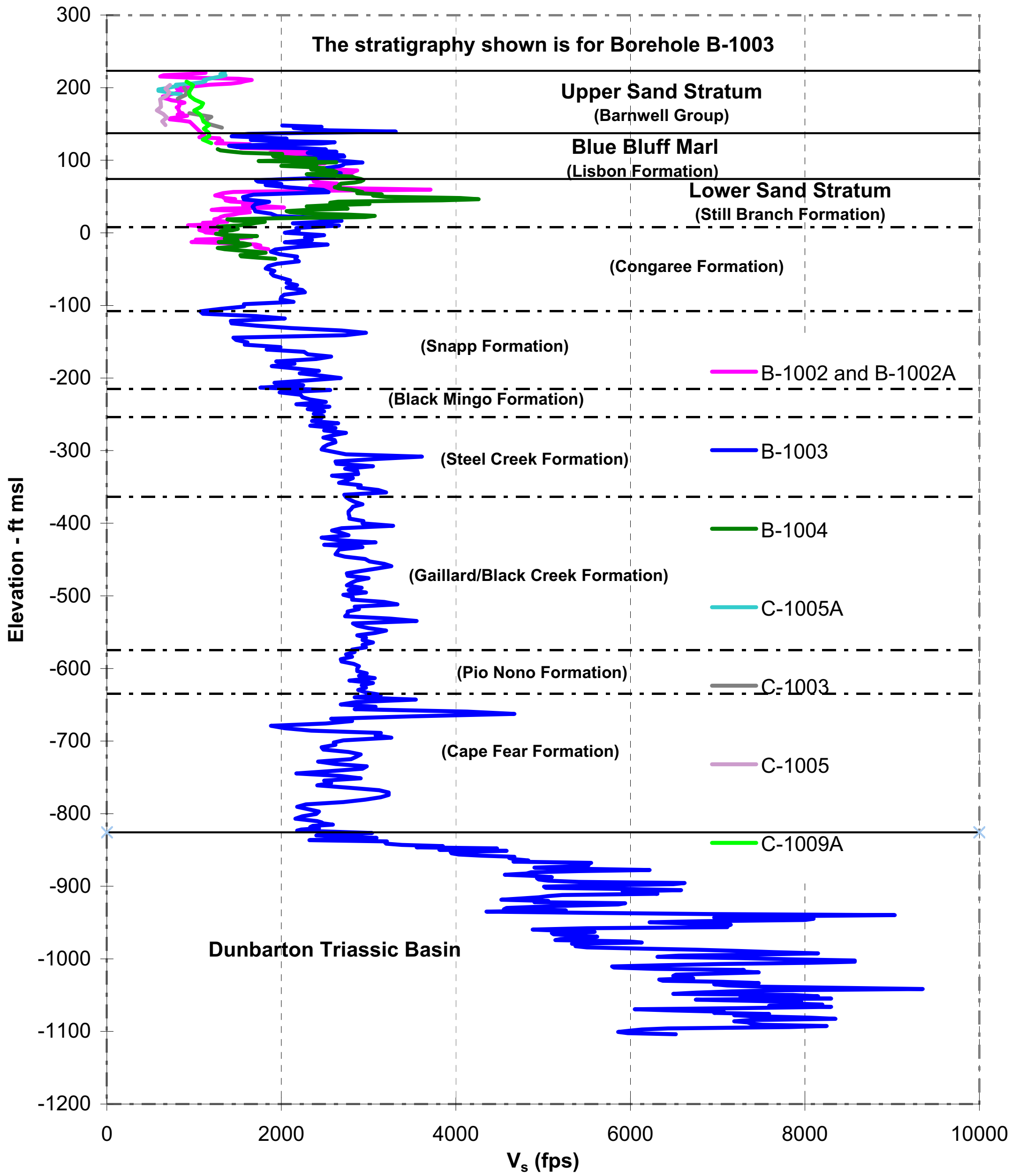


Figure 2.5.4-6 Shear Wave Velocity Measurements

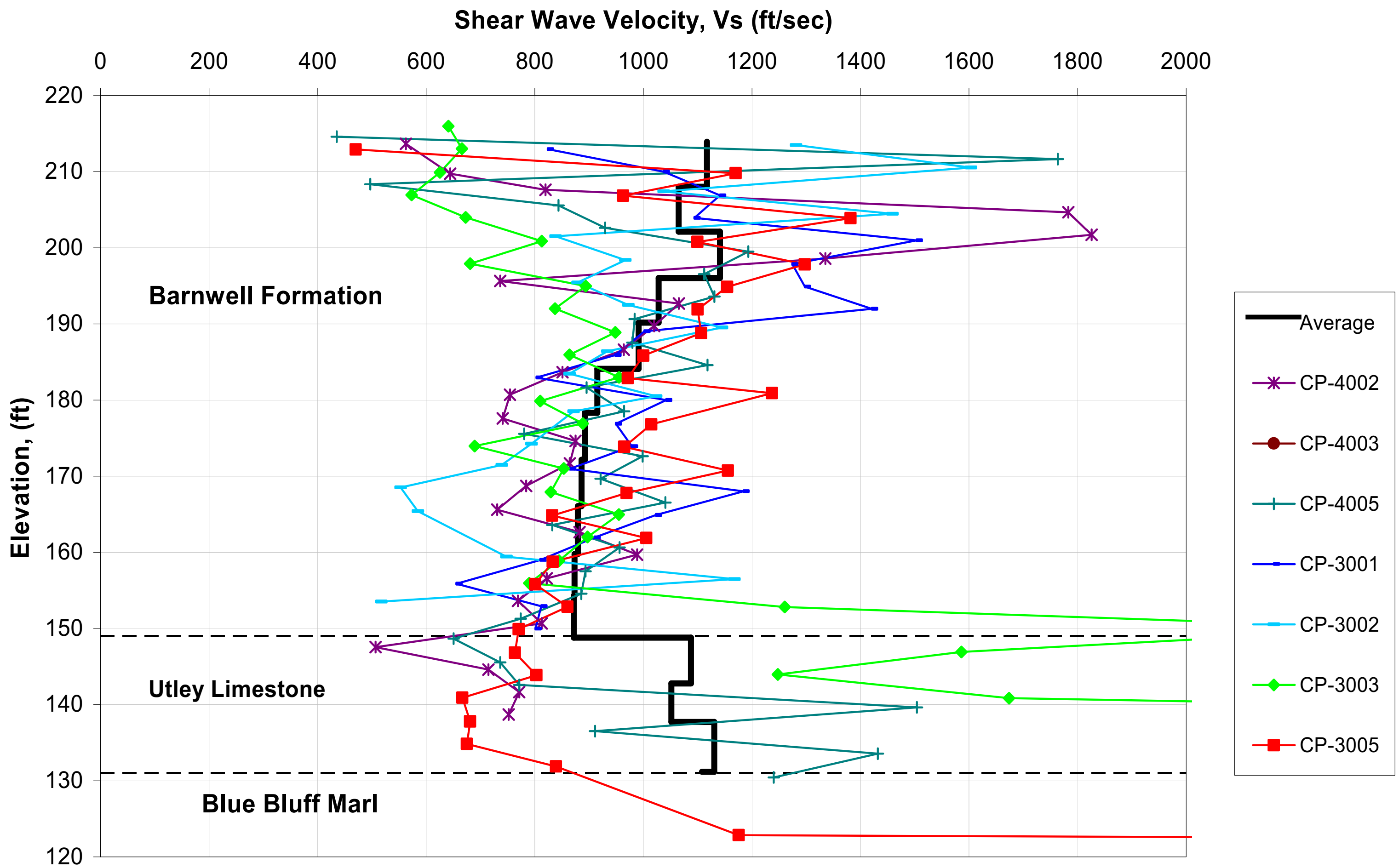


Figure 2.5.4-6a Shear Wave Velocity Measurements in the Upper Sand Stratum as Measured by COL SCPT

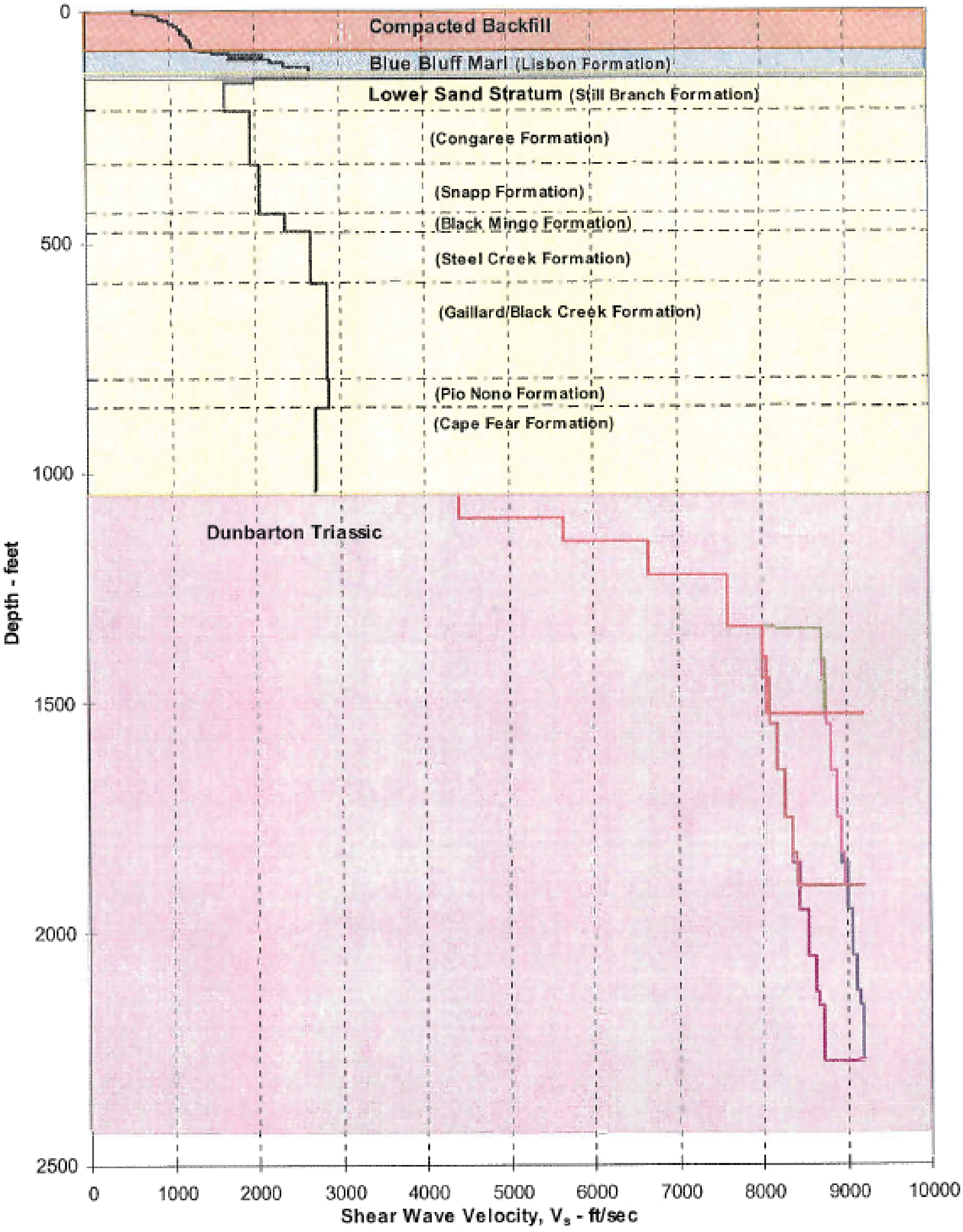


Figure 2.5.4-7 Shear Wave Velocity Profile for SHAKE Analysis

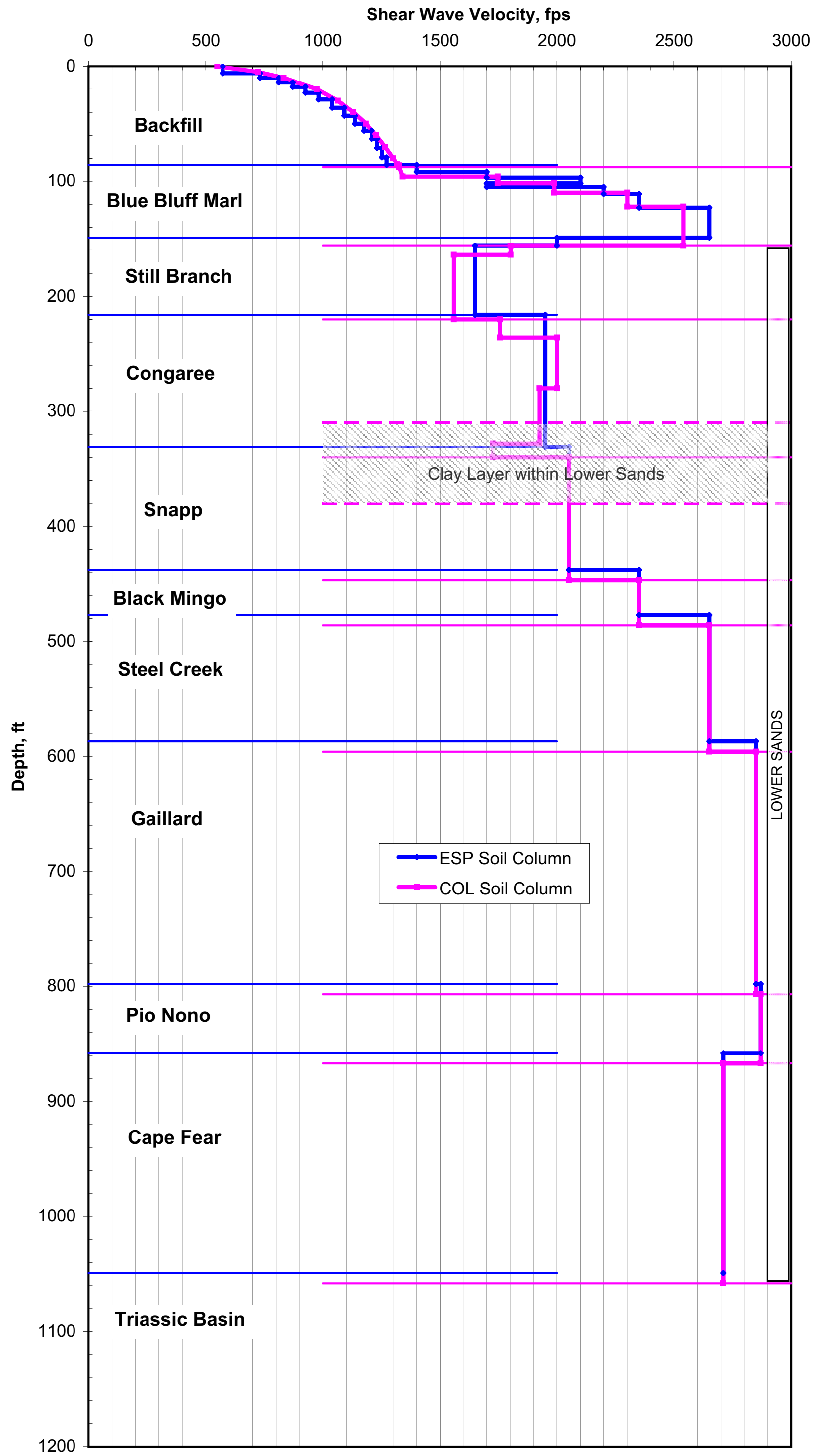


Figure 2.5.4-7a Shear Wave Velocity Profile — ESP and COL Soil Columns

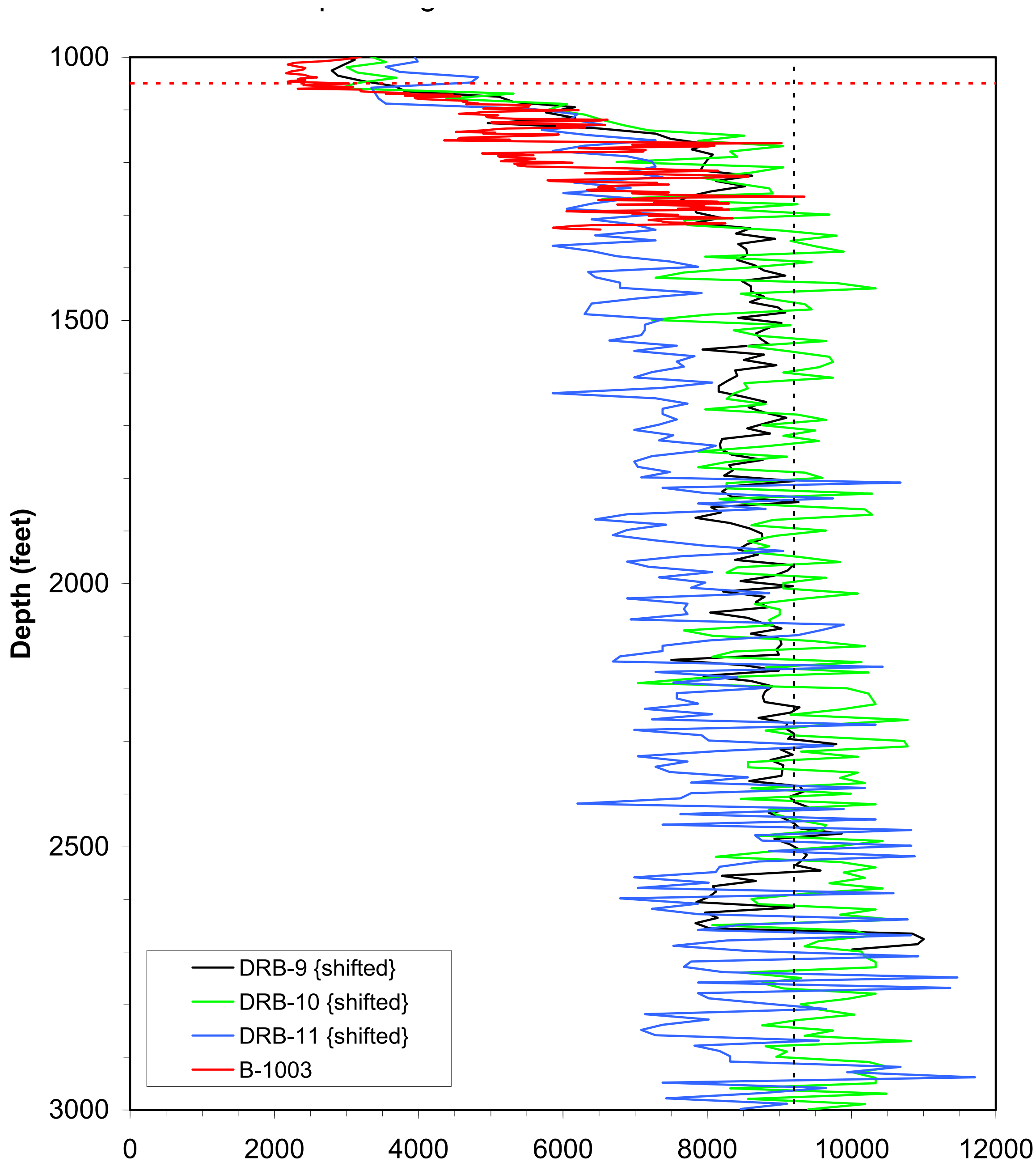


Figure 2.5.4-8 Rock shear-wave velocities for three SRS sites [DRB] (SRS 2005) and B-1003 [Figure 2.5.4-6]. The DRB data has been shifted in depth so that the depth to top of rock is consistent with B-1003.

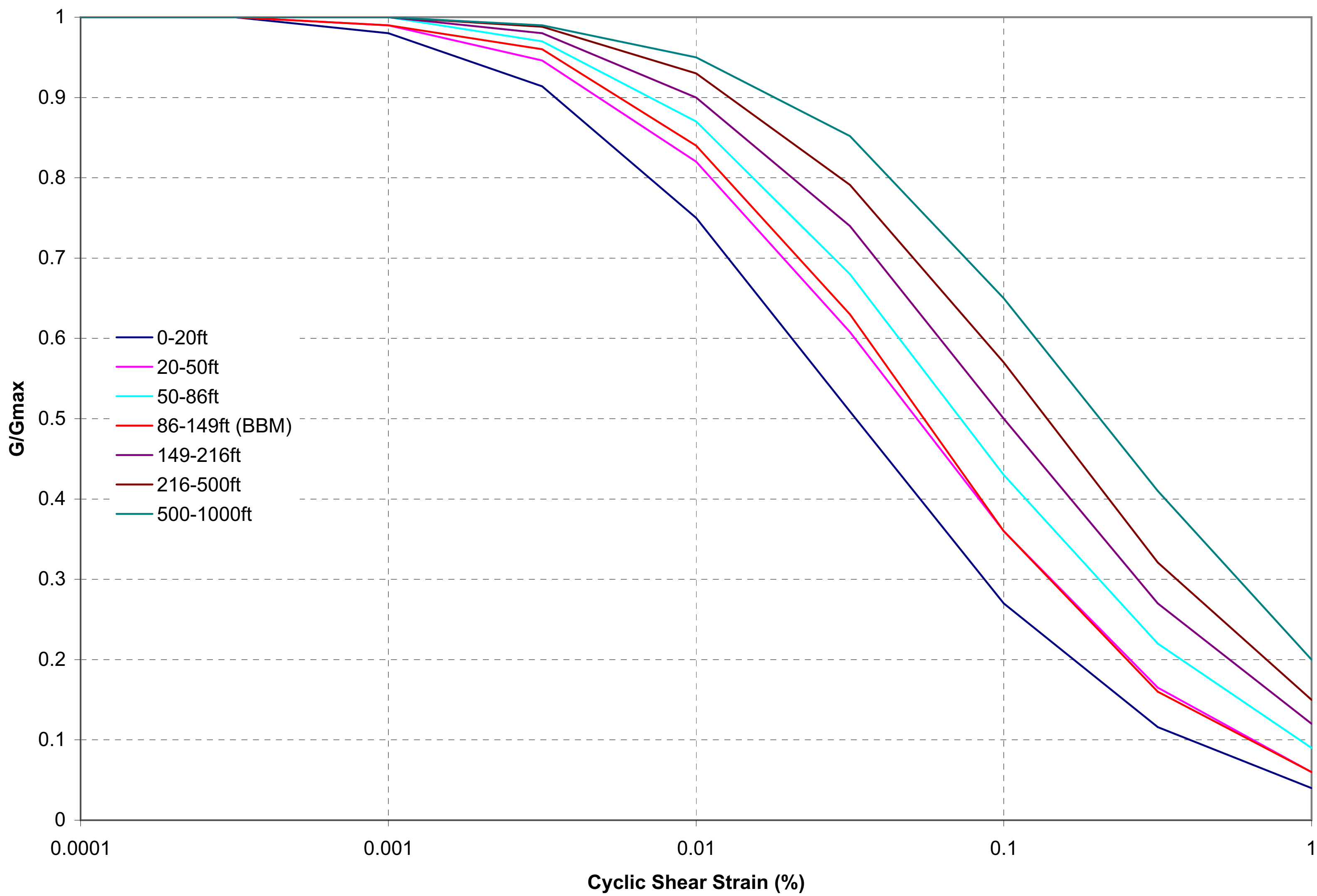


Figure 2.5.4-9 Shear Modulus Reduction Curves for SHAKE Analysis – EPRI Curves

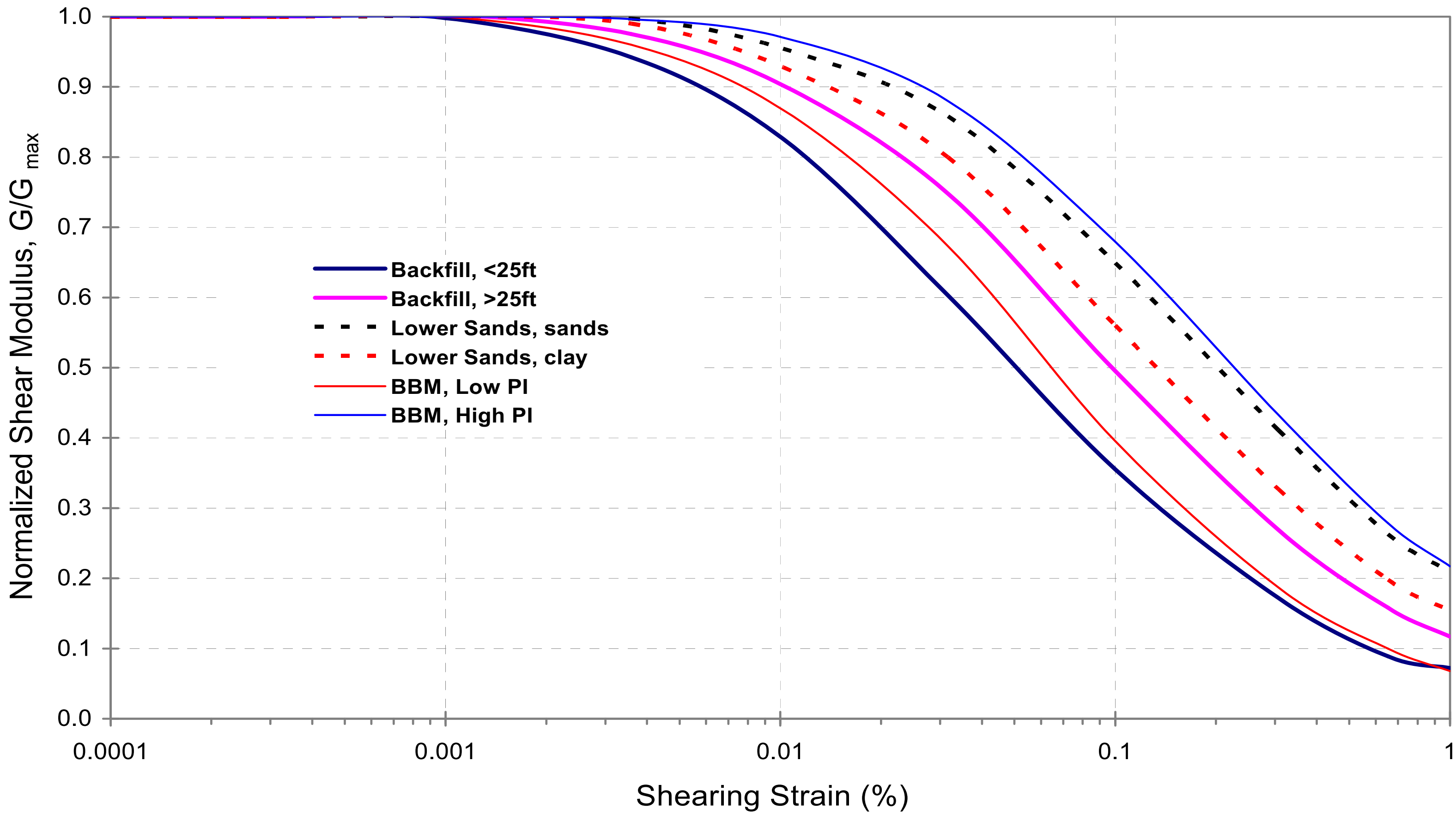


Figure 2.5.4-9a Site-Specific Shear Modulus Reduction Curves

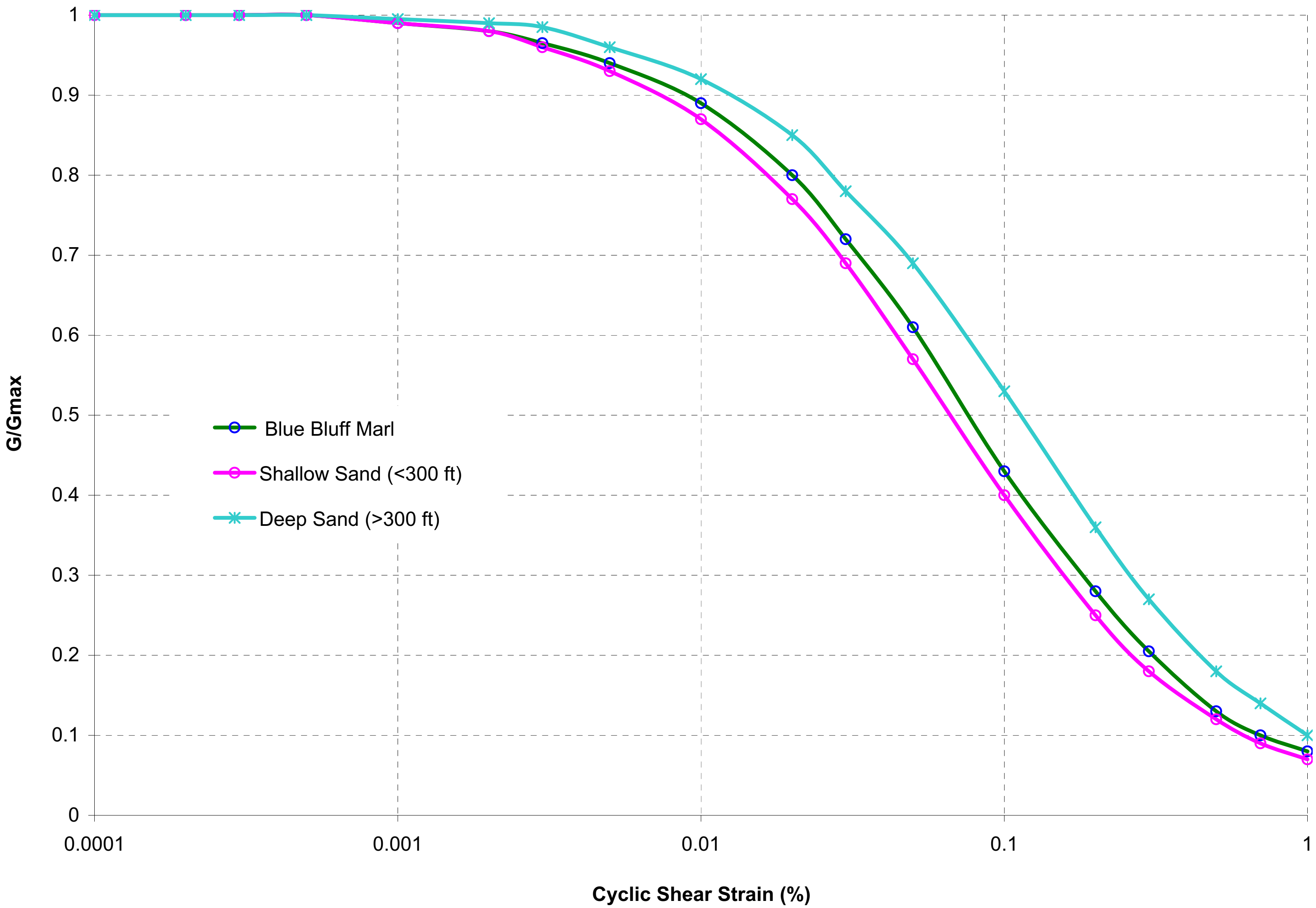


Figure 2.5.4-10 Shear Modulus Reduction Curves for SHAKE Analysis – SRS Curves

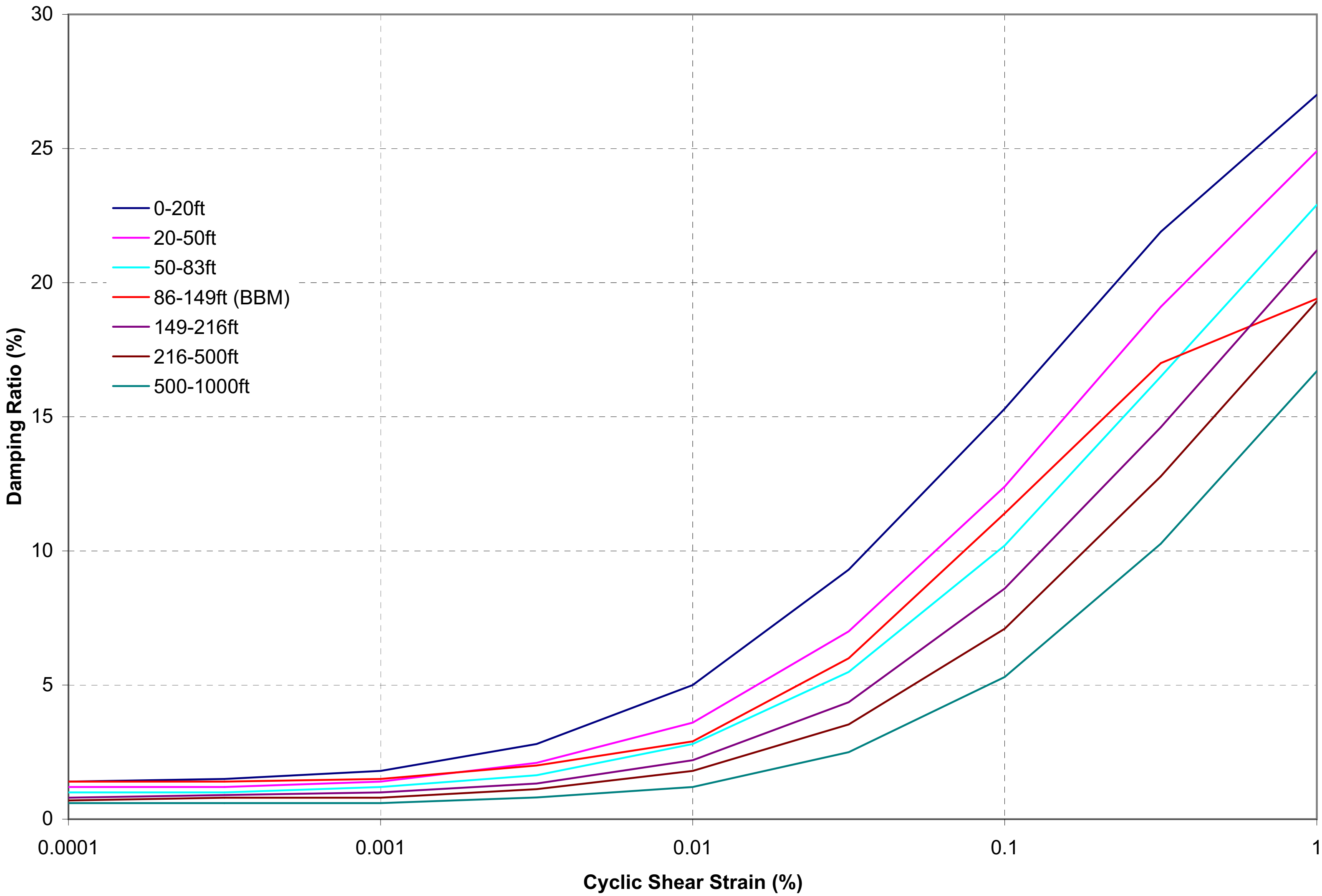


Figure 2.5.4-11 Damping Ratio Curves for SHAKE Analysis – EPRI Curves

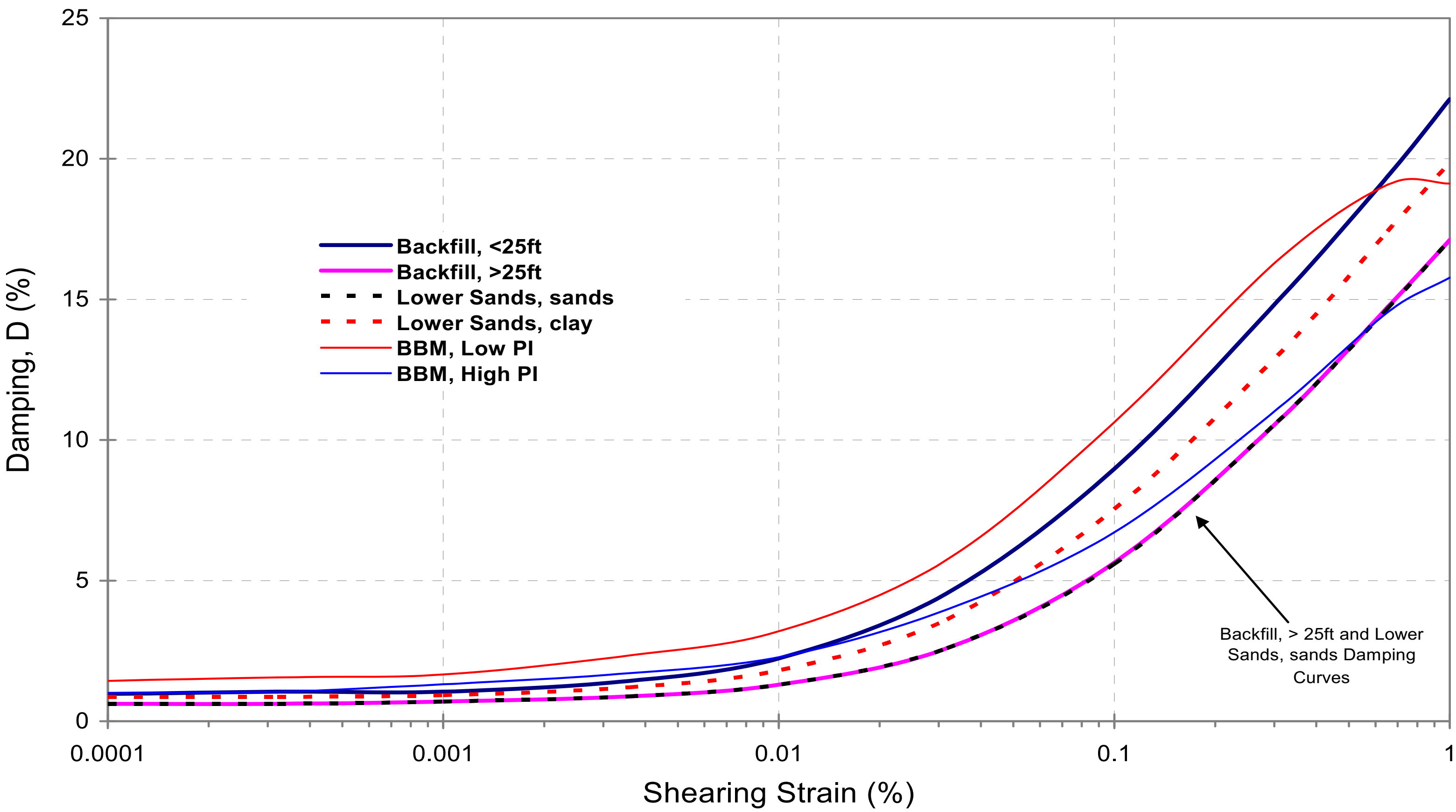


Figure 2.5.4-11a Site-Specific Damping Ratio Curves

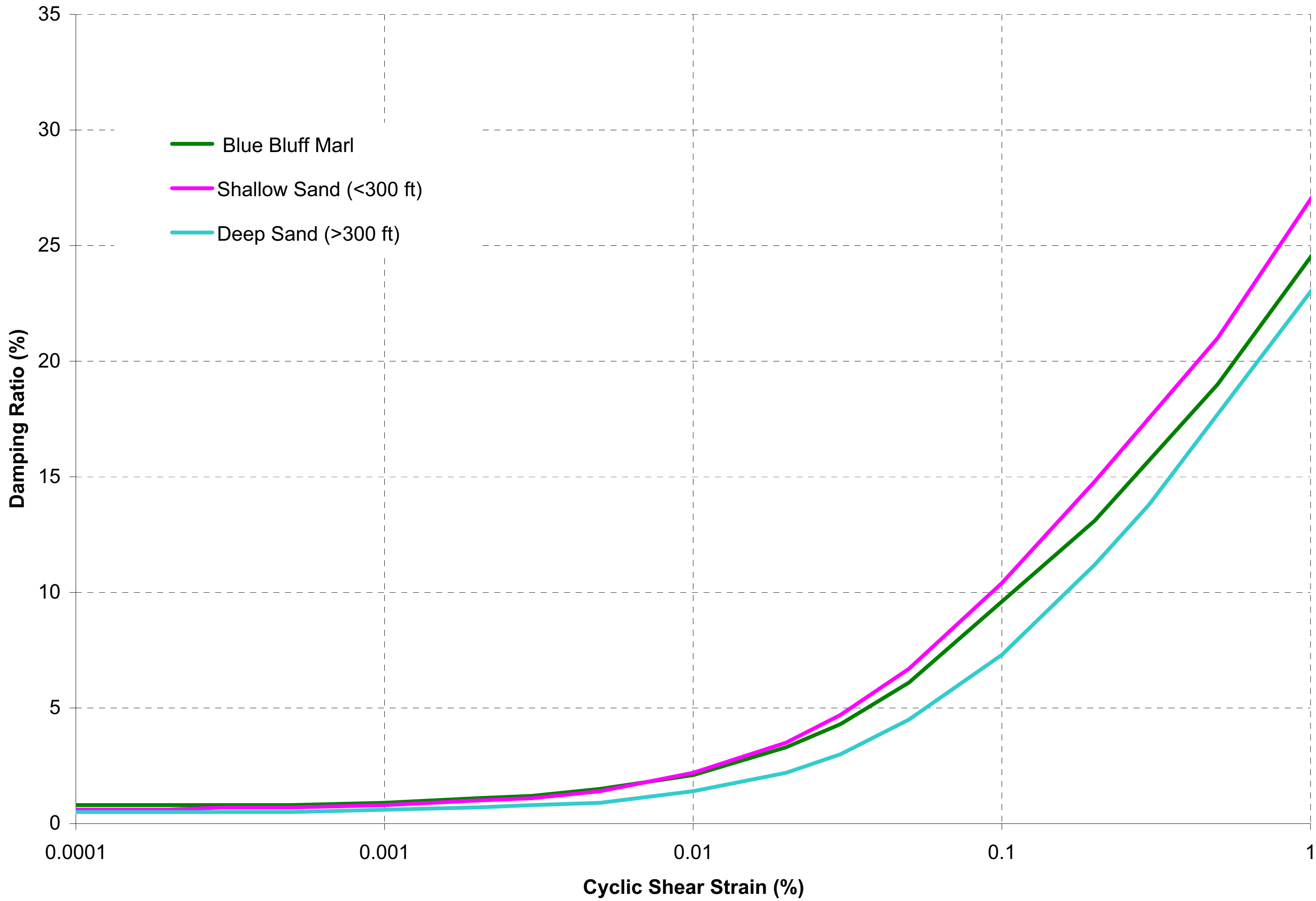


Figure 2.5.4-12   Damping Ratio Curves for SHAKE Analysis – SRS Curves

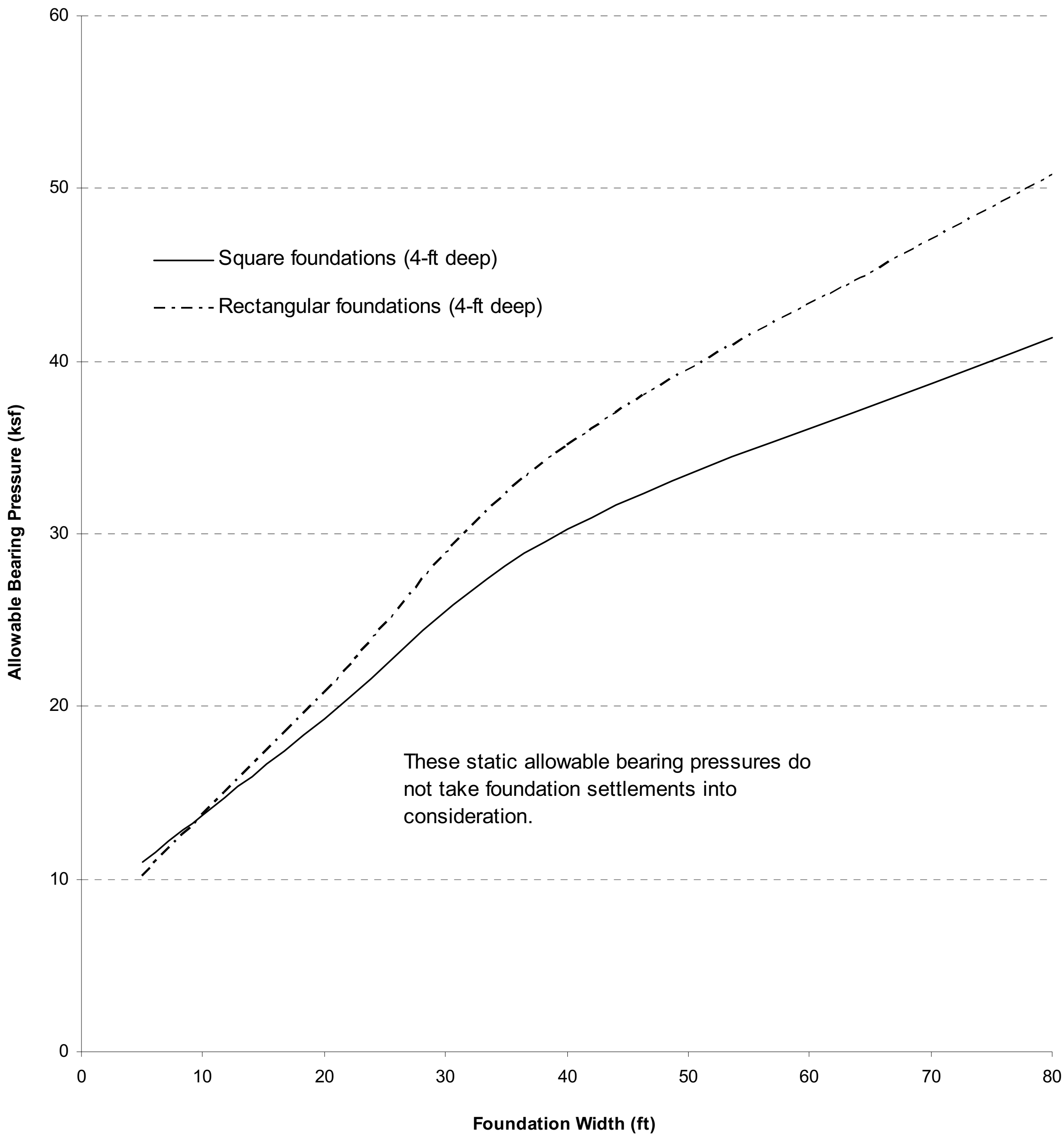


Figure 2.5.4-13 Allowable Bearing Capacity of Typical Foundation

Figure 2.5.4-14 Deleted

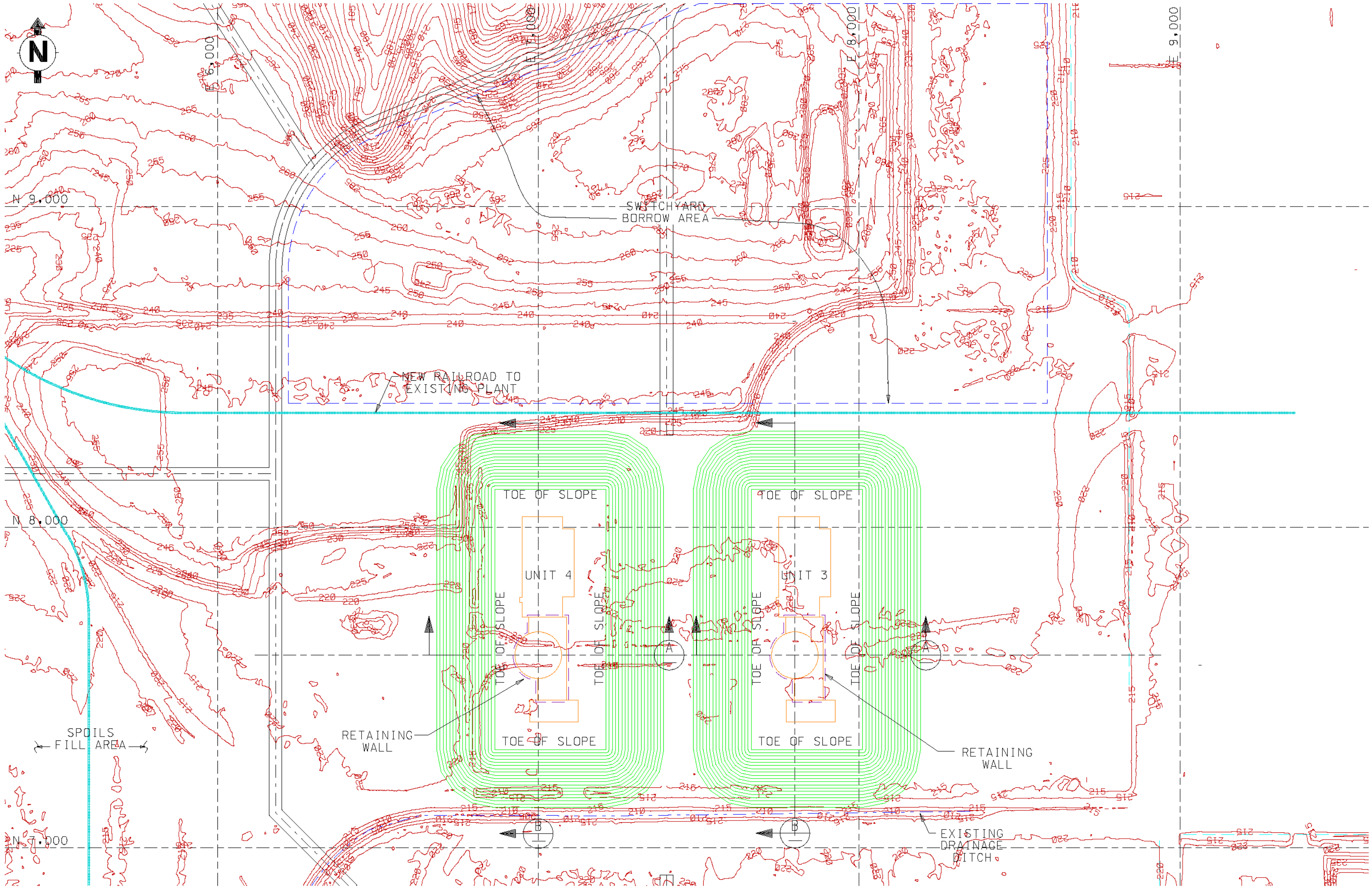


Figure 2.5.4-15 Power Block Excavation and Switchyard Borrow Area

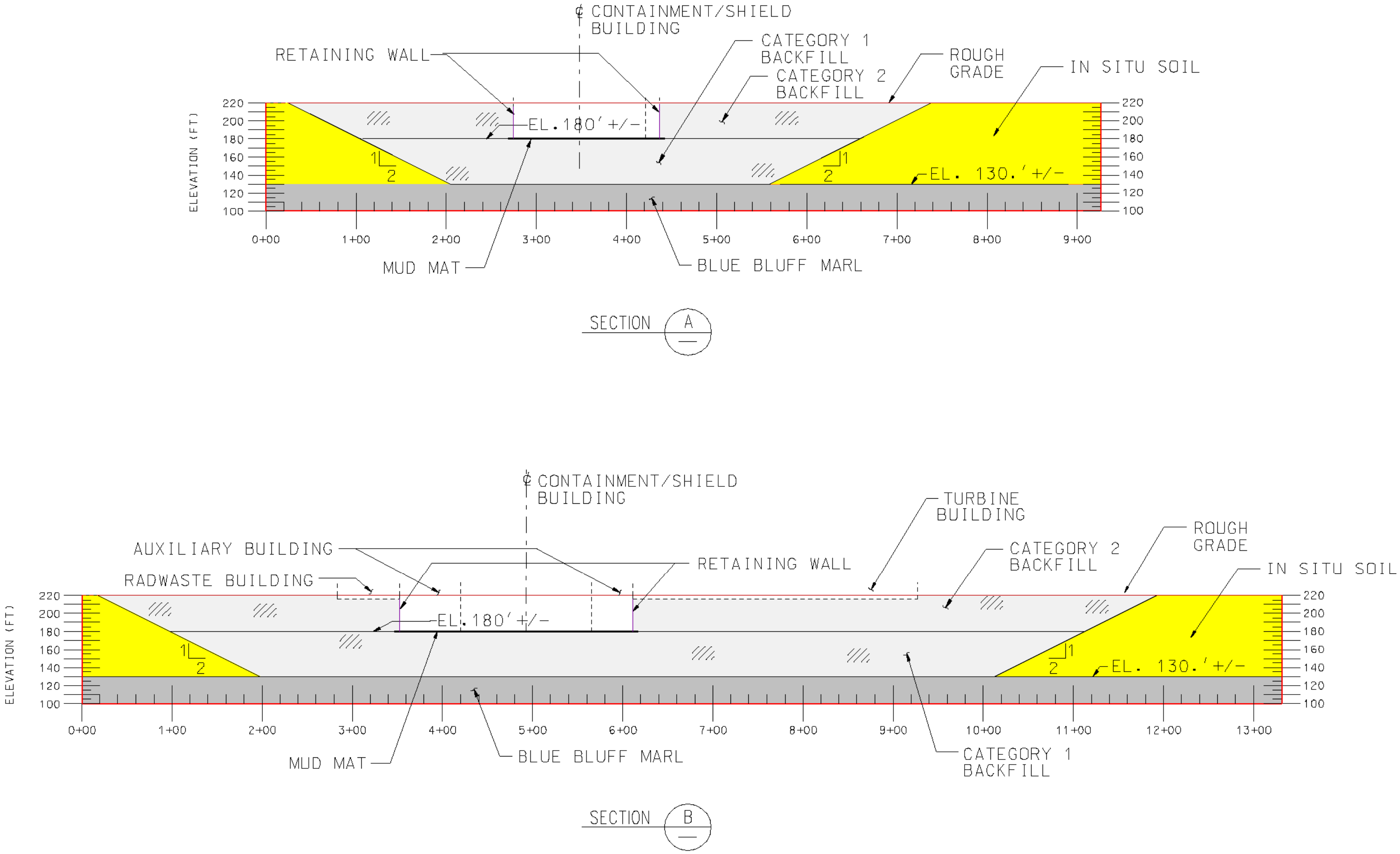


Figure 2.5.4-16 Power Block Excavation Sections

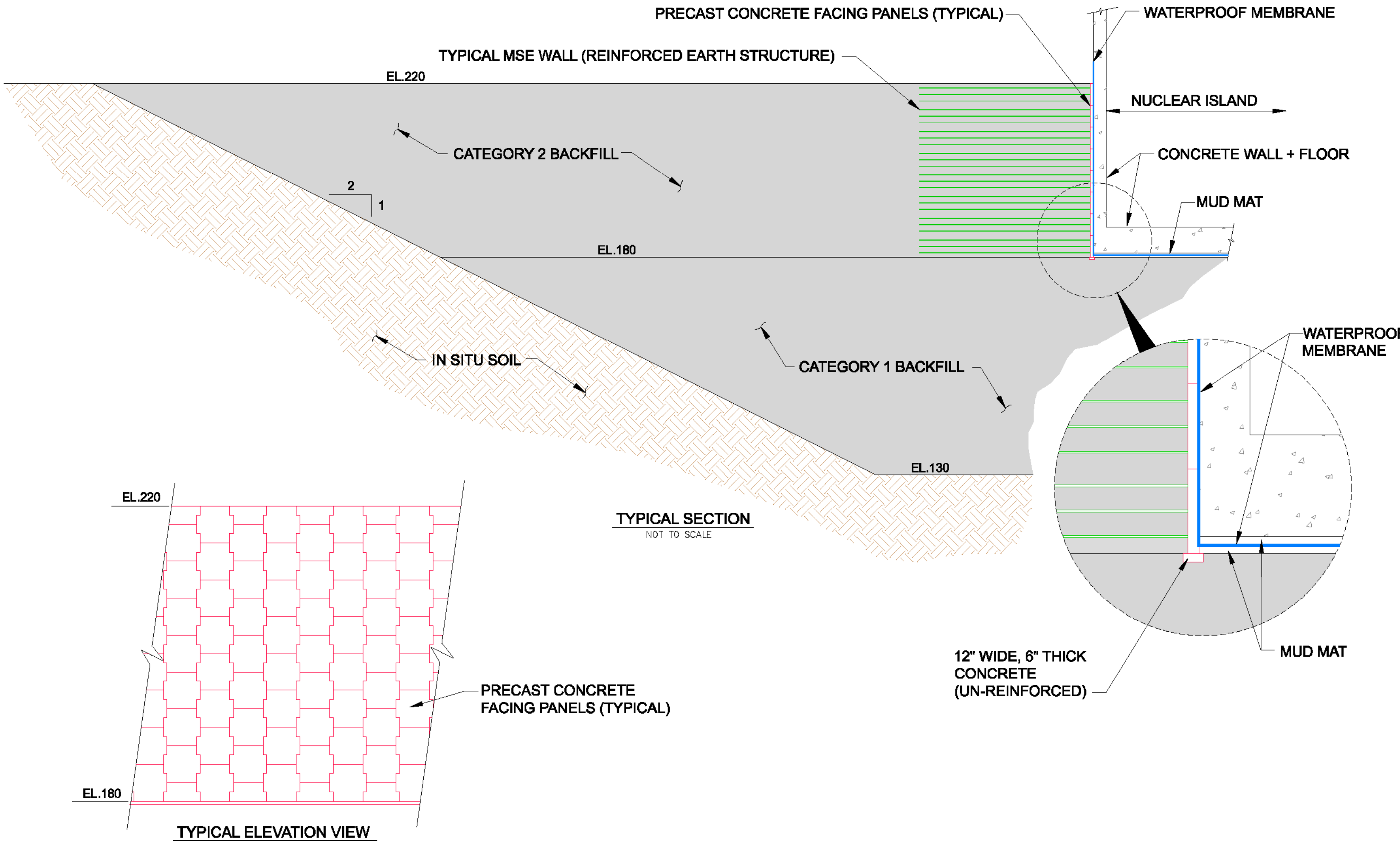


Figure 2.5.4-17 Nuclear Island Temporary Retaining Wall

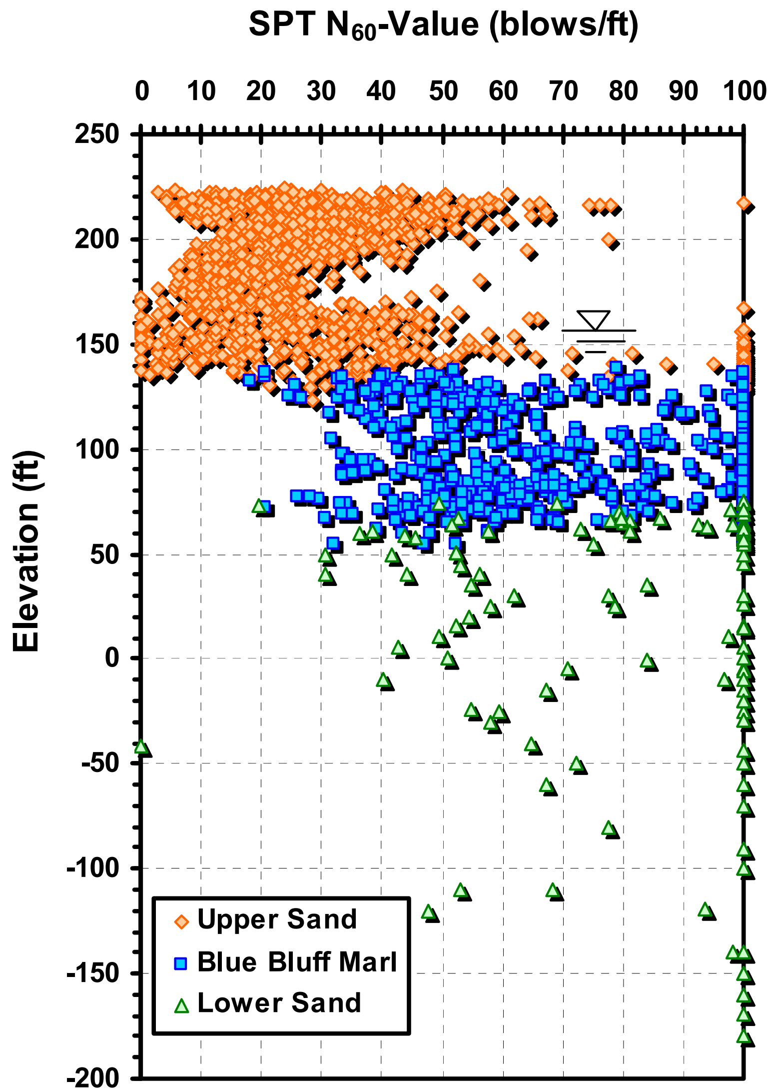


Figure 2.5.4-18 Distribution of SPT N60–Value with Elevation (COL)

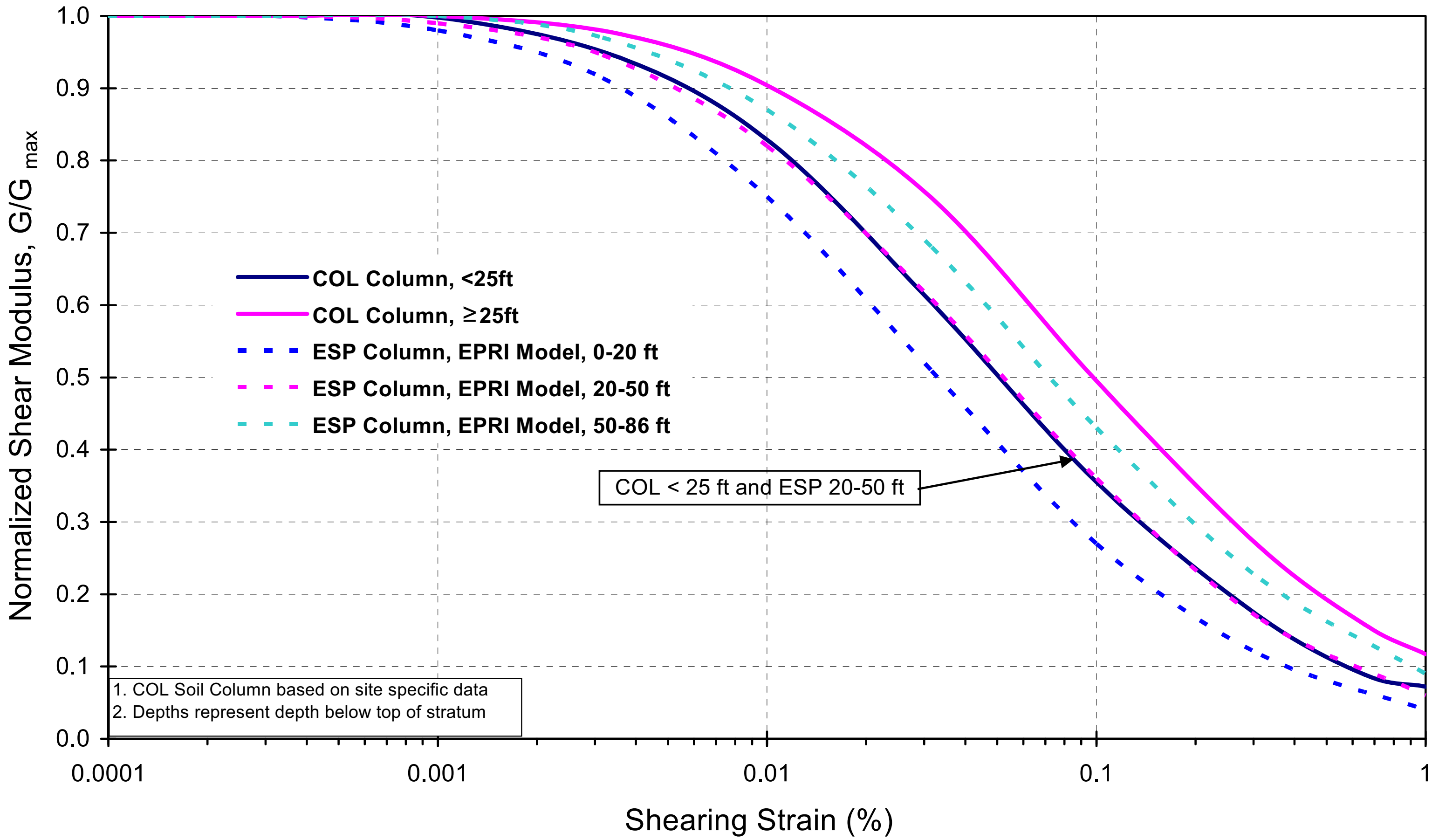


Figure 2.5.4-19a Comparison of Shear Modulus Reduction Curves - Backfill Soils

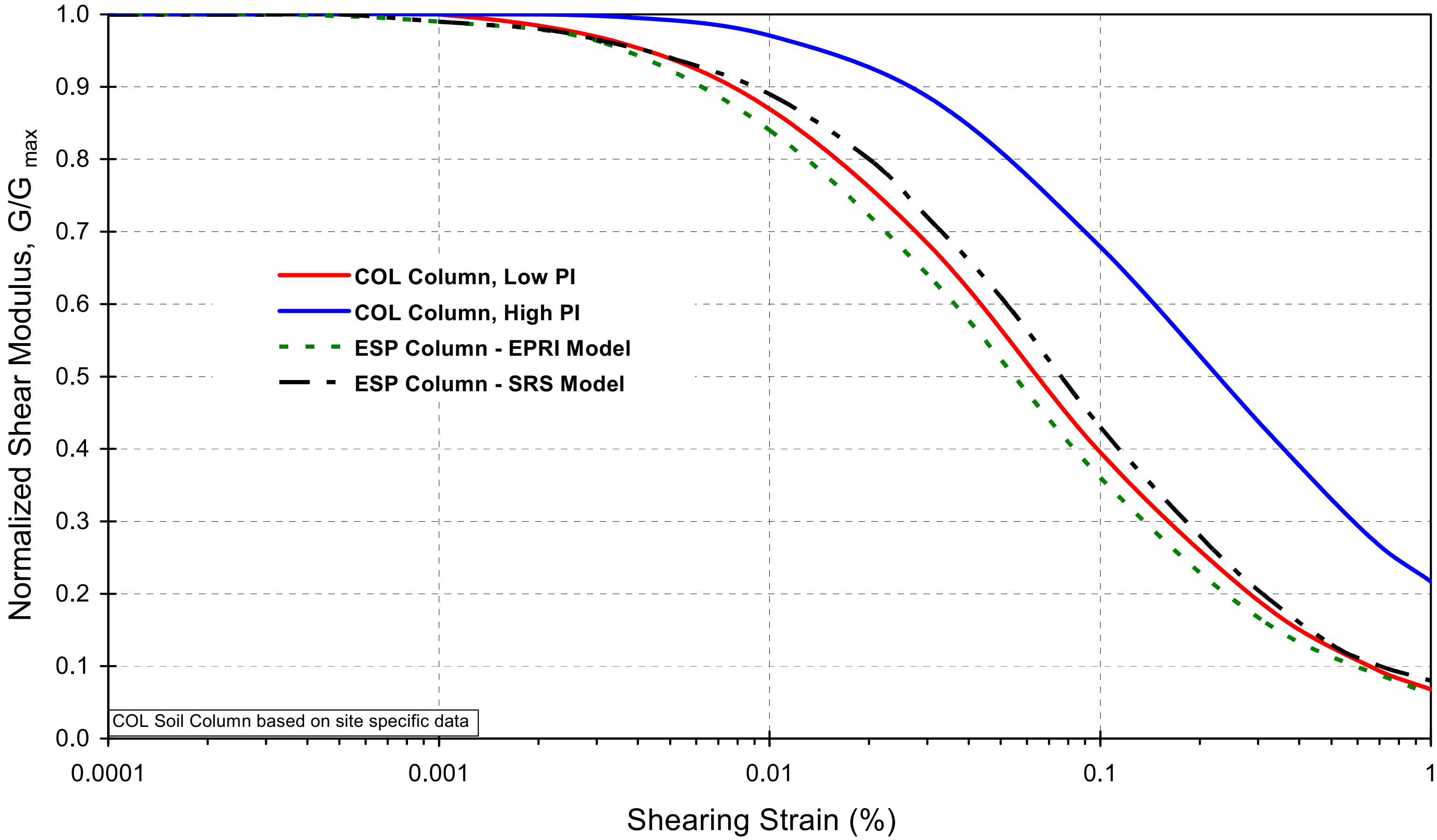


Figure 2.5.4-19b Comparison of Shear Modulus Reduction Curves - Blue Bluff Marl

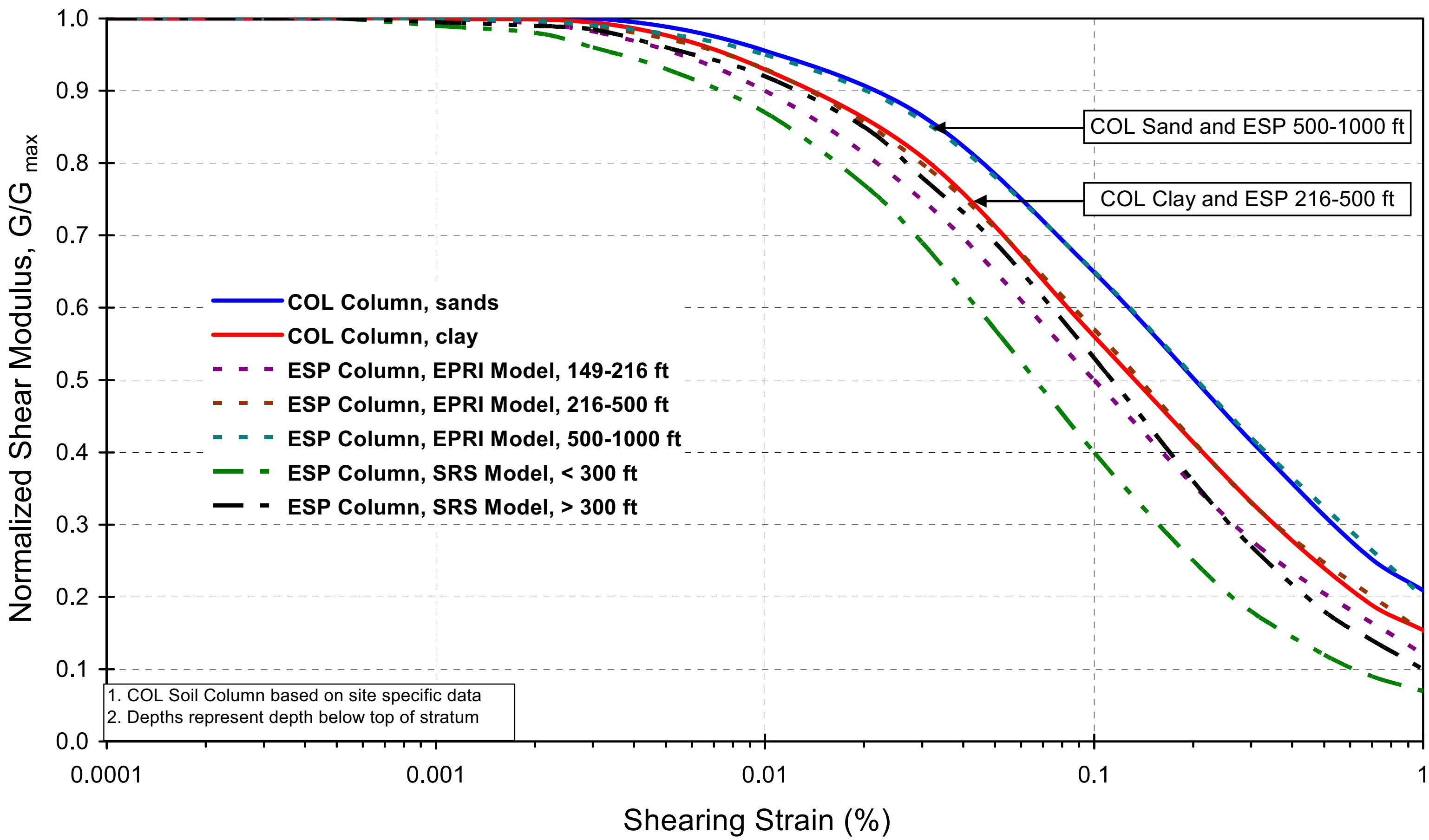


Figure 2.5.4-19c Comparison of Shear Modulus Reduction Curves - Lower Sands

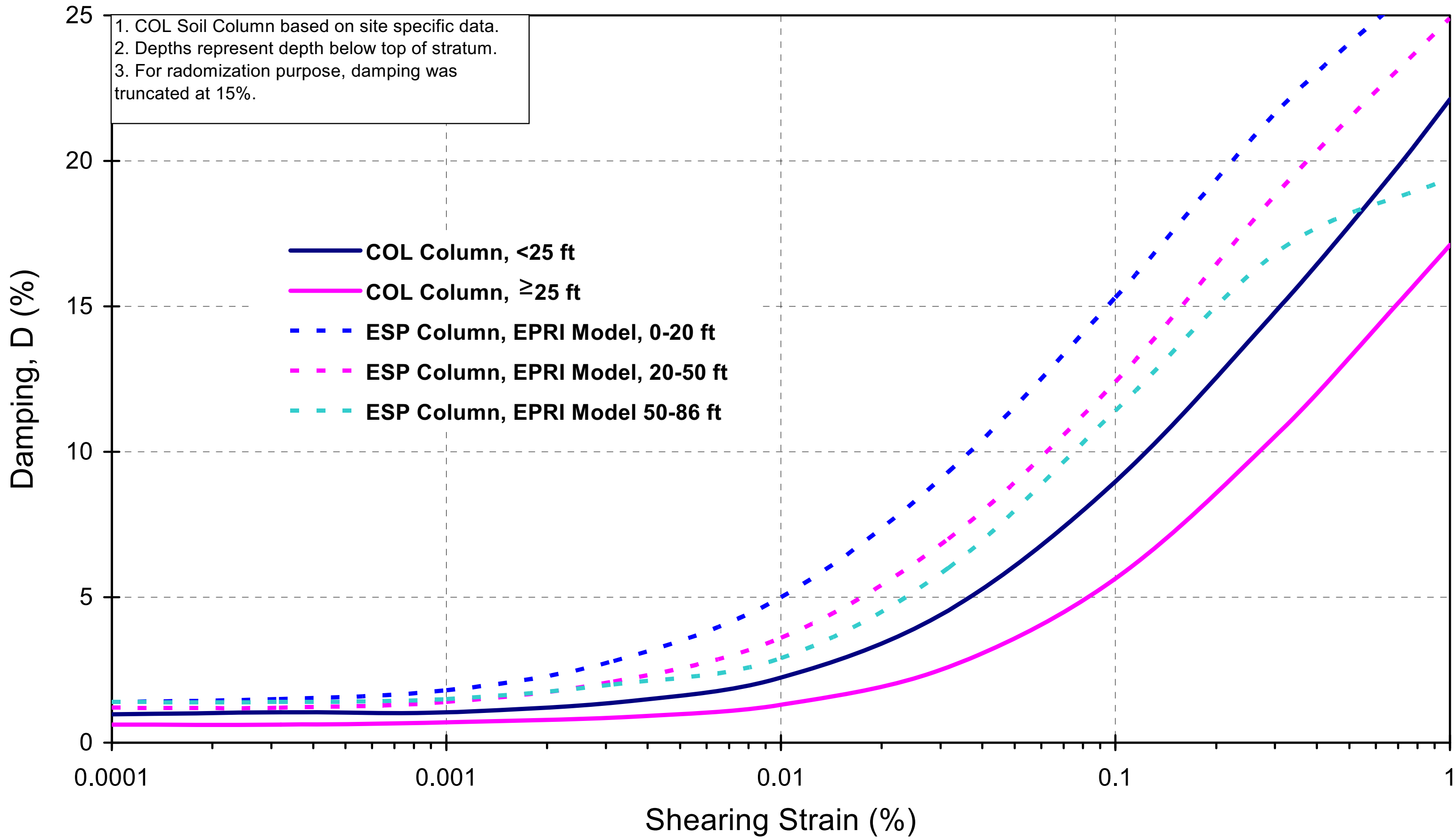


Figure 2.5.4-20a Comparison of Damping Ratio Curves - Backfill Soils

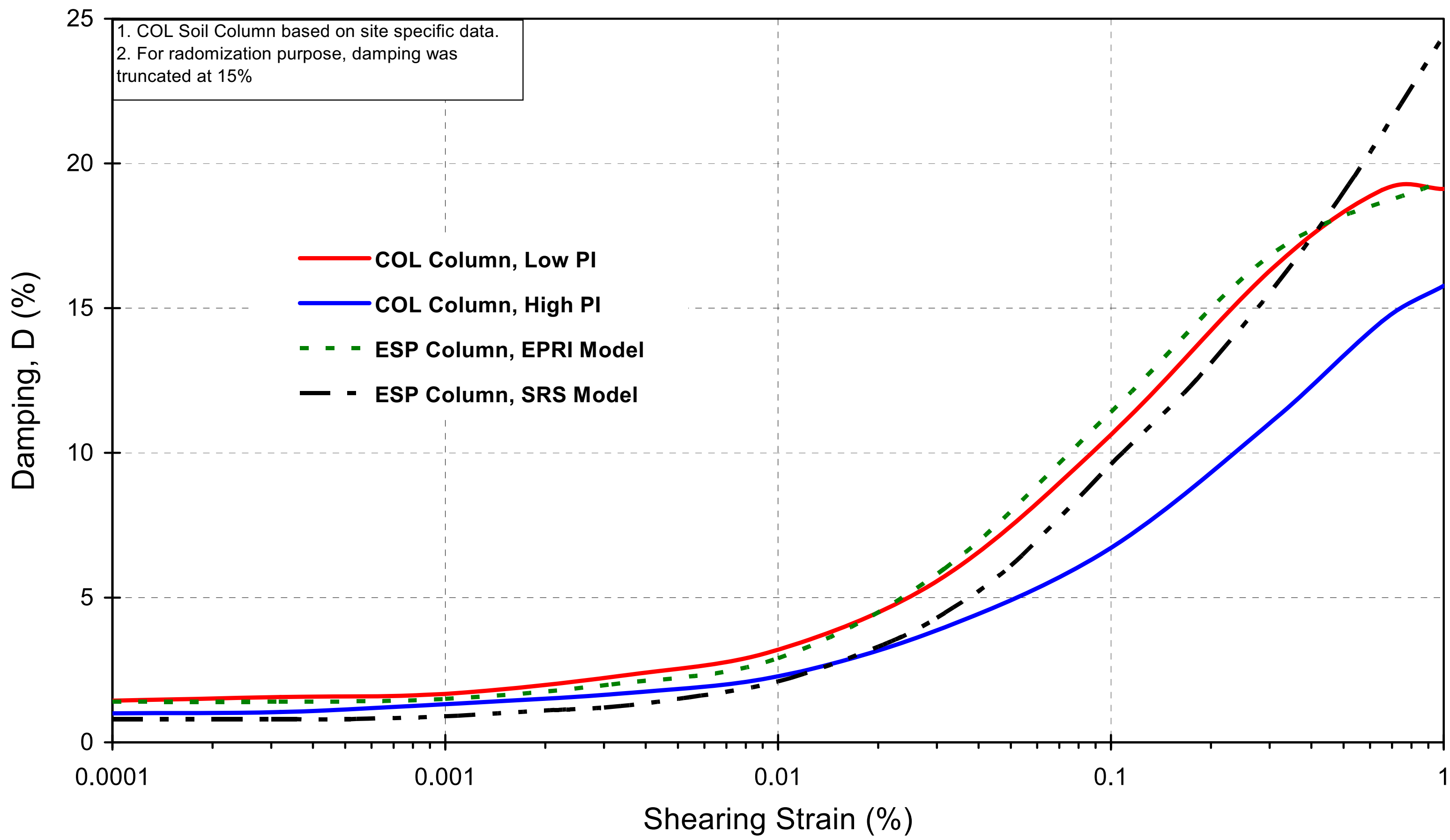


Figure 2.5.4-20b Comparison of Damping Ratio Curves - Blue Bluff Marl

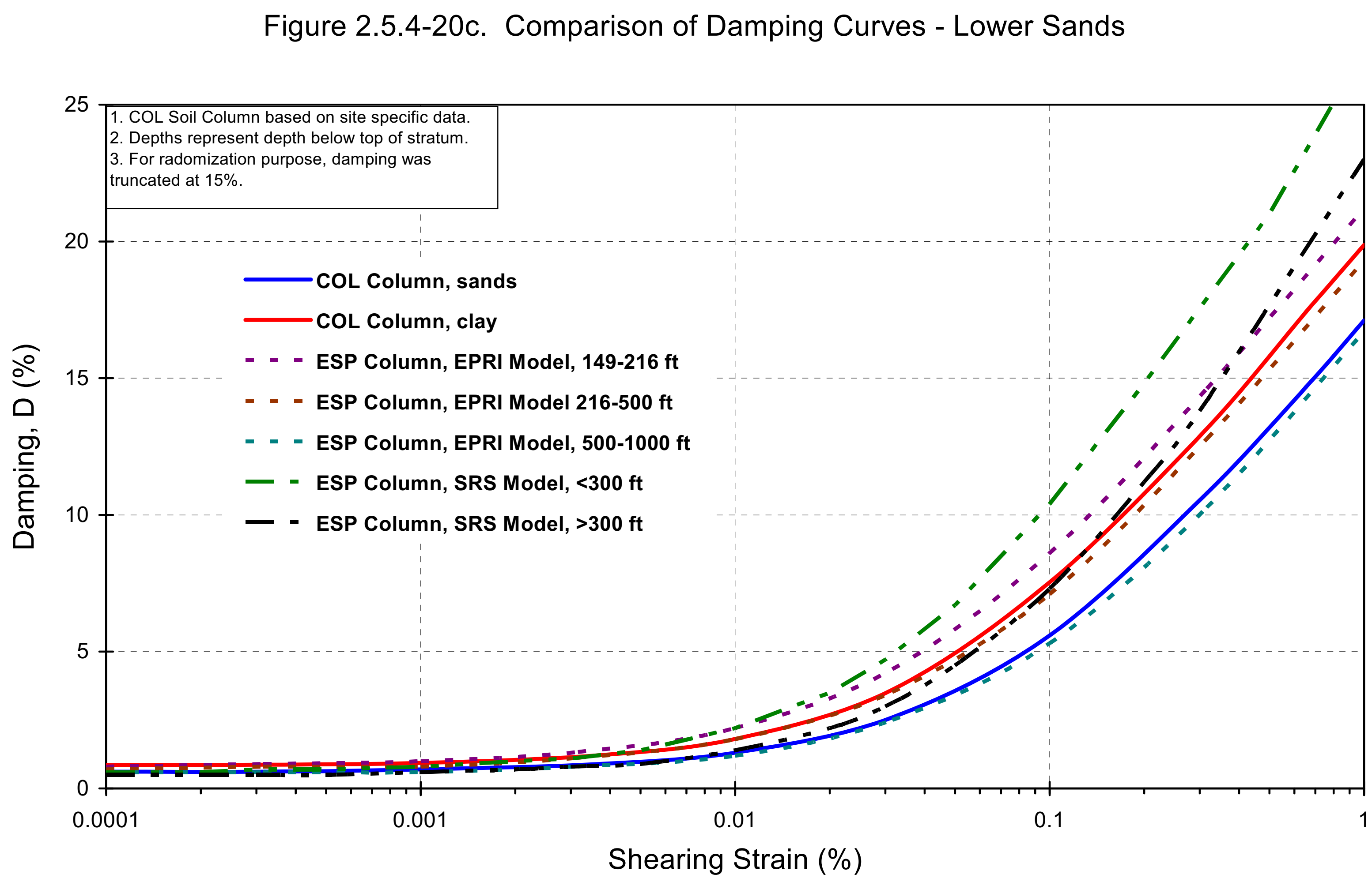


Figure 2.5.4-20c Comparison of Damping Ratio Curves - Lower Sands

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