2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

The U.S. EPR DCD includes the following COL Item for Section 2.5.4:

> A COL Applicant that references the U.S. EPR design certification will present site-specific information about the properties and stability of all soils and rocks that may affect the nuclear power plant facilities, under both static and dynamic conditions including the vibratory ground motions associated with the Safe Shutdown Earthquake (SSE)

This COL item is addressed in the following sections.

This subsection addresses subsurface materials and foundation conditions for the {Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 site.} It was prepared based on the guidance in relevant sections of NRC Regulatory Guide 1.206, Combined License Applications for Nuclear Power Plants (LWR Edition) (NRC, 2007).

The information presented in this subsection is based on results of a subsurface investigation program implemented at the CCNPP Unit 3 site, and evaluation of the collected data, unless indicated otherwise. The data are contained in Geotechnical Subsurface Investigation Data Report (Schnabel, 2007). The data is also presented as Appendix 2.5-A.

The CCNPP Units 1 and 2 Updated Final Safety Analysis Report (UFSAR) (BGE, 1982) contains a summary of the geotechnical information collected previously for the construction of CCNPP Units 1 and 2. The planned CCNPP Unit 3 is approximately 2,000 ft south of the existing units. CCNPP Units 1 and 2 UFSAR (BGE, 1982) contains mostly general information that is quantitatively limited in its extent and depth of exploration relative to the investigation performed for the CCNPP Unit 3. Therefore, the comparison information was limited to those cases when comparable information obtained from the CNNPP Unit 3 subsurface investigation (Schnabel, 2007) was available in the CCNPP Units 1 and 2 UFSAR (BGE, 1982).

References to elevation values in this subsection are based on the National Geodetic Vertical Datum of 1929 (NGVD29), unless stated otherwise.}

2.5.4.1 Geologic Features

{Section 2.5.1.1 addresses the regional geologic settings, including regional physiography and geomorphology, regional geologic history, regional stratigraphy, regional tectonic and non-tectonic conditions, and geologic hazards, as well as maps, cross-sections, and references. Section 2.5.1.2 addresses the geologic conditions specific to the site, including site structural geology, site physiography and geomorphology, site geologic history, site stratigraphy and lithology, site structural geology, seismic conditions, and site geologic hazard evaluation, accompanied by figures, maps, and references. Pre-loading influences on soil deposits, including estimates of consolidation, pre-consolidation pressures, and methods used for their estimation are addressed in Section 2.5.4.2. Related maps and stratigraphic profiles are also addressed in Section 2.5.4.2.

In summary, the site is located in the Atlantic Coastal Plain physiographic province. The soils were formed by ancient rivers carrying large quantities of solids from the northern and western regions into the Atlantic Ocean. These deposits were placed under both freshwater (fluvial) and saltwater (marine) environments, and are about 2,500 feet thick at the site (BGE, 1982). The upper soils are Quaternary, Holocene- and/or Pleistocene-Age deposits formed as beaches or terraces. The lower soils are Miocene-, Eocene-, Paleocene-, and Cretaceous-Age deposits. The Miocene and Eocene soils belong to the Chesapeake and Nanjemoy groups. The

Holocene, Pleistocene, Miocene, and Eocene soils were the subject of a detailed subsurface exploration for the COL investigation, as described below.

2.5.4.2 Properties of Subsurface Materials

This subsection addresses the properties of subsurface materials. It is divided into several parts, as follows.

- Sections 2.5.4.2.1.1 through 2.5.4.2.1.3 describe the subsurface conditions and properties of soils
- Section 2.5.4.2.1.4 describes the chemical properties of soils
- Section 2.5.4.2.1.5 addresses materials below a depth of 400 ft
- Section 2.5.4.2.1.6 provides a summary of the field investigation program
- Section 2.5.4.2.1.7 provides a summary of the laboratory testing program

2.5.4.2.1 Description of Subsurface Materials

The site geology is comprised of deep Coastal Plain sediments underlain by bedrock, which is about 2,500 ft below the ground surface for CCNPP Units 1 and 2 UFSAR (BGE, 1982). The site soils consist of marine and fluvial deposits. The upper approximately 400 ft of the site soils were the subject of the CCNPP Unit 3 subsurface investigation. These soils can be divided into the following stratigraphic units.

- Stratum I: Terrace Sand
- Stratum IIa: Chesapeake Clay/Silt
- Stratum IIb: Chesapeake Cemented Sand
- Stratum IIc: Chesapeake Clay/Silt
- Stratum III: Nanjemoy Sand

Information on deeper soils (below 400 ft) was obtained from the available literature, and it will be discussed later in this subsection. Identification of Strata I through III was based on their physical and engineering characteristics. The characterization of the soils was based on a suite of tests performed on these soils, consisting of standard penetration tests (SPT) in soil borings including hammer energy measurements, cone penetration test (CPT) soundings, test pits, geophysical suspension P-S velocity logging, field electrical resistivity testing, and observation wells, as well as extensive laboratory testing. The extent of the field tests is summarized in Table 2.5.4-1. Locations of these tests are shown in Figure 2.5.4-1 through Figure 2.5.4-3. Subsurface profiles inferred from these tests are shown in Figure 2.5.4-5 through Figure 2.5.4-9, with a subsurface profile legend provided in Figure 2.5.4-4.

The natural topography at the CCNPP site, at the time of the subsurface exploration, was gently rolling. Site-wide, however, the relief could vary by as much as 100 ft. In the area where CCNPP Unit 3 is planned, ground surface elevations at the time of the exploration ranged from approximately elevation 50 ft to elevation 120 ft, with an average of about elevation 88 ft. The planned elevation (rough grade) in the powerblock area ranges from about elevation 75 ft to elevation 85 ft, with the centerline of Unit 3 at elevation 84.7 ft, or approximately elevation 85 ft. The Powerblock includes the Reactor Building, Fuel Building, Safeguard Building, Emergency Power Generating Building, Nuclear Auxiliary Building, Access Building, Radioactive Waste Building, Turbine Building, and Ultimate Heat Sink.

The subsurface conditions were established from the information contained in the Geotechnical Subsurface Investigation Data Report (Schnabel, 2007). The subsurface profiles illustrate these conditions. The maximum depth explored was about 400 ft beneath the ground surface at boring locations B-301 and B-401. The maximum depth explored by CPT soundings was 142 ft beneath the ground surface at location C-407 (CPT soundings encountered repeated refusal and, therefore, could not be consistently extended to greater depths). Field tests (borings, CPTs, etc.) identified as 300-series, e.g., B-301 or C-301, are located in Unit 3 area. Tests identified as 400-series, e.g., B-401 or C-401, are located in an area adjacent to CCNPP Unit 3, hereafter referred to as Construction Laydown Area 1 (CLA1). Field tests identified as 700 series, e.g., B-701 or C-701, are located outside of these two areas, and include the proposed cooling tower, switchyard, and intake/discharge piping locations. Bedrock is too deep (about 2,500 ft below ground) to be of interest for earthwork and foundation design and construction. Therefore, rock properties will not be addressed in similar detail as the overlying soils. The major strata identified from the boring logs, as shown on the subsurface profiles (Strata I, II, and III), are described in detail in the next subsections.

2.5.4.2.1.1 Stratum I – Terrace Sand

The Terrace Sand stratum consists primarily of light-brown to brown sand with varying amounts of silt, clay, and/or gravel, sometimes with silt or clay interbeds. This stratum was fully penetrated by boreholes installed within CCNPP Unit 3 Powerblock area and the adjoining CLA1 area (the 300 and 400 series borings) and by a majority of boreholes drilled outside of these two areas (the 700 series borings). This stratum is not present in low lying areas.

The thickness of Stratum I soils was estimated from the boring logs. In CCNPP Unit 3 area, its thickness with respect to the existing ground surface varies from about 2 ft to 51 ft, with an average thickness of about 21 ft. The average bottom for Stratum I soils is about elevation 66 ft in CCNPP Unit 3 area. The average thickness and bottom elevation for Stratum I soils for the combined CCNPP Unit 3 and CLA1 areas is about 27 ft and elevation 65 ft, respectively. Additional information on thickness and termination elevation for this stratum at locations other than Unit 3, including site-wide, is presented in Tables 2.5.4-2 and 2.5.4-3. Based on site-wide information, the termination elevation for Stratum I was estimated at about elevation 61 ft. An elevation of 60 ft was adopted for simplicity.

It should be noted that at isolated locations, sandy soils with an appearance similar to Stratum I soils were encountered. These soils are suspected of being man-made fill. They were present at the ground surface, above Stratum I soils, and were encountered in 17 borings (B-309, B-336, B-340, B-341, B-406, B-409, B-412, B-415, B-419, B-420, B-438/A, B-439, B-440, B-701, B-710, B-713, and B-768). Mainly, they were found in areas which had previously been developed at the site, such as Camp Conoy, roadways, and ball field areas. Their thickness ranged from approximately 0.5 ft to 17 ft, with an average thickness of about 7 ft.

Soil samples were collected from the borings via SPT and tube samples. Samples were collected more frequently in the upper portion of the borings than in the lower portion, e.g., typically 6 samples were obtained in the upper 15 ft. Thereafter, samples were obtained at 5 ft intervals. SPT N-values were measured during the sampling and recorded on the boring logs. In CCNPP Unit 3 area, the SPT N-values in Stratum I soils ranged from 0 blows/ft (weight of hammer [WOH] or weight of rod [WOR]) to 70 blows/ft, with an average measured N-value of 10 blows/ft. Additional SPT information on this layer at locations other than Unit 3, including site-wide, is presented in Table 2.5.4-4. The measured N-values versus elevation are presented in Figure 2.5.4-10. It indicates that a majority of the SPT N-values are within a relatively uniform range of about 3 to 13 blows/ft, with occasional higher values between about elevation 70 ft and elevation 90 ft.

The WOH and WOR values were very infrequent in Stratum I soils. A total of 5 WOH and WOR conditions were encountered in borings at CCNPP Unit 3 location, and a total of 5 were observed in all other borings. At the CCNP Unit 3 location, three of these conditions were in boring B-309 in materials designated as "fill," which will be removed during construction. The fourth episode was in boring B-314 at the ground surface which will also be removed during construction. The fifth value was in boring B-322 at about elevation 70 ft, at the location of the Essential Service Water System (ESWS) Cooling Tower. The cause of this low SPT value is likely due to sampling disturbance. A review of the boring logs and stratigraphic profiles for the same soils at other locations do not indicate this to be the predominant situation. Rather, the low SPT value is an isolated, infrequent situation, most likely caused by factors other than the natural condition of Stratum I soils. Nonetheless, these soils will be removed during excavation for the ESWS Cooling Tower to at least elevation 60 ft. In conclusion, at the CCNP Unit 3 location, the 5 WHO and WOR results are inconsequential to the stability of Stratum I soils.

Five drill rigs were used for the COL subsurface exploration. SPT hammer energies were measured for each of the five drilling rigs utilized. Energy measurements were made in 5 borings (B-401, B-403, B-404, B-409, and B-744). Because the SPT N-value used in correlations with engineering properties is the value corresponding to 60 percent hammer efficiency, the measured SPT N-values were adjusted based on the energy measurements, in accordance with American Society for Testing and Materials (ASTM) D6066 (ASTM, 2004f)). The average energy transfer ratio (ETR) obtained from hammer energy measurements for each drilling rig was applied to the measured SPT N-values. The average ETR ranged from 78 percent to 87 percent, or an N-value adjustment factor ranging from 1.30 to 1.45. A summary of the measured ETR values for each drill rig is shown in Table 2.5.4-5. The measured SPT N-values from each boring were adjusted using the appropriate ETR value shown in Table 2.5.4-5 for the drill rig utilized. The adjusted average field-measured N-value for Stratum I soils is 16 blows/ft. A value of 15 blows/ft was conservatively adopted for engineering purposes, as shown in Table 2.5.4-6. Based on corrected SPT N-values, Stratum I soils are considered medium dense on average.

CPT soundings were performed in Stratum I soils. The cone tip resistance, q_c, in these soils ranged from about 2 to 570 tons per square ft (tsf), with an average of about 120 tsf. The CPT tip resistance profile versus elevation is shown in Figure 2.5.4-11. The results indicate the q_c values in Stratum I soils to be typically limited to about 200 tsf, with values peaking much higher between elevation 80 ft to elevation 90 ft. The CPT results also indicate the presence of clay zones within this stratum, at about elevation 115 ft, elevation 100 ft, and elevation 90 ft. Estimated relative density from CPT data ranges from about 30 to near 100 percent, with an average of about 65 percent.

Laboratory index tests and testing for determination of engineering properties were performed on selected samples from Stratum I soils. Laboratory test quantities are summarized in Table 2.5.4-7. Sample selection for testing was primarily based on the observed soil uniformity from the field classification, or conversely, the variation in material description based on logging in the field, in order to obtain a quantitative measure of the uniformity, or the variation, respectively. The following index tests were performed on Stratum I soils, with results as noted.

	No. of Tests	Min. Value	Max. Value	Average Value
Water Content (%)	31	3	44	15
Liquid Limit (LL)(%)	31	Non-Plastic (NP)	75	NP
Plastic Limit (PL)(%)	31	NP	23	NP
Plasticity Index (PI)	31	NP	52	NP
Fines Content (%)	85	3	71	19
Unit Weight (pcf)	3	115	124	120

The test results are summarized in Table 2.5.4-8. The water content and Atterberg limits are presented versus elevation in Figure 2.5.4-12. They are also shown on a plasticity chart in Figure 2.5.4-13. For engineering purposes, Stratum I soils were characterized, on average, as non-plastic with an average fines content (materials passing No. 200 Sieve) of 20 percent. Grain size analyses indicated that these soils are primarily fine or fine-medium sands. The Unified Soil Classification System (USCS) designations were poorly-graded sand/silty sand, silty sand, well-graded sand, clayey sand, clay of high plasticity, silt, clay, and silt with high plasticity, with the predominant classifications of SP-SM and SM. The often plastic and fine-grained soil classifications are from the interbeds within this stratum. Based on the laboratory results, an average unit weight of 120 pounds per cubic foot (pcf) was adopted for these soils.

The shear strength of Stratum I soils was evaluated based on laboratory testing and correlations with SPT N-values and CPT results. Initially, an angle of shearing resistance (or friction angle), ϕ', for the granular Terrace Sand was estimated from an empirical correlation with SPT N-values (Bowles, 1996). Using the SPT N-value adjusted for hammer efficiency, a ϕ' of about 34 degrees was obtained for N=15 blows/ft and for medium-grained sand. A value of ϕ'=33 degrees was considered appropriate. Friction angle values were also obtained from the CPT results, estimated using the method recommended in EPRI Report EL-6800 (EPRI, 1990). They are presented versus elevation in Figure 2.5.4-14. The values shown in Figure 2.5.4-14 range from about 29 to 49 degrees, with an average value of about 40 degrees. One direct shear test was performed on a sample of Stratum I soils designated as clay by USCS, resulting in ϕ'=29.2 degrees and c'=0.3 tsf. The laboratory strength results are given in Table 2.5.4-9. From the above interpretations, a summary of average ϕ' values for Stratum I soils is compiled as follows.

	SPT	CPT	Direct Shear
ϕ' (degrees)	33	40	29*

* c'=0.3 tsf not shown

Based on the above, ϕ'=32 degrees and c'=0 is conservatively adopted for Stratum I soils.

Consolidation properties and stress history of Stratum I soils were evaluated via laboratory testing and evaluation of the CPT data. A summary of the laboratory consolidation test results is presented in Table 2.5.4-10. The results are also shown versus elevation in Figure 2.5.4-15. Results indicate that, on average, these soils are preconsolidated to 5 tsf, with an overconsolidation ratio (OCR) of at least 4. OCR derived from CPT data are shown in Figure 2.5.4-16. The CPT-interpreted results are scattered over a large range, from OCR=0.6 to

OCR=10, with no unique trend. At best, an average OCR may be discerned from the CPT data, or an approximate average OCR of 4 to 5. Summary OCR values from CPT data are shown in Table 2.5.4-10. An average OCR=4 and preconsolidation pressure (Pp') of 4 tsf were adopted for Stratum I soils based on available data.

Static (or high strain) elastic modulus, E, for coarse-grained soils can be evaluated using the following relationship (Davie, 1998).

$$E = 18\ N \text{ (in tsf)} \qquad \text{Eq. 2.5.4-1}$$

where N=SPT N-value in blows/ft. Substituting the previously established N-value for Stratum I soils (SPT N-value=15), an elastic modulus of 270 tsf was estimated for these soils. Also, elastic modulus can be estimated based on shear wave velocity for sandy soils (Senapathy, 2001), as follows.

$$E = 2\ G\ (1 + \mu) \qquad \text{Eq. 2.5.4-2}$$

where,

$$G_{.0001\%} = \gamma/g\ (Vs)^2 \qquad \text{Eq. 2.5.4-3}$$

$$G_{.0001\%} / G_{.375\%} = 10 \quad \text{(for sands)} \qquad \text{Eq. 2.5.4-4}$$

In Eqs. 2.5.4-2 through 2.5.4-4, $G_{.0001\%}$=small strain shear modulus (i.e., strain in the range of 10^{-4} percent), $G_{.375\%}$=large strain (static) shear modulus (i.e., strains in the range of 0.25 percent to 0.5 percent), μ=Poisson's ratio, γ=total soil density, g=acceleration of gravity, and Vs=shear wave velocity.

Using Vs=790 ft/sec obtained from the measurements at the site (refer to Section 2.5.4.4 for discussions on this topic), γ=120 pcf, and taking μ=0.3 for sand, a static (or high strain) modulus of elasticity of 302 tsf is estimated from Eq. 2.5.4-2. Using an average of the estimated values from SPT and shear wave velocity, an elastic modulus of 286 tsf is estimated. A value of 280 tsf was adopted for Stratum I soils. Values of E are shown in Table 2.5.4-11.

The static shear modulus, G, is related to the static modulus of elasticity by the following relationship:

$$G = E / [2\ (1+ \mu)] \qquad \text{Eq. 2.5.4-5}$$

Using μ =0.3 for sandy soils, a shear modulus of 108 tsf was estimated for these soils based on E=280 tsf. A value of 116 tsf was estimated using Eq. 2.5.4-3. A value of 110 tsf was conservatively adopted for Stratum I soils. Values of G are shown in Table 2.5.4-11.

The coefficient of subgrade reaction for 1-ft wide or 1-ft square footings, k_1, was obtained from "Evaluation of Coefficient of Subgrade Reaction" (Terzaghi, 1955). Based on material characterization for Stratum I soils, k_1 = 75 tons per cubic ft (tcf) was estimated and adopted.

Active, passive, and at-rest static earth pressure coefficients, K_a, K_p, and K_0, respectively, were estimated assuming frictionless vertical walls and horizontal backfill using Rankine's Theory and based of the following relationships (Lambe, 1969):

$$K_a = \tan^2(45-\phi'/2) \qquad \text{Eq. 2.5.4-6}$$

$$K_p = \tan^2(45+\phi'/2) \qquad \text{Eq. 2.5.4-7}$$

$$K_0 = 1-\sin(\phi') \qquad \text{Eq. 2.5.4-8}$$

Substituting previously adopted ϕ'=32 degrees for Stratum I soils, the following earth pressure coefficients were estimated; K_a=0.3, K_p=3.3, K_0=0.47. Values adopted for engineering purposes are K_a=0.3, K_p=3.3, and K_0=0.5.

The sliding coefficient is tangent δ, where δ is the friction angle between the soil and the material it is bearing against, in this case concrete. Based on "Foundations & Earth Structures" (NFEC, 1986), tangent δ=0.4 was adopted for Stratum I soils.

All of the material properties adopted for engineering purposes for Stratum I soils, as well as other relevant information, are summarized in Table 2.5.4-12.

2.5.4.2.1.2 Stratum II – Chesapeake Soils

The Chesapeake soils are the dominant materials in the upper 400 ft of the site, with a combined thickness of about 270 ft. They were subdivided into three layers, based on visual appearance and material properties, namely

- Stratum IIa - Chesapeake Clay/Silt
- Stratum IIb - Chesapeake Cemented Sand
- Stratum IIc - Chesapeake Clay/Silt

Each of these strata is described below.

Stratum IIa – Chesapeake Clay/Silt

The Chesapeake Clay/Silt stratum was encountered beneath the Terrace Sand in all boreholes, except in low lying areas where Stratum I soils had been eroded. Stratum IIa typically consists of light to dark gray clay and/or silt, although it is predominately clay, with varying amounts of sand.

The thickness of Stratum IIa soils was estimated from the boring logs. In CCNPP Unit 3 area, its thickness varies from about 4 ft to 35 ft, with an average thickness of about 20 ft. Additional information on thickness of this stratum at locations other than CCNPP Unit 3, including site-wide, is presented in Table 2.5.4-2.

The stratum thickness was based on estimating the termination elevations encountered for the layer at boring locations. In CCNPP Unit 3 area, the termination elevations of Stratum IIa soils were estimated to range from about elevation 56 ft to elevation 38 ft, with an average termination elevation 47 ft. In combined CCNPP Unit 3 and CLA1 areas, the termination elevations were from elevation 56 ft to elevation 27 ft, with an average elevation 46 ft. An elevation 45 ft was adopted for simplicity. Additional termination information on this layer at locations other than CCNPP Unit 3, including site-wide, is presented in Table 2.5.4-3. Only data from borings that fully penetrated the layer were considered for determination of termination elevations.

Soil samples were collected from the borings via SPT and tube sampling. SPT N-values were measured during the sampling and recorded on the boring logs. In the CCNPP Unit 3 area, the SPT N-values ranged from 1 blow/ft to 46 blows/ft, with an average uncorrected N-value of 9 blows/ft. Additional SPT information on this layer at locations other than CCNPP Unit 3, including site-wide, is presented in Table 2.5.4-4. The measured N-values versus elevation are presented in Figure 2.5.4-10. This figure indicates the SPT N-values to be within a relatively narrow range, indicating uniformity in both depth and laterally, although some increase in SPT N-value with depth is evident in Figure 2.5.4-10.

The measured SPT N-values from each boring were adjusted using the appropriate ETR value shown in Table 2.5.4-5 for the drill rig utilized. The adjusted average field-measured N-value for Stratum IIa soils is 13 blows/ft. A value of 10 blows/ft was conservatively adopted for engineering purposes, as shown in Table 2.5.4-6. Based on adjusted SPT N-values, Stratum IIa soils are considered stiff on average.

CPT soundings were performed in Stratum IIa soils. The cone tip resistance values ranged from about 10 to 200 tsf, with an average value of about 50. A profile of q_c versus elevation is shown in Figure 2.5.4-11. The results also indicate a mild increase in tip resistance with depth.

Index tests and testing for determination of engineering properties were performed on selected samples from Stratum IIa soils. Sample selection for testing was primarily based on the observed soil uniformity from the field classification, or conversely, the variation in material description based on logging in the field, in order to obtain a quantitative measure of the uniformity, or the variation, respectively. The following index tests were performed on Stratum IIa soils, with results as noted.

	No. of Tests	Min. Value	Max. Value	Average Value
Water Content (WC) (%)	67	11	88	32
Liquid Limit (LL) (%)	67	Non-Plastic (NP)	86	57
Plastic Limit (PL) (%)	67	NP	22	22
Plasticity Index (PI)	67	NP	64	35
Fines Content (%)	72	29	99	77
Unit Weight (pcf)	40	103	124	116

The test results are summarized in Table 2.5.4-8. The water content and Atterberg limits are presented versus elevation in Figure 2.5.4-12. They are also shown on the plasticity chart in Figure 2.5.4-13. For engineering purposes, Stratum IIa soils were characterized, on average, as medium-high plasticity clays, with an average PI=35. Their predominant USCS designation was clay of high plasticity and silt of high plasticity (CH and MH), however, sometimes with silty sand, silty sand to clayey sand, and organic clay. The organic designation was based on laboratory (liquid limit) tests. As visual indications to presence of organic soils were not noted during the field sampling, follow up laboratory organic contents tests were performed. Results of 8 tests indicated organic contents in the range of 0.1 percent to 1.6 percent, with an average of 0.9 percent. With less than 1 percent organic matter on average, and observations during sampling, these soils are not considered organic. Also from the laboratory test results, an average unit weight of 115 pcf was adopted for these soils.

The shear strength of Stratum IIa soils was evaluated based on laboratory testing, and using correlations with SPT N-values and the CPT results. The results are summarized in Table 2.5.4-13.

The undrained shear strength, su, was estimated from empirical correlations with SPT N-value (Lowe, 1975), using

s_u = N/16 (in tsf) Eq. 2.5.4-9

where N=SPT N-value in blows/ft. Substituting the previously established N-value for Stratum IIa soils (SPT N-value=10), s_u=0.63 tsf is estimated for these soils. Undrained shear strength was also estimated using the CPT data, following a CPT-su correlation from Robertson (Robertson, 1988, as follows.

$$s_u = (q_t - \sigma_v) / N_{kt} \qquad \text{Eq. 2.5.4-10}$$

where, qt is the cone tip resistance, σv is the total overburden stress, and Nkt is cone factor that varies between 10 and 20. A cone factor Nkt=15 was used as an average value for the analysis of the CPT data. The shear strength values obtained from the CPT data indicate an average su=1.6 tsf. Results of 43 laboratory unconsolidated undrained (UU) triaxial and unconfined compression (UC) tests on selected samples indicate an average su=1.1 tsf. The laboratory shear strength test results are shown versus elevation in Figure 2.5.4-17. The CPT-derived values are shown versus elevation in Figure 2.5.4-18. Based on these results, an undrained shear strength of 1.0 tsf was conservatively adopted for Stratum IIa soil.

The angle of shearing resistance of these soils was evaluated from laboratory test results. The results are shown in Table 2.5.4-9. Eleven direct shear tests were performed on samples of Stratum IIa soils, mostly designated as CL and CH by the USCS soil classification system, resulting in an average ϕ'=25 degrees and c'=0.5 tsf. Strength parameters from 6 isotropically consolidated triaxial (CIU-bar) tests, indicated average (effective) ϕ'=27 degrees and c'=0.4 tsf and average (total) ϕ=14 degrees and c=0.7 tsf. From the above, the following is a summary of average ϕ' and ϕ values for Stratum IIa soils based on various data and interpretation.

	Direct Shear	CIU-bar
ϕ' (degrees)	25	27
c' (tsf)	0.5	0.4
ϕ (degrees)	---	14
c (tsf)	---	0.7

The direct shear and CIU-bar results are comparable. Based on the above, ϕ'=26 degrees and c'=0.4 tsf is adopted for Stratum IIa soils.

Consolidation properties and stress history of Stratum IIa soils were assessed via laboratory testing and evaluation of the CPT data. A summary of the laboratory consolidation test results is presented in Table 2.5.4-10. The results are also plotted versus elevation and shown in Figure 2.5.4-15. Results indicate that, on average, these soils are preconsolidated to about 9 tsf, with an OCR of at least 5. OCR data derived from CPT results are shown in Figure 2.5.4-16. The CPT-interpreted results are scattered over a large range, from OCR=0.6 to OCR=10, with no unique trend. At best, an average OCR may be discerned from the CPT data, or an approximate OCR of 5 to 6. Summary of OCR values from CPT data is shown in Table 2.5.4-10. An OCR=4 and a preconsolidation pressure of 6 tsf were conservatively adopted for Stratum IIa soils.

Static modulus of elasticity for fine-grained soils was evaluated using the following relationship (Davie, 1988).

$$E = 600\ su \qquad \text{Eq. 2.5.4-11}$$

This relationship was modified for the CCNPP site soils based on their plasticity, as follows.

$$E = 450\ s_u \qquad \text{Eq. 2.5.4-12}$$

Substituting the previously established s_u for Stratum IIa soils (i.,e., s_u=1 tsf), an elastic modulus of 450 tsf is estimated. Other relationships for static modulus of elasticity are also available for fine-grained soils (Ref. 2.5.4-8), as follows.

$$G_{.0001\%} / G_{.375\%} = 21/\sqrt{PI} \quad \text{(for clays)} \qquad \text{Eq. 2.5.4-13}$$

$$G_{.375\%} / s_u = 200 \quad \text{(for clays)} \qquad \text{Eq. 2.5.4-14}$$

It is noted that Eq. 2.5.4-14 (Senapathy, 2001) was derived based on Eqs. 2.5.4-2 and 2.5.4-11 using a Poisson's ratio of 0.5, and therefore this equation has similarities with Eq. 2.5.4-12. Using Vs=1,100 ft/sec obtained from the measurements at the site (refer to subsection 2.5.4.4 for discussions on this topic), γ=115 pcf, PI=35, and using μ=0.45 for clay, static (or high strain) modulus of elasticity of 1,766 tsf is estimated from Eqs. 2.5.4-2, 2.5.4-3, and 2.5.4-13. Using s_u=1.0 tsf, an elastic modulus of 580 tsf is estimated from Eqs. 2.5.4-2 and 2.5.4-14. Of the preceding estimates, the value based on PI appears high whereas the other two estimates are comparable, therefore, the PI-based value was omitted in selecting an average elastic modulus for Stratum IIa soils. Using an average of the estimated values from undrained strength and shear wave velocity correlated with s_u, an elastic modulus of 515 tsf is estimated (average of 450 and 580 tsf). A value of 510 tsf was conservatively adopted for Stratum IIa soils. The values are shown in Table 2.5.4-11.

The static shear modulus, G, was estimated using Eq. 2.5.4-5. Using μ=0.45 for clay soils, a shear modulus of 176 tsf is estimated based on the corresponding E value. Values of 609 and 200 tsf were estimated using Eqs. 2.5.4-13 and 2.5.4.-14. The highest value was ignored for conservatism. An average of the two other values, 180 tsf, was conservatively adopted for Stratum IIa soils. The values are shown in Table 2.5.4-11.

The coefficient of subgrade reaction for 1-ft wide or 1-ft square footings, k_1, was obtained from Terzaghi (Terzaghi, 1995). Based on material characterization for Stratum IIa soils, k_1=75 tcf was estimated and adopted.

Active, passive, and at rest static earth pressure coefficients, K_a, K_p, and K_0, respectively, were estimated using Eqs. 2.5.4-6, 2.5.4-7, and 2.5.4-8. Substituting the previously adopted ϕ'=26 degrees for Stratum IIa soils, the following earth pressures coefficients are estimated; K_a=0.4, K_p=2.6, and K_0=0.6. Given the overconsolidated nature of the soils, and considering the adopted OCR value, the K_0 value was increased by 33 percent based on experience. The adopted values for engineering purposes are K_a=0.4, K_p=2.6, and K_0=0.8.

The sliding coefficient (tangent δ) of 0.35 was adopted for Stratum IIa soils in contact with concrete (NFEC, 1986).

All of the material properties adopted for engineering purposes for Stratum IIa soils, as well as other useful information, are summarized in Table 2.5.4-12.

Stratum IIb – Chesapeake Cemented Sand

The Chesapeake Cemented Sand stratum was encountered beneath Stratum IIa in all the boreholes. This stratum includes interbedded layers of light to dark gray silty/clayey sands, sandy silts, and low to high plasticity clays, with varying amounts of shell fragments and with varying degrees of cementation. The predominant soils, however, are sandy.

The thickness of Stratum IIb soils was estimated from the boring logs. In the CCNPP Unit 3 area, its thickness varies from about 57 ft to 73 ft, with an average thickness of about 66 ft. Additional information on the thickness of this layer at locations other than CCNPP Unit 3, including site-wide, is presented in Table 2.5.4-2.

In CCNPP Unit 3 area, the termination elevations of Stratum IIb soils were estimated to range from about elevation 3 ft to elevation -31 ft, with an average termination elevation -19 ft. In combined CCNPP Unit 3 and CLA1 areas, the termination elevations were in the same range as in CCNPP Unit 3, however, with an average elevation -17 ft. An elevation of -15 ft was adopted for simplicity. Additional information on termination elevations for this layer at locations other than CCNPP Unit 3, including site-wide, is presented in Table 2.5.4-3. Only data from borings that fully penetrated the layer were considered for determination of termination elevations.

Soil samples were collected from the borings via SPT and tube samples. In the CCNPP Unit 3 area, the SPT N-values ranged from 4 blows/ft to greater than 100 blows/ft, with an average N-value of 45 blows/ft. Site-wide, an average SPT N-value of 41 blows/ft was estimated. SPT values exceeding 100 blows/ft were common in these soils, resulting in sampler refusal. Based on SPT N-values and penetration resistances observed, on average, Stratum IIb soils are considered very dense. The measured SPT N-values from each boring were adjusted using the appropriate ETR value shown in Table 2.5.4-5 for the drill rig utilized. The adjusted average field-measured N-value for Stratum IIb soils is 48 blows/ft when the adjusted values are "capped" at 100 blows/ft. When the adjusted values are not capped at 100 blows/ft, an average N-value of 56 blows/ft is obtained. For conservatism, a value of 45 blows/ft was adopted for engineering purposes, as shown in Table 2.5.4-6. Additional SPT information on this layer at locations other than Unit 3, including site-wide, is presented in Table 2.5.4-4. The measured N-values versus elevation are presented in Figure 2.5.4-10. They indicate large variations in SPT N-value over the entire thickness of this stratum, due to varying degrees of cementation. Higher cementation in the top half and relatively lower cementation in the lower half is suggested by the SPT N-values. Laterally, the variation in cementation is rather uniform across both CCNPP Unit 3 powerblock and CLA1 areas.

CPT soundings were attempted in Stratum IIb soils. However, the soils could only be partly penetrated. All CPT soundings experienced refusal when encountering the highly cemented portions of these soils. The CPT soundings could only be advanced after predrilling through the highly cemented zones, and sometimes the predrilling had to be repeated due to the intermittent presence of hard zones at the same sounding. Values of q_c from the soundings ranged from about 40 to over 600 tsf. The average q_c value may range from 200 to 300 tsf. The results corroborate with the SPT N-values where the highest N-values were measured in zones that CPT soundings encountered refusal or could not penetrate these soils, approximately between elevation 20 and elevation 40 ft. The q_c profile is shown in Figure 2.5.4-11.

Low SPT N-values and q_c values are very infrequent in this stratum, given the influence of cementation. The low values are very likely the result of sampling disturbance, or in one case (at C-406, elevation ~30 ft, q_c~10 tsf) the low tip resistance is due to the relatively low overburden pressure at that location. They could also be influenced by groundwater, given that the "confined" groundwater level is roughly near the top of this stratum (refer to Section 2.5.4.6 for groundwater information). The cementation in Stratum IIb soils varies, including zones that are highly cemented and others with little cementation. The degree of cementation was subjectively evaluated during the field exploration by observing the degree of shell fragmentation present and testing the soils with diluted hydrochloric acid, as noted on the boring logs. The cementation is affected by the presence of shells in these soils. The influence of iron oxide may also be a factor, although no specific test was performed on the samples for verification of iron contents. These soils, however, have been studied in the past by others, as follows.

Based on a study of soils near Calvert Cliffs (Rosen, 1986), dolomite or calcite, which is present in the local soils, is identified as the cementing agent. The absence of dolomite or calcite in certain parts may be due to low pH groundwater. Abundant iron cement is also reported in some areas near Calvert Cliffs, with significant accumulation of shells that had dissolved. The degree of cementation is affected by the level of dolomitization in the sandy soils, a process that began in the Chesapeake Groups soils once they were covered by the clayey soils above.

The abundant shells in some zones within this stratum renders these zones very porous. In a few borings, loss of drilling fluid was noted, e.g., in borings B-302, B-406, B-414, B-426, B-703, and B-710. These porous zones were encountered either near the upper or the lower part of the stratum. Fluid loss was estimated to be in the range of 300 to 600 gallons at each of the 400-series borings. The loss was judged to be due to the nested accumulation of coarse materials, particularly shell fragments at these locations. The fluid loss in boring B-309, and in the upper portion of boring B-710, was in suspected fill materials.

Index tests and testing for the determination of engineering properties were performed on selected samples from Stratum IIb soils. Sample selection for testing was primarily based on the observed soil uniformity from the field classification, or conversely, the variation in material description based on logging in the field, in order to obtain a quantitative measure of the uniformity, or the variation, respectively. The following index tests were performed on Stratum IIb soils, with results as noted.

	No. of Tests	Min. Value	Max. Value	Average Value
Water Content (WC)(%)	67	26	88	34
Liquid Limit (LL)(%)	67	Non-Plastic (NP)	78	46
Plastic Limit (PL)(%)	67	NP	52	24
Plasticity Index (PI)	67	NP	43	22
Fines Content (%)	115	3	71	24
Unit Weight (pcf)	16	115	124	118

The test results are summarized in Table 2.5.4-8. The water content and Atterberg limits are presented versus elevation in Figure 2.5.4-12. They are also shown on the plasticity chart in Figure 2.5.4-13. Grain size analyses indicated that Stratum IIb soils are primarily medium-fine sands. The USCS designations were silty sand, poorly-graded sand to silty sand, clayey sand, silt, silt of high plasticity, clay of high plasticity, clay, and organic clay. The predominant classifications, however, were silty sand, clayey sand, and poorly-graded sand to silty sand (SM, SC, and SP-SM). Three samples were classified as organic clay or organic silt, although evidence of high organic content was not present during the field exploration. The organic designation was based on laboratory (liquid limit) testing. Follow up organic content testing on one sample indicated an organic content of 3.2 percent. Despite the presence of organic matter in this sample, Stratum IIb soils are not considered organic soils since organic materials are virtually absent in these soils. The plastic and fine-grained soil classifications are generally from the clayey/silty interbeds within this stratum. For engineering analysis purposes, and given the predominance of granular proportions, Stratum IIb soils were characterized, on average, as sands with low plasticity, and with an average fines content of 20 percent. Based on laboratory test results, an average unit weight of 120 pcf was also adopted for engineering purposes.

The shear strength of Stratum IIb soils was evaluated based upon laboratory testing and correlations with SPT N-values and CPT results. Initially, the angle of shearing resistance of the soils was estimated from an empirical correlation with SPT N-values (Bowles, 1996). Using the SPT N-value adjusted for hammer efficiency, a ϕ' of about 50 degrees is obtained for N = 45 blows/ft for sands. A value of ϕ'=40 degrees was conservatively considered. Friction angle values were also obtained from the CPT results, despite limited success penetrating these soils in entirety with the CPT. Estimates of friction angle using the method recommended in EPRI EL-6800 (EPRI, 1990) are presented versus elevation in Figure 2.5.4-14. The estimated values range from 28 degrees to 49 degrees, with an average value of 39 degrees. Three direct shear tests were performed on samples of Stratum IIb soils designated as organic silt and clayey sand by the USCS classification, resulting in average ϕ'=31 degrees and c'=0.4 tsf. The laboratory strength results are given in Table 2.5.4-9. Strength parameters from three CIU-bar tests classified as organic clay, poorly-graded sand to silty sand, and silty sand, indicated average (effective) ϕ'=31 degrees and c'=0.5 tsf and average (total) ϕ=16 degrees and c=1.7 tsf. From the above results, the following is a summary of strength parameters for Stratum IIb soils based on various data and interpretation.

	SPT	CPT	Direct Shear	CIU-bar
ϕ' (degrees)	40	39	31	31
c' (tsf)	0	0	0.4	0.5
ϕ (degrees)	---	---	---	16
c (tsf)	---	---	---	1.7

The direct shear and CIU-bar results are comparable, as are values interpreted from the SPT and CPT data. Based on the above, ϕ'=34 degrees and c'=0 is adopted for Stratum IIb soils.

Consolidation properties and stress history of Stratum IIb soils were evaluated via laboratory testing and evaluation of the CPT data. A summary of the laboratory consolidation test results is presented in Table 2.5.4-10. The laboratory results are also plotted versus elevation and shown in Figure 2.5.4-15. Results indicate that, on average, these soils are preconsolidated to 9 tsf, with an OCR of at least 5. OCR data were derived from the CPT results and are shown in Figure 2.5.4-16. The results are scattered over a large range, from OCR=0.8 to OCR=10, with no unique trend. At best, an average OCR may be discerned from the CPT data in Figure 2.5.4-16, or an approximate OCR of 7. A summary of OCR values from CPT data is shown in Table 2.5.4-10. An OCR=3 and a preconsolidation pressure of 8 tsf were conservatively adopted for Stratum IIb soils.

The elastic modulus, E, of Stratum IIb soils was evaluated using the relationship in Davie (Davie, 1988), and Eq. 2.5.4-1. Using the previously established N-value of 45 blows/ft, an elastic modulus of 810 tsf is estimated for these soils. Also, an elastic modulus was estimated based on shear wave velocity for sandy soils (Senapathy, 2001), and Eqs. 2.5.4-2 through

2.5.4-4. Using an average Vs=1530 ft/sec obtained from the measurements at the site (refer to Section 2.5.4.4 for discussions on this topic), γ=120 pcf, and assuming μ=0.3 for sand, a modulus of elasticity of 1,134 tsf is estimated from Eq. 2.5.4-2. Using an average of the two estimates from SPT and shear wave velocity, an elastic modulus of 972 tsf is estimated. A value of 970 tsf was adopted for Stratum IIb soils. The values are shown in Table 2.5.4-11.

The static shear modulus, G, was estimated using Eq. 2.5.4-5. Using a Poisson's ratio of 0.3 for sandy soils, a shear modulus of 373 tsf is estimated for these soils. A value of 436 tsf was estimated using Eq. 2.5.4-3. Using an average of the two estimates, a value of 400 tsf was adopted for Stratum IIb soils. The values are shown in Table 2.5.4-11.

The coefficient of subgrade reaction for 1-ft wide or 1-ft square footings, k_1, was obtained from Terzaghi (Terzaghi, 1955). Based on material characterization for Stratum IIb soils, k_1 = 300 tcf was estimated and adopted.

Active, passive, and at rest static earth pressure coefficients, K_a, K_p, and K_0, respectively, were estimated assuming frictionless vertical walls and horizontal backfill using Rankine's Theory (Lambe, 1969), Eqs. 2.5.4-6 through 2.5.4-8, and the adopted ϕ'=34 degrees for Stratum IIb soils. The estimated earth pressure coefficients are K_a=0.28, K_p=3.5, and K_0=0.44. Values adopted for engineering purposes are K_a=0.3, K_p=3.5, and K_0=0.5.

The sliding coefficient, tangent δ, for Stratum IIb soils in contact with concrete was estimated based on data in "Foundations and Earth Structures" (NFEC, 1986). Tangent δ=0.45 was adopted for Stratum IIb soils.

All of the material properties adopted for engineering purposes for Stratum IIb soils, as well as other useful information, are summarized in Table 2.5.4-12.

Stratum IIc – Chesapeake Clay/Silt

Underlying the cemented soils, another Chesapeake Clay/Silt stratum was encountered, although distinctly different from the one above the cemented soils. This stratum was encountered in all borings that were sufficiently deep to encounter these soils within the CCNPP Unit 3 powerblock and CLA1 areas. Although primarily gray to greenish gray clay/silt soils, they contain interbedded layers of sandy silt, silty sand, and cemented sands with varying amounts of shell fragments. The greenish tone is the result of glauconite in these soils. Glauconite is a silicate mineral of greenish color with relatively high iron content (about 20 percent). Galuconite oxidizes on contact with air, producing a dark color tone. It is normally found as sand-size, dark green nodules. It can precipitate directly from marine waters or develop as a result of decaying of organic matter in animal shells or bottom-dwellers.

The thickness of Stratum IIc soils was estimated from the boring logs. Only two borings, B-301 and B-401, were sufficiently deep to completely penetrate this stratum. Based on borings B-301 and B-401, the thickness of this stratum is estimated as 190 ft, as shown in Table 2.5.4-2.

The stratum thickness was based on estimating the termination elevations encountered for the layer at the boring locations. In Unit 3 area, the termination elevation of Stratum IIc soils was estimated at elevation -208 ft, whereas in CLA1 area it was estimated at elevation -211 ft, or an average elevation -209 ft, as shown in Table 2.5.4-3. An elevation of -200 ft was adopted for simplicity.

Soil samples were obtained from the borings via SPT and tube samples. SPT N-values were measured during the sampling and recorded on the boring logs. In the CCNPP Unit 3 area, the SPT N-values ranged from 12 to greater than 100 blows/ft, with an average N-value of 23 blows/ft. In the adjacent CLA1 area, the SPT N-values ranged from 10 to 39 blows/ft, with an average N-value of 20 blows/ft. The combined average SPT N-value is 21 blows/ft. Based on SPT N-values, Stratum IIc soils are considered very stiff on average. Additional SPT information on this layer is presented in Table 2.5.4-4. The measured N-values versus elevation are presented in Figure 2.5.4-10. They indicate a relatively uniform trend in SPT N-value with depth in the upper half and an increasing trend in the lower half of the profile. It also indicates lateral uniformity in SPT N-values across the CCNPP Unit 3 and CLA1 areas to be within a narrow range, as also evident from the average values in the two areas. Evidences of intermittent cementation, or otherwise hardened zones, are also indicated by increasing SPT N-values at intermittent elevations, e.g., near elevation -40, elevation -110, and elevation -170 ft.

The SPT N-values were adjusted for hammer energy; the adjusted average field-measured N-value for Stratum IIc soils is 29 blows/ft. A value of 25 blows/ft was conservatively adopted for engineering purposes, as shown in Table 2.5.4-6.

CPT soundings were attempted in Stratum IIc soils, following several attempts to penetrate these soils due to persistent refusal in overlying soils. A profile of q_c versus elevation is shown in Figure 2.5.4-11. The results suggest relative uniformity in q_c values with depth and lateral extent, as well as evidence of cemented (or hardened zones) near elevation -40 ft which was similarly reflected in the SPT N-value profile in Figure 2.5.4-10. The q_c values range from about 50 to 100 tsf, with an average of about 75 tsf.

Index tests and testing for determination of engineering properties were performed on selected samples from Stratum IIc soils. Sample selection for testing was primarily based on the observed soil uniformity from the field classification, or conversely, the variation in material description based on logging in the field, in order to obtain a quantitative measure of the uniformity, or the variation, respectively. The following index tests were performed on Stratum IIc soils, with results as noted.

	No. of Tests	Min. Value	Max. Value	Average Value
Water Content (WC) (%)	88	26	123	54
Liquid Limit (LL) (%)	88	39	218	94
Plastic Limit (PL) (%)	88	30	100	50
Plasticity Index (PI)	88	9	118	44
Fines Content (%)	82	18	100	54
Unit Weight (pcf)	19	86	117	107

The test results are summarized in Table 2.5.4-8. The water content and Atterberg limits are presented versus elevation in Figure 2.5.4-12. They are also shown on the plasticity chart in Figure 2.5.4-13. For engineering analysis purposes, Stratum IIc soils were characterized, on average, as high plasticity clay and silt, with an average PI=45. Their predominant USCS designation was clay of high plasticity and silt of high plasticity (CH and MH), however, sometimes with silty sand, clay, and organic clay classifications indicated. Based on field observations during sampling, the organic designation based on laboratory (Liquid Limit) testing is not representative of these soils, and therefore, they are not considered organic soils. The organic designation (based on Liquid Limit tests) may be impacted by the glauconite content in the soils. Based on laboratory testing, an average unit weight of 110 pcf was also adopted for Stratum IIc soils for engineering purposes.

The shear strength of Stratum IIc soils was evaluated based on laboratory testing, and using correlations with SPT N-values and the CPT results. The results are summarized in Table 2.5.4-13.

The undrained shear strength, s_u, was estimated from Eq. 2.5.4-9 based on SPT N-values. Substituting the previously established N-value for Stratum IIc soils (SPT N-value=25), s_u=1.6 tsf is estimated for these soils. Undrained shear strength was also estimated using the CPT data, following a CPT-s_u correlation from Robertson (Robertson, 1988), using a cone factor N_{kt}=15. The shear strength values obtained from the CPT data are shown versus elevation in Figure 2.5.4-18, indicating an average s_u=4.7 tsf as summarized in Table 2.5.4-13. A number of laboratory unconsolidated undrained (UU) triaxial and unconfined compression (UC) tests were performed on selected undisturbed samples. Laboratory test results on 10 samples resulted in an average s_u=2.2 tsf. The laboratory shear strength test results are shown versus elevation in Figure 2.5.4-17. Based on these results, an average undrained shear strength of 2.0 tsf was conservatively adopted for Stratum IIc soils.

The angle of shearing resistance of these soils was evaluated from laboratory test results. The results are shown in Table 2.5.4-9. Four direct shear tests were performed on samples of Stratum IIc soils, designated as clay, clay of high plasticity, and clayey sand by the USCS soil classification system, resulting in an average ϕ'=25 degrees and c'=1.6 tsf. Strength parameters from one CIU-bar test, indicated effective stress ϕ'=29.1 degrees, c'=1.0 tsf, and total stress ϕ=15.4 degrees, and c=1.5 tsf. From the above, the following is a summary of average ϕ' values for the Stratum IIc soils based on various data and interpretation.

	Direct Shear	CIU-bar
ϕ' (degrees)	25	29.1
c' (tsf)	1.6	1.0
ϕ (degrees)	---	15.4
c (tsf)	---	1.5

Based on the above, ϕ'=27 degrees and c'=1.0 tsf is adopted for Stratum IIc soils.

Consolidation properties and stress history of Stratum IIc soils were evaluated via laboratory testing and evaluation of the CPT data. A summary of the laboratory consolidation test results is presented in Table 2.5.4-10. The laboratory results are also plotted versus elevation and shown in Figure 2.5.4-15. Results indicate that, on average, these soils are preconsolidated to about 15 tsf, with an OCR of at least 3. OCR data derived from CPT results are shown in Figure 2.5.4-16. The CPT-derived results are scattered over a large range, from about OCR=1.2 to OCR=10, with no unique trend, although most values are in the range of about 5 to 10. An average OCR from the CPT data would be approximately 9. A summary of OCR values from CPT data is shown in Table 2.5.4-10. An OCR=3 and preconsolidation pressure of 14 tsf were conservatively adopted for Stratum IIc soils. It is noted that this preconsolidation pressure is equivalent to about 200 to 300 ft of preloading by sediments that once covered these soils during prehistoric times. This is consistent with a study on the depositional history of Miocene-age soils in Maryland (Rosen, 1986) that estimated the burial depth of these soils in Western Maryland, e.g., Calvert County, at "much less" than 590 ft, which would be equivalent to about 200 to 300 ft assuming one-third to one-half of the referenced burial depth.

Static modulus of elasticity for Stratum IIc was evaluated using Eq. 2.5.4-12. For the adopted s_u=2 tsf, an elastic modulus of 900 tsf is estimated. Also, elastic modulus was estimated based on shear wave velocity from Eqs. 2.5.4-13 and 2.5.4-14. Using an average Vs=1,250 ft/sec obtained from the measurements at the site (refer to Section 2.5.4.4 for discussions on this topic), unit weight of 110 pcf, and assuming Poisson's ratio 0.45 for clayey soils, a modulus of elasticity of 2,477 tsf is estimated from Eq. 2.5.4-13. Using s_u=2.0 tsf, an elastic modulus of 1,160 tsf is estimated from Eq. 2.5.4-14. Of the preceding estimates, the value based on PI appears high. Therefore, the PI-based value is conservatively omitted when estimating an average elastic modulus for Stratum IIc soils. Using an average of the estimated values from undrained strength and shear wave velocity, an elastic modulus of 1,030 tsf is estimated and adopted for Stratum IIc soils, as shown in Table 2.5.4-11.

The static shear modulus, G, was estimated using Eq. 2.5.4-5. Using μ=0.45 for clay soils, a shear modulus of 355 tsf is estimated for these soils. Values of 853 and 400 tsf were estimated using Eqs. 2.5.4-13 and 2.5.4.-14. The higher value was ignored for conservatism. An average of the two other values, 370 tsf, was conservatively adopted for Stratum IIc soils, as shown in Table 2.5.4-11.

The coefficient of subgrade reaction for 1-ft wide or 1-ft square footings, k_1, was obtained from Tezaghi (Terzaghi, 1955). Based on material characterization for Stratum IIc soils, k_1=150 tcf was estimated and adopted.

Active, passive, and at rest static earth pressure coefficients, K_a, K_p, and K_0, respectively, were estimated assuming frictionless vertical walls and horizontal backfill using Rankine's Theory, Eqs. 2.5.4-6 through 2.5.4-8, and the adopted ϕ'=27 degrees for Stratum IIc soils, the following earth pressures coefficients are estimated; K_a=0.4, K_p=2.6, and K_0=0.55. Given the overconsolidated nature of the soils, the K_0 value was increased. The adopted values for engineering purposes are K_a=0.4, K =2.6, and K_0=0.7.

The sliding coefficient, tangent δ, of 0.40 was adopted for Stratum IIc soils in contact with concrete (NFEC, 1986).

All of the material properties adopted for engineering purposes for Stratum IIc soils, as well as other useful information, are summarized in Table 2.5.4-12.

2.5.4.2.1.3 Stratum III – Naniemoy Sand

Underlying the Chesapeake Clay/Silt stratum are the Nanjemoy soils (Stratum III). Stratum III was encountered in deep borings B-301 and B-401. This stratum consists primarily of dark, greenish-gray glauconitic sand, however, it contains interbedded layers of silt, clay, and cemented sands with varying amounts of shell fragments and varying degrees of cementation. The glauconite in these soils could vary from less than 10 percent to as much as 50 percent.

The thickness of Stratum III soils cannot be estimated from the information obtained from the CCNPP Unit 3 subsurface investigation (boring logs B-301 and B-401), as these borings did not penetrate these soils in their entirety, although they penetrated them by about 100 ft. The Nanjemoy soils are about 200 ft thick at the site (Hansen, 1996), consisting of primarily sandy soils in the upper 100 ft and clayey soils in the lower 100 ft. On this basis, the termination (bottom) of the upper sandy portion can be estimated at about elevation -315 ft and the termination of the lower clayey portion can be estimated at about elevation -415 ft. Information from borings B-301 and B-401 sufficiently characterizes the upper half of this geologic unit, as these borings were terminated at elevation -308 ft and elevation -329 ft, respectively.

Soil samples were collected from the borings via SPT sampling. Only one tube sample was collected in these soils, however, despite several attempts, given the depth and penetration difficulties involved. SPT N-values were measured during the sampling and recorded on the boring logs. In the CCNPP Unit 3 area, the SPT N-values ranged from 34 blows/ft to greater than 100 blows/ft, with an average N-value of 64 blows/ft. In the adjacent CLA1 area, the SPT N-values ranged from 28 blows/ft to greater than 100 blows/ft, with an average N-value of 56 blows/ft. The combined average SPT N-value is 61 blows/ft. Based on SPT N-values, Stratum III soils are considered very dense on average. The SPT information is presented in Table 2.5.4-4. The measured N-values versus elevation are presented in Figure 2.5.4-10. They indicate a generally increasing trend in SPT N-value with depth, although SPT N-values begin to decline near the bottom of the explored depth, a possible indication of nearing the underlying clay soils. Limited SPT values are available from this stratum to judge its lateral uniformity, however, most available data appear to fall in a relatively narrow range, except for intermittent "peak" values. The peak SPT N-values are likely due to the presence of cemented or otherwise hardened zones. CPT sounding could not reach these soils due to refusal in overlying soils.

The SPT N-values were adjusted for hammer energy; the adjusted average field-measured N-value for Stratum III soils is 72 blows/ft. A value of 70 blows/ft was conservatively adopted for engineering purposes, as shown in Table 2.5.4-6.

Index tests were performed on several samples from Stratum III soils. Sample selection for testing was primarily based on the observed soil uniformity from the field classification, or conversely, the variation in material description based on logging in the field, in order to obtain a quantitative measure of the uniformity, or the variation, respectively. Due to the limited quantity of available samples, the testing was limited. The following index tests were performed on selected samples of Stratum III soils, with the results as noted.

	No. of Tests	Min. Value	Max. Value	Average Value
Water Content (WC) (%)	7	23	37	30
Liquid Limit (LL)(%)	7	47	76	59
Plastic Limit (PL)(%)	7	32	40	32
Plasticity Index (PI)	7	15	36	27
Fines Content (%)	10	12	29	19

The test results are summarized in Table 2.5.4-8. The water content and Atterberg limits are presented versus elevation in Figure 2.5.4-12. They are also shown on the plasticity chart in Figure 2.5.4-13. For engineering analysis purposes, Stratum III soils were characterized, on average, as sand of high plasticity, with an average PI=30. Their predominant USCS designations were clayey sand and silty sand (SC and SM), although clay of high plasticity and silt of high plasticity were also indicated. Testing for unit weight was not performed since only disturbed SPT samples could be obtained from this stratum; however, based on correlation with SPT N-values (Bowles, 1996), an average unit weight of 120 pcf was adopted for these soils.

The shear strength of Stratum III soils was evaluated using correlations with SPT N-values, assuming predominately granular behavior. For an average SPT N-value=70 blows/ft, an average ϕ'=50 degrees is estimated (Bowles, 1996). A ϕ'=40 degrees was conservatively adopted.

Given the relatively high plasticity in these soils (LL=60 and PI=30 on average), their behavior could also be characterized using undrained parameters. Although no laboratory strength tests were performed on these soils, their undrained shear strength, s_u, may be estimated from Eq. 2.5.4-9. For an average N-value=70 blows/ft, s_u=4.4 tsf is estimated for these soils. An undrained shear strength of 4.0 tsf may conservatively be assigned to Stratum III soils as summarized in Table 2.5.4-13.

Given the high SPT N-value, and associated strength, Stratum III soils are considered highly preconsolidated. Although no consolidation or CPT tests were performed in these soils, their preconsolidation pressure is judged to be at least as high as the overlying soils (a preconsolidation pressure of 14 tsf was assigned to the overlying Stratum IIc soils). The high degree of preconsolidation is evident by the indices that were measured, e.g., a profile of the water content versus elevation in Figure 2.5.4-12 clearly demonstrates the water contents to be consistently near the Plastic Limit, a strong indication of high preconsolidation in these soils.

Static modulus of elasticity for Stratum III soils was evaluated using Eq. 2.5.4-1. For the adopted SPT N-value=70 blows/ft, an elastic modulus of 1,260 tsf is estimated. Similarly, Eqs. 2.5.4-12 through 2.5.4-14 were utilized, along with corresponding parameters previously noted, and elastic modulus values of 1,800, 1,879, and 2,080 tsf were estimated, as noted in Table 2.5.4-11. A value of 1,750 tsf is estimated and adopted for Stratum III soils.

The static shear modulus, G, was estimated using Eq. 2.5.4-5. Using μ=0.3 for sandy soils, a shear modulus of 700 tsf is estimated and adopted for these soils, as shown in Table 2.5.4-11.

Foundations are not anticipated in Stratum III soils, therefore, estimating their coefficient of subgrade reaction, earth pressure, and sliding coefficient is unnecessary.

All of the material properties adopted for engineering purposes for Stratum III, as wells as other information, are summarized in Table 2.5.4-12.

2.5.4.2.1.4 Chemical Properties of Soils

Chemical laboratory tests were performed on selected soil and groundwater samples. The groundwater test results, and soil portions tested as part of the groundwater characterization, are addressed in Section 2.4.13. A brief summary of available information is evaluated and provided below.

Chemical Testing for CCNPP Units 1 and 2

Chemical test results on soils are available in a report that was prepared as part of the design of an additional Diesel Generator Building (Bechtel, 1992) at the project site. Three samples from each investigated stratum were tested, for pH, sulfate, and chloride. A summary of the results is presented in Table 2.5.4-14.

Chemical Testing on CCNPP Unit 3 Samples

Field electrical resistivity tests were performed along four arrays, at locations shown in Figures 2.5.4-1 and 2.5.4-2. The results are presented in Appendix 2.5-A, and summarized in Table 2.5.4-15. The results are approximately correlated with depth based on the array spacing, as shown in Table 2.5.4-15.

Field Electrical Resistivity Testing for COL Investigation

Field electrical resistivity tests were performed along four arrays, at locations shown in Figures 2.5.4-1 and 2.5.4-2. The results are presented in Appendix 2.5-A, and summarized in Table 2.5.4-15. The results are approximately correlated with depth based on the array spacing, as shown in Table 2.5.4-15.

Evaluation of Chemical Data

Guidelines for interpretation of chemical test results are provided in Table 2.5.4-16, based on the following consensus standards, API Recommended Practice 651 (API, 2007), Reinforced Soil Structures (FWHA, 1990), Standard Specification for Portland Cement (ASTM C150), Manual of Concrete Practice (ACI, 1994), and Standard Specification for Blended Hydraulic Cement (ASTM, C595). From the average values of available results shown in Tables 2.5.4-14 and 2.5.4-15, and guidelines in Table 2.5.4-16, the following conclusions were developed.

Attack on Steel (Corrosiveness): The resistivity test results indicate that all soils are "little corrosive," except for Stratum IIc Chesapeake Clay/Silt that may be "little to mildly corrosive." Based on the chloride contents being typically below 10 ppm, all soils are essentially non-corrosive. The pH results, however, indicate that all soils are "corrosive to very corrosive," except for Stratum IIc Chesapeake Clay/Silt that may be "mildly corrosive." It is noted that few chemical test results are available from Stratum IIc; however, that should be of no special importance because no Category I structure (or piping) is anticipated within these soils. The pH data dominate the corrosive characterization of the soils. Nevertheless, all natural soils at the site will be considered corrosive to metals, requiring protection if placed within these soils. Protection of steel against corrosion may include cathodic protection, or other measures, which will be determined during the detailed design phase of the project. It should be noted that additional pH testing on groundwater samples obtained from the observation wells (refer to Section 2.4.13) indicate pH values of average 5.5, 6.8, and 7.1 for wells screened in Stratum I, Stratum IIa, and Stratum IIb soils, respectively. Except for values obtained in groundwater associated with Stratum I soils indicating "corrosive" conditions, remaining pH data from other strata only indicate "mildly corrosive" conditions.

Attack on Concrete (Aggressiveness): The sulfate test results in all tested soils indicate a "severe" potential for attack on concrete, except for Stratum IIc Chesapeake Clay/Silt that may cause a "moderate" attack. As noted above, few chemical test results are available for Stratum IIc; however, that should be of no special importance because no Category I structure (or piping) is anticipated within these soils. Nevertheless, all natural soils at the site will be considered aggressive to concrete, requiring protection if placed within these soils. Protection of concrete from sulfate attack will be addressed during the detailed design phase of the project. The incorporation of sacrificial concrete thickness, whose compatibility may be verified by bench scale laboratory testing, is one of the measures that will be considered during the detailed design phase of the project.

2.5.4.2.1.5 Subsurface Materials Below 400 Feet

As indicated earlier, the field exploration for the CCNPP Unit 3 extended to a maximum depth of about 400 ft below ground. Coastal Plain sediments, however, are known to extend below this depth, to a depth of approximately 2,500 ft, or to top of bedrock (BGE, 1982). The subsurface conditions below 400 ft were addressed through reference to existing literature and work that had been done by others, primarily for the purpose of seismic site characterization. The subsurface conditions below 400 ft are addressed in Sections 2.5.4.7 and 2.5.2.5.

2.5.4.2.1.6 Field Investigation Program

The planning of the field investigation referred to the guidance provided in NRC Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants" (NRC, 2003). References to the industry standards used for field tests completed for the CCNPP Unit 3 subsurface investigation are shown in Table 2.5.4-1. The details and results of the field investigation are provided in Geotechnical Subsurface Investigation Data Report (Schnabel, 2007) and included as Appendix 2.5-A. The work was performed under the Bechtel QA program with work procedures developed specifically for the CCNPP Unit 3 subsurface investigation, including a subsurface investigation plan developed by Bechtel The locations of borings in Figures 2.5.4-1 and 2.5.4-2, although in agreement with the guidance in Regulatory Guide 1.132 (NRC, 2003a) at the time of developing the subsurface investigation plan, do not agree with the guidance of Regulatory Guide 1.132 (NRC, 2003a), for the current CCNPP Unit 3 layout since the layout has evolved over time and the locations of some of the structures have shifted. This has resulted in borings or CPT soundings being outside the outline of some structures. Although differing soil conditions are not expected, due to the observed lateral stratigraphic uniformity at the site, a complementary investigation will be performed as part of the detailed design of the project, with reference to guidance in Regulatory Guide 1.132 (NRC, 2003a) to verify subsurface uniformity at these locations. If this additional investigation yields nonconservative results that impact the conclusions of this section, an update to the COL application will be made.

2.5.4.2.1.7 Laboratory Testing Program

The laboratory investigations of soils and rock was performed with in accordance with the guidance outlined in Regulatory Guide 1.138, Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants (NRC, 2003b). Deviations are identified, alternatives and/or basis for deviation are provided.

A summary, as well as detailed results, of all laboratory tests performed as part of the subsurface investigation is provided in Geotechnical Subsurface Investigation Data Report (Schnabel, 2007), included as Appendix 2.5-A.

The laboratory work was performed under the Bechtel QA program with work procedures developed specifically for the CCNPP Unit 3 subsurface investigation.

Soil samples were shipped under chain-of-custody protection from the on-site storage to the testing laboratories. ASTM D4220 (ASTM, 2000a) provides guidance on standard practices for preserving and transporting soil samples. This guidance was referenced in preparing technical specifications for the CCNPP Unit 3 subsurface investigation, addressing sample preservation and transportation, as well as other subsurface investigation and geotechnical requirements.

Laboratory testing consisted of testing soils and groundwater samples obtained from the investigation program. Testing of groundwater samples is addressed is Section 2.4.13. Laboratory testing of soil samples consisted of index and engineering property tests on selected SPT, undisturbed, and bulk samples. The SPT and undisturbed samples were recovered from the borings and the bulk samples were obtained from the test pits. Soil laboratory tests included the following: water content, grain size (sieve and hydrometer), Atterberg limits, organic content, chemical analysis (pH, chloride, and sulfate), unit weight, specific gravity, moisture-density, consolidation, unconfined compression (UC), unconsolidated-undrained triaxial compression (UU), consolidated-undrained triaxial compression (CIU-bar), direct shear (DS), and resonant column torsional shear (RCTS) testing.

Regulatory Guide 1.138 (NRC, 2003b) provides guidance for laboratory testing procedures for certain specific tests, including related references. Some of these references are not in common practice in the U.S. or are out-of-date. Laboratory testing of samples for the CCNPP Unit 3 subsurface investigation used commonly accepted, and updated practices such as more recent ASTM and EPA standards which are equivalent to the testing procedures referenced in the Regulatory Guide. Laboratory testing of samples for the CCNPP Unit 3 subsurface investigation did not rely upon non-U.S. or out-of-date versions of practices or standards provided in the Regulatory Guide. References to the industry standards used for this laboratory investigation, standards delineated in Regulatory Guide 1.138 (NRC, 2003b), and quantity of test are shown in Table 2.5.4-7.

One laboratory is currently performing the Resonant Column Torsional Shear (RCTS) testing, with results expected to become available at a later date (as discussed further in Section 2.5.4.7). All other laboratories have completed their testing, with the results contained in Geotechnical Subsurface Investigation Data Report (Schnabel, 2007), included as Appendix 2.5-A.

The soil and rock laboratory tests listed in Regulatory Guide 1.138 (NRC,2003b) are common tests performed in most well-equipped soil and rock testing laboratories, and they are covered by ASTM standards. Additional test that are not covered in regulatory guidance were also performed for the CCNPP Unit 3 subsurface investigation (e.g., CBR tests to assess suitability of subgrade or fill materials for pavement, and RCTS tests, which will be used in lieu of the resonant column test alone to obtain shear modulus and damping ratio values for a wide range of strains). Results of Cation Exchange Capacity tests are addressed with the groundwater chemistry data in Section 2.4.13.

2.5.4.2.1.8 Investigations

<u>Previous Subsurface Investigations</u>

Based on limited information available from the CCNPP Units 1 and 2 UFSAR (BGE, 1982), the original subsurface investigations for the CCNPP Units 1 and 2 performed in 1967 consisted of a total of 10 exploratory borings, ranging in depth from 146 to 332 ft, with soil samples obtained at various intervals for soil identification and testing. Seven piezometers were also installed for groundwater observation and monitoring. The 1967 investigation included other field investigations (two seismic survey lines using Microtremor) and laboratory testing (moisture content, density, particle size, permeability, cation exchange, and x-ray diffraction). Supplemental investigations in support of detailed design were performed in July 1967 (5 borings), August 1967 (23 borings), December 1968 (18 borings), and 1969 (5 borings). Additional investigations were performed in 1980/1981 (borings, CPT soundings, and observation wells) in order to site a "generic Category I structure," and in 1992 additional investigations (borings, dilatometer soundings, crosshole seismic survey, field resistivity) were performed for an additional Diesel Generator Building. Various laboratory testing was also performed on selected portions of the recovered soils.

Geological descriptions in CCNPP Units 1 and 2 UFSAR (BGE, 1982) indicate the surficial deposits to be Pleistocene Age soils extending from the ground surface to about elevation 70 ft. These soils were estimated to extend to an average elevation 60 ft based on the CCNPP subsurface investigation. CCNPP Units 1 and 2 UFSAR (BGE, 1982) indicates that Chesapeake Group soils were encountered in the 1967 investigation between elevation 70 ft and elevation -200 ft. These soils were estimated to extend to approximately elevation -200 ft based on the COL investigation. CCNPP Units 1 and 2 UFSAR (BGE, 1982) indicates that Eocene deposits lie below elevation -200 ft and consist of glauconitic sands. Comparable observations were made on these, and the overlying deposits, from the CCNPP subsurface investigation borings. CCNPP Units 1 and 2 UFSAR (BGE, 1982) remarked that "good correlation of subsurface stratigraphy was obtained between the borings." This remark is corroborated by the results obtained from the CCNPP subsurface investigation.

It is noted that the CCNPP Unit subsurface investigation involved a significantly larger quantity of testing than performed for the original CCNPP Units 1 and 2. Given the reasonably parallel geologic conditions between CCNPP Units 1 and 2 and the CCNPP Unit 3 site, and the greater intensity in exploration and testing at the CCNPP Unit 3 site which should result in enhanced characterization of the subsurface conditions, findings from previous investigations are not discussed further, unless a differing condition is reported from the previous investigations.

CCNPP Unit 3 Subsurface Investigation

The subsurface investigation program was performed with in accordance with the guidance outlined in Regulatory Guide 1.132 (NRC, 2003a). Deviations are identified at point of use, alternatives and/or basis for deviation are provided. The fieldwork was performed under the contractors QA program and work procedures developed specifically for the CCNPP Unit 3 subsurface investigation.

Regulatory Guide 1.132 (NRC, 2003a) provides guidance on spacing and depth of borings, sampling procedures, in-situ testing, geophysical investigations, etc. This guidance was used in preparing a technical specification, addressing the basis for the CCNPP Unit 3 subsurface investigation. The quantity of borings and CPTs for Category I structures was based on a minimum of one boring per structure and the one boring per 10,000-square ft criterion. The maximum depths of the borings for Category I structures were based on a foundation to overburden stress ratio criterion of 10 percent. The sampling intervals typically exceeded the guidance document by shortening the sample spacing in the upper 15 ft and maintaining 5-ft sampling intervals at depths greater than 50 ft, except for the case of 400-ft borings. Continuous sampling was also performed, and will be described later.

Regulatory Guide 1.132 (NRC, 2003a) provides guidance in selecting the boring depth, d_{max}, based on a foundation to overburden stress ratio of 10 percent. Using this criterion, a boring depth of approximately 350 ft was determined for the most heavily loaded structures supported on the Common Basemat. Regulatory Guide 1.132 (NRC, 2003a), also indicates that at least one-fourth of the principal borings should penetrate to a depth equal to $d_{max.}$ Given the previously available knowledge of subsurface conditions as documented in the CCNPP Units 1 and 2 UFSAR (BGE, 1982) indicating stable, geologically old deposits at the site which would not adversely impact foundation stability, it was determined that one boring should be extended to about 400 ft, 4 borings extended to about 200 ft, and 4 borings extended to about 150 ft for the Common Basemat. (The consistency across the site of the Miocene-age Chesapeake Group clays and silts that exist below about 100 ft depth and the underlying Nanjemoy Formation sands that start at around 300 ft depth is aptly demonstrated by the similarity of the shear wave velocity profiles obtained in boreholes almost 1,000 ft apart (Figure 2.5.4-22)). Also included were 3 CPT soundings. Borings associated with the Common Basemat extended at least 33 ft below the foundation level. Additional boring are to be taken to meet the Regulatory Guide 1.132 guidance during detailed design.

As noted in subsection 2.5.4.2.1.6, the current quantity and locations of tests, shown in Figures 2.5.4-1 and 2.5.4-2, do not necessarily coincide with the footprint of structures, for the current CCNPP Unit 3 layout has evolved since the investigation, as well as the need during the field work to relocate the tests to locations that avoided wetlands, reduced cutting trees, and were accessible to the drilling equipment. Although a differing subsurface condition is not anticipated due to the observed soil uniformity at the site, a complementary investigation will be performed during the detailed design stage to verify subsurface uniformity at these locations.

A team consisting of a geologist, a geotechnical engineer, and a member of the project management performed a site reconnaissance prior to start of the field investigation. The focus of this task was to observe the site and access conditions, locations of borings and wells, and identify potential test relocation areas. Information on site geology and geotechnical conditions, used as a basis for developing the soils investigation plan for the CCNPP subsurface investigation was obtained from the information contained in the CCNPP Units 1 and 2 UFSAR (BGE, 1982).

Regulatory Guide 1.132, (NRC, 2003a) provides that boreholes with depths greater than about 100 ft (30.5 m) should be surveyed for deviation. In lieu of surveying for deviation in boreholes greater than 100 ft (30.5 m), deviation surveys, were used in the 10 suspension P-S velocity logging boreholes to depths ranging from about 200 to 400 ft. The results indicated minimum, maximum, and average deviation of 0.6, 1.6, and 1.0 percent, respectively. The information collected the necessary data for proper characterization of the CCNPP Unit 3 subsurface materials."

Regulatory Guide 1.132, (NRC, 2003a) provides guidance for color photographs of all cores to be taken soon after removal from the borehole to document the condition of the soils at the time of drilling. For soil samples, undisturbed samples are sealed in steel tubes, and cannot be photographed. SPT samples are disturbed, and by definition they do not resemble the condition of the material in-situ. Sample photography is a practice typically limited to rock core samples, not soils, therefore, it was not used. X-ray imaging, however, will be performed on tube samples selected for RCTS testing.

The CCNPP Unit 3 subsurface field exploration was performed from April through August 2006. This work consisted of an extensive investigation to define the subsurface conditions at the project area. The exploration locations are shown in Figures 2.5.4-1 through 2.5.4-3. The scope of work and investigation methods were determined to be as follows:

- Surveying to establish the horizontal and vertical locations of exploration points.
- Evaluating the potential presence of underground utilities at exploration points.
- Drilling 145 test borings with SPT sampling and collecting in excess of 200 undisturbed samples (using Shelby push tubes, Osterberg sampler, and Pitcher sampler) to a maximum depth of 403 ft, including 4 borings with continuous SPT samples (B-305, B-409, B-324, and B-417), with the first two borings being 150 ft deep each and the last two borings being 100 ft deep each. Note that "continuous sampling" was defined as one SPT sample for every 2.5-ft interval with one ft distance between each SPT sample.
- Installing and developing 40 groundwater observation wells to a maximum depth of 122 ft, including Slug testing in each well.
- Excavating 20 test pits to a maximum depth of 10 ft and collecting bulk soil samples.
- Performing 63 CPT soundings, including off-set soundings that required pre-drilling to overcome CPT refusal, to a maximum depth of 142 ft, as well as seismic CPT and pore pressure dissipation measurements.
- Conducting 2-dimensional field electrical resistivity testing along four arrays.
- Performing borehole geophysical logging, consisting of suspension P-S velocity logging, natural gamma, long- and short-term resistivity, spontaneous potential, 3-arm caliper, and directional survey in 10 boreholes.

- Conducting SPT hammer-rod combination energy measurements on 5 drilling rigs.
- Performing laboratory testing of soils, consisting of natural water content, unit weight, specific gravity, sieve and hydrometer analysis, Atterberg limits, organic content, moisture-density, CBR, unconfined compression, consolidated and unconsolidated undrained triaxial compression, direct shear, consolidation, and chemical analysis (pH, sulfate, and chloride). RCTS testing was also initiated and is currently underway. RCTS testing is further discussed in Section 2.5.4.7.3.
- Performing laboratory testing on groundwater samples obtained from the observation wells, consisting of pH, conductivity, dissolved oxygen, alkalinity, ammonia nitrogen, bromide, chloride, dissolved solids, fluoride, nitrate as N, nitrite as N, sulfate, and sulfide, including cation exchange testing on soils in the well screen area. These results are discussed in Section 2.4.13.

The location of each exploration point was investigated for the presence of underground utilities prior to commencing exploration at that location. Locations of several exploration points had to be adjusted due to proximity to utilities, inaccessibility due to terrain conditions, or proximity to wetlands. Access had to be created to most exploration locations, via clearing roads and creating temporary roads, due to heavy brush and forestation. These areas were restored subsequent to completion of the field investigation.

An on-site storage facility for soil samples was established before the exploration program commenced. Each sample was logged into an inventory system. Samples removed from the facility were noted in the inventory logbook. A chain-of-custody form was also completed for all samples removed from the facility. Material storage handling was in accordance with ASTM D4220 (ASTM, 2000a).

Complete results of the investigation are in Appendix 2.5-A. Laboratory test results are discussed and summarized in Section 2.5.4.2. Geophysical test results are discussed and summarized in Section 2.5.4.4. Further details pertaining to field activities related to borings, CPTs, Slug tests, geophysical surveys, and other activities are summarized below.

<u>Test Boring and Sampling</u>

Soils were sampled using the SPT sampler in accordance with ASTM D1586 (ASTM, 1999). The soils were sampled at continuous intervals (one sample every 2.5-ft) to 15 ft depth. Subsequent SPT sampling was performed at regular 5 ft intervals. At boring B-401, with a total depth of 401.5 ft, SPT sampling was performed at about 10 ft intervals below a depth of 300 ft. The recovered soil samples were visually described and classified by the engineer or geologist in accordance with ASTM D2488 (ASTM, 2006d)). A representative portion of the soil sample was placed in a glass jar with a moisture-preserving lid. The sample jars were labeled, placed in boxes, and transported to the on-site storage facility. Table 2.5.4-17 provides a summary of all test borings performed. The boring locations are shown in Figures 2.5.4-1 and 2.5.4-2. The boring logs are included in Appendix 2.5-A. At boring completion, the boreholes were tremie-grouted using cement-bentonite grout.

Undisturbed samples were obtained in accordance with ASTM D1587 (ASTM, 2000c) using the push Shelby tubes, Osterberg sampler, and rotary Pitcher sampler. Upon sample retrieval, the disturbed portions at both ends of the tube were removed, both ends were trimmed square to establish an effective seal, and pocket penetrometer (PP) tests were performed on the trimmed lower end of the samples. Both ends of the sample were then sealed with hot wax, covered with plastic caps, and sealed once again using electrician tape and wax. The tubes were labeled and transported to the on-site storage area. Table 2.5.4-18 provides a summary of undisturbed sampling performed during the subsurface investigation. Undisturbed samples are also identified on the boring logs included in Appendix 2.5-A.

Energy measurements were made on the hammer-rod system on each of the five drilling rigs used in the subsurface investigation. A Pile Driving Analyzer (PDA) was used to acquire and process the data. A summary of measured energies is provided in Table 2.5.4-5. Energy measurements were made at sampling intervals of 15 ft, with the total number of measurements made per boring ranging from 6 (at boring B-744) to 26 (at boring B-401), depending on boring depth. Energy transfer to the gage locations was estimated using the Case Method, in accordance with ASTM D4633 (ASTM, 2005a). The resultant energy transfer efficiency measurements ranged from 78 to 87 percent, with an average energy transfer efficiency of 83 percent. Detailed results are presented in Appendix 2.5-A.

Cone Penetration Testing

CPT soundings were performed using an electronic seismic piezocone compression model, with a 15 cm^2 tip area and a 225 cm^2 friction sleeve area. CPT soundings were performed in accordance with ASTM D5778 (ASTM, 2000e)), except that tolerances for wear of the cone tip were in accordance with report SGF 1:93E, Recommended Standard for Cone Penetration Tests, (SGS, 1993) which are comparable to ASTM. It is noted that for the 10-cm^2 base cone, the ASTM D5778 (ASTM, 2000e) specified dimensions for "base diameter," "cone height," and "extension" are minimum 34.7 mm, 24 mm, and 2 mm, respectively, compared to the report SGF 1:93E (SGS, 1993) recommended tolerances of minimum 34.8 mm, 24 mm, and 2 mm, for the same cone. The 2-mm SGF Report (SGS, 1993) value accounts for a constant 5-mm porous filter. Pore pressures were measured in the soundings. The equipment was mounted on a track-operated rig dedicated only to the CPT work. Cone tip resistance, sleeve friction, and dynamic pore pressure were recorded every 5 cm (approximately every 2 in) as the cone was advanced into the ground. Seismic shear wave velocity tests were also performed using a geophone mounted in the cone, a digital oscilloscope, and a beam, which was struck on the ground surface with a sledge hammer. Pore pressure dissipation data were also obtained, with the data recorded at 5-sec intervals.

A total of 63 CPT soundings were performed, including additional off-set soundings due to persistent refusal in dense/hard or cemented soils. At selected sounding locations, the soils causing refusal were pre-augered so that deeper CPT penetration could be obtained at the sounding location. Pre-augering was performed at six locations, but often several times at the same sounding. The sounding depths ranged from about 12 ft to 142 ft.. Seismic CPT was performed at eight sounding locations. Pore pressure dissipation tests were performed in 20 soundings, at 26 different depths. Table 2.5.4-19 provides a summary of CPT locations and details. The locations are shown in Figures 2.5.4-1 and 2.5.4-2. The CPT logs, shear wave velocity, and pore pressure dissipation results are contained in Appendix 2.5-A.

Observation Wells and Slug Testing

A total of 40 observation wells were installed to a maximum depth of 122 ft during the CCNPP Unit 3 subsurface investigation under the full-time supervision of geotechnical engineers or geologists. Wells were installed either in SPT boreholes or at an off-set location, in accordance with ASTM D5092 (ASTM, 2004a). Wells installed in SPT boreholes were grouted to the bottom of the well, and the portion above was reamed to a diameter of at least 6 in. using rotary methods and biodegradable drilling fluid. Off-set wells were installed using either 6¼-in. ID hollow-stem augers or 6-in. diameter holes using the rotary method and biodegradable drilling fluid. Each well was developed by pumping and/or flushing with clean water. Table 2.5.4-20 provides a summary of the observation well locations and details. The locations are shown in Figures 2.5.4-1 and 2.5.4-2. Complete observation well details are provided in Section 2.4.12.

Slug testing, for the purposes of measuring the in-situ hydraulic conductivity of the soils, was performed in all 40 wells. The tests were conducted using the falling head method, in accordance with Section 8 of ASTM D4044 (ASTM, 2002b). Slug testing included establishing the 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. Electronic transducers and data loggers were used to measure the water levels and times during the test. Table 2.5.4-21 provides a summary of the hydraulic conductivity values. Details on testing are provided in Section 2.4.12.

Appendix 2.5-A contains the details of well installation records, boring logs for observation wells, and the hydraulic conductivity test results.

Test Pits

A total of 20 test pits were excavated to a maximum depth of 10 ft each using a mechanical excavator. Bulk samples were collected at selected soil horizons in some of the test pits for laboratory testing. Table 2.5.4-22 provides a summary of the test pit locations. The locations are shown in Figure 2.5.4-3. Appendix 2.5-A contains the test pit records.

Field Electrical Resistivity Testing

A total of four field electrical resistivity (ER) tests were performed to obtain apparent resistivity values for the site soils. Table 2.5.4-23 provides a summary of the ER test locations. ER testing was conducted using an Advanced Geosciences, Inc., Sting resistivity meter, a Wenner four-electrode array, and "a" spacings of 1.5 ft, 3 ft, 5 ft, 7.5 ft, 10 ft, 15 ft, 20 ft, 30 ft, 40 ft, 50 ft, 100 ft, 200 ft, and 300 ft in accordance with ASTM G57 (ASTM, 2001a) and IEEE 81 (IEEE, 1983), except as noted below. The arrays were centered on each of the staked locations R-1 and R-2, R-3, and R-4, and are shown in Figures 2.5.4-1 and 2.5.4-2. The electrodes were located using a 300-ft measuring tape along the appropriate bearings using a Brunton compass.

ASTM G57 (ASTM, 2001a) states that electrodes not be driven more than 5% of the electrode separation, which is about 0.9 in. for the smallest "a" spacing of 1.5 ft used. Electrodes, however, were driven about 2.25 in. (or about 12%) at locations where leaves and vegetation were present on the ground, to ensure adequate contact with the soils. ASTM G57 (ASTM, 2001a) states that a decade box be used to check the accuracy of the resistance meter. This verification, however, was conducted using a resistor supplied by the equipment manufacturer in compliance with the manufacturer's recommendations. ASTM G57 (ASTM, 2001a) states that measurement alignments be chosen along uniform topography. Given the topography at the site, however, the array alignments along R-1 and R-2 (shown in Figure 2.5.4-1) contained topographic variation. Finally, IEEE 81 (IEEE, 1983) states that electrodes not be driven into the ground more than 10% of the "a" spacing. As discussed above, at some locations electrodes were driven about 2.25 in. (or about 12%) into the ground. Despite the noted deviations, the collected resistivity values are considered valid and suitable for use.

The raw field data are considered "apparent" resistivity values. The data were modeled in an attempt to remove the geometric and sampling influences and develop vertical profiles that estimate "true" subsurface resistivity values. The values, shown in Table 2.5.4-15, provide a summary of the field resistivity results, as well as "true" resistivity values with depth. For developing vertical profiles, depth values were taken as 1/3 of the a-spacing in the Geotechnical Subsurface Investigation Data Report (Schnabel, 2007). The raw data are provided in Appendix 2.5-A.

Suspension P-S Velocity Logging Survey

Borehole geophysical logging was performed in a total of 10 boreholes. The geophysical survey consisted of natural gamma, long- and short-normal resistivity, spontaneous potential, three-arm caliper, direction survey, and suspension P-S velocity logging. Geotechnical engineers or geologists provided full-time field inspection of borehole geophysical logging activities. Detailed results are provided in Appendix 2.5-A. The P-S logging results are discussed in detail in Section 2.5.4.4.

Subsurface and Excavation Profiles

Subsurface profiles depicting the inferred subsurface stratigraphy are presented in Figures 2.5.4-5 through 2.5.4-9. Profiles depicting excavation geometries and locations of Category I structures, as well as the relationship between their foundations with the subsurface materials, are addressed in Section 2.5.4.5.}

2.5.4.3 Foundation Interfaces

{Foundation interfaces are discussed as an integral part of 2.5.4.5 and 2.5.4.10}

{The logs of test pits that were dug are included in the subsurface investigation report (Schnabel, 2007). Based on the information obtained during the review of information from the CCNPP Units 1 and 2 UFSAR (BGE, 1982) and the observations of the soil samples being taken for the CCNPP Unit 3 subsurface investigation, it was determined that exploratory trenches were not necessary in order to characterize the soils at the CCNPP Unit 3 site.}

2.5.4.4 Geophysical Surveys

{This Section provides a summary of the geophysical survey undertaken for CCNPP Unit 3. Section 2.5.4.4.1 summarizes previous geophysical surveys performed at the CCNPP Units 1 and 2 area. Section 2.5.4.4.2 summarizes those completed during the CCNPP Unit 3 subsurface investigation.

2.5.4.4.1 Previous Geophysical Survey for CCNPP Units 1 and 2

Various geophysical techniques were employed during the original site investigation for CCNPP Units 1 and 2 in 1967. These investigations are addressed in detail in the UFSAR (BGE, 1982). A brief summary of the investigations, reproduced from this reference, is as follows.

2.5.4.4.1.1 Seismic Refraction Survey

Refraction surveys were performed along two lines, 2,000 ft and 2,100 ft in length, for the purpose of obtaining compressional wave velocity data. The data indicated compressional wave velocities in the upper (approximately 40 ft) Pleistocene soils of about 2,200 ft/sec and in the lower (thickness undefined in the UFSAR) Miocene soils of about 5,500 ft/sec to 5,900 ft/sec. Data for deeper deposits, including bedrock, were obtained from measurements at a location several miles south of the site. The results are provided in a summary table, reproduced and shown in Table 2.5.4-24.

2.5.4.4.1.2 Uphole Seismic Velocity Survey

An uphole seismic survey was performed in the plant area for the purpose of correlating the results with those from the seismic refraction survey. The uphole survey was performed in a borehole (DM-4), about 148 ft deep. The results indicated a compressional wave velocity of 2,000 ft/sec in the upper approximately 40 ft and 5,500 ft/sec below, to the maximum depth of about 148 ft. The results are reproduced and shown in Figure 2.5.4-19.

2.5.4.4.1.3 Shear Wave Velocity Measurements

Shear wave propagation was evaluated from surface waves using a Sprengnether velocity meter. The measurements indicated that the shear wave velocity of the Miocene soils is about 1,600 ft/sec. Measurements for other deposits are not reported.

2.5.4.4.1.4 Micromotion Measurements

Micromotion measurements were made at three locations at the site using Microtremor equipment. The results indicated a predominant period of background vibration of about 0.5 sec to 0.75 sec. These measurements were reported to be consistent with results for reasonably dense soils. Based on these observations it was concluded that no special problems could arise in designing the facility at the site.

2.5.4.4.1.5 Laboratory Shockscope Tests

Several samples of the site soils were tested in the laboratory using the Shockscope to obtain compressional wave velocity measurements for correlation with the field measurements. The test results indicated compressional wave velocity measurements ranging from 1,000 ft/sec to 3,200 ft/sec for confining pressures of 0 to 6,000 psf, respectively. The results are reproduced herein and shown in Table 2.5.4-25.

2.5.4.4.1.6 Velocity Profile for CCNPP Units 1 and 2

Based on results of the refraction survey, uphole survey, shear wave velocity measurements, micromotion data, and laboratory shockscope, as well as measurements made in 1943 that extended to greater depth, including bedrock, at locations several miles south of the site, a compressional and shear wave velocity model was prepared for the site, using estimated Poisson's ratios. The results are reproduced herein and shown in Figure 2.5.4-20.

2.5.4.4.2 Geophysical Survey for CCNPP Unit 3

Suspension P-S velocity logging and down-hole seismic CPT tests were performed at 10 boreholes and 8 soundings, respectively, during the CCNPP subsurface investigation. The results are discussed below.

2.5.4.4.2.1 Suspension P-S Velocity Logging

Suspension P-S velocity logging was performed in borings B-301, B-304, B-307, B-318, B-323, B-401, B-404, B-407, B-418, and B-423. The boreholes were uncased and filled with drilling fluid. Boreholes B-301 and B-401 were approximately 400 ft deep each, while the remaining boreholes were approximately 200 ft deep each. The OYO/Robertson Model 3403 unit and the OYO Model 170 suspension logging recorder and probe were used to obtain the measurements. Details of the equipment are described in Ohya (Ohya, 1986). The velocity measurement techniques used for the project are described in Electric Power Research Institute (EPRI) Report TR-102293, Guidelines for Determining Design Basis Ground Motions, (EPRI, 1993). The results are provided as tables and graphs in Appendix 2.5-A.

At this time, an ASTM standard is not available for the suspension P-S velocity logging method, therefore, a brief description follows. Suspension P-S velocity logging 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. At each receiver location, the P- and S-waves are converted to pressure waves in the fluid and received by the geophones mounted in the probe, which in turn send the data to a recorder on the surface. At each measurement depth, two opposite horizontal records and one vertical record are obtained. This procedure is typically repeated every 1.65 ft (0.5 m) or 3.3 ft (1 m) as the probe is moved from the bottom of the borehole toward the ground. 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. For quality assurance, analysis is also performed on source-to-receiver data.

Compressional wave velocity (Vp) and shear wave velocity (Vs) results obtained during the CCNPP Unit 3 subsurface investigation are summarized in Figures 2.5.4-21 and 2.5.4-22 and are discussed herein. Ignoring the measurements above elevation 85 ft (approximate planned finished grade), Vp measurements in Stratum I Terrace Sand ranged from about 850 ft/sec to 5,560 ft/sec, with an increasing trend with depth. Vp measurements in Stratum IIa Chesapeake Clay/Silt ranged from about 3,120 ft/sec to 5,750 ft/sec, with typically decreasing trend with depth. Vp measurements in Stratum IIb Chesapeake Cemented Sand ranged from about 2,350 ft/sec to 8,130 ft/sec, with initially increasing trend with depth, however, with fairly uniform values after a few feet of penetration, except at intermittent cemented zones with peak Vp values. Vp measurements in Stratum IIc Chesapeake Clay/Silt ranged from about 4,800 ft/sec to 5,600 ft/sec, with relatively uniform values throughout the entire thickness, except for occasional minor peaks at intermittent depths. Vp measurements in Stratum III Nanjemoy Sand ranged from about 5,420 ft/sec to 7,330 ft/sec, with relatively uniform values, except for occasional minor peaks at intermittent depths. Results are relatively consistent with those reported from CCNPP Units 1 and 2 (Table 2.5.4-24 and Figure 2.5.4-19) for similar soils. It is noted that Vp values below about elevation 80 ft are typically at or above 5,000 ft/sec; these measurements reflect the saturated condition of the soils below the referenced elevation.

Ignoring the measurements above elevation 85 ft, Vs measurements in Stratum I Terrace Sand ranged from about 400 ft/sec to 1,150 ft/sec, with a relatively uniform trend with depth. Vs measurements in Stratum IIa Chesapeake Clay/Silt ranged from about 590 ft/sec to 1,430 ft/sec, with typically increasing trend with depth. Vs measurements in Stratum IIb Chesapeake Cemented Sand ranged from about 560 ft/sec to 3,970 ft/sec, with significant variation with depth owing to significant changes in density and cementation. Vs measurements in Stratum IIc Chesapeake Clay/Silt ranged from about 1,030 ft/sec to 1,700 ft/sec, with relatively uniform trend in values throughout the entire thickness, except for occasional minor peaks at intermittent depths. Vs measurements in Stratum III Nanjemoy Sand ranged from about 1,690 ft/sec to 3,060 ft/sec, with initially increasing trend in depth, however, relatively uniform at greater depth, except for occasional minor peaks at intermittent depths. Results are relatively consistent with those reported from CCNPP Units 1 and 2 (Figure 2.5.4-20). Based on all 10 suspension P-S velocity measurements, an average Vs profile was estimated for the upper 400 ft, as shown in Figure 2.5.4-23. The measurements from the two deepest boreholes (B-301 and B-401) are also shown for comparison purposes.

Poisson's ratio values were determined based on the Vp and Vs measurements, and are shown in Figure 2.5.4-24. Ignoring the values above elevation 85 ft, Poisson's ratio measurements in Stratum I Terrace Sand ranged from about 0.27 to 0.50. Poisson's ratio measurements in Stratum IIa Chesapeake Clay/Silt ranged from about 0.4 to 0.49, with typically decreasing trend with depth. Poisson's ratio measurements in Stratum IIb Chesapeake Cemented Sand ranged from about 0.26 to 0.49. Poisson's ratio measurements in Stratum IIc Chesapeake Clay/Silt ranged from about 0.45 to 0.48, with a relatively uniform trend in values throughout the entire thickness. Poisson's ratio measurements in Stratum III Nanjemoy Sand ranged from about 0.39 to 0.46, with initially a decreasing trend in depth, however, becoming relatively uniform at greater depth, except for occasional minor peaks at intermittent depths. Based on all 10 borehole measurements, an average Poisson's ratio profile was estimated for the upper 400 ft, which is shown in Figure 2.5.4-25. The values obtained based on velocity measurements from the two deepest boreholes (B-301 and B-401) are also shown for comparison purposes.

It is noted that the above Vp, Vs, and Poisson's ratio measurements reflect the conditions for the approximately upper 400 ft of the site, or to about elevation -317 ft. Information on deeper soils, as well as bedrock, was obtained from the available literature; it is discussed in Section 2.5.4.7.

2.5.4.4.2.2 CPT Seismic Measurements

Shear wave velocity measurements were made using a seismic cone at eight soundings (C-301, C-304, C-307, C-308, C-401, C-404, C-407, and C-408). The measurements were made at 5-ft intervals. At several locations, the soils required pre-drilling to advance the cone, particularly in the cemented zones. Although the deepest CPT sounding was about 142 ft, the combined measurements provided information for the upper approximately 200 ft of the site soils, extending to about elevation -80 ft. Further penetration was not possible due to continued cone refusal. An average of the seismic CPT results is compared with the suspension P-S velocity logging results and shown in Figure 2.5.4-26. The CPT results are found to be relatively consistent with the suspension P-S velocity logging results. The variations in different soils that were observed in the suspension P-S velocity logging data are readily duplicated by the CPT results, including the peaks associated with cemented or hard zones. Further details on testing and the results are provided, in tables and graphs, in Appendix 2.5-A.

Given the similarity between the suspension P-S velocity logging and the seismic CPT results, and that the CPT results only extend to limited depth, the suspension P-S velocity logging results were used as the basis for determination of shear wave velocity profile for the site. The overall recommended velocity profile for the site soils is addressed in Section 2.5.4.7, including the velocity profile for soils below 400 ft depth and bedrock. }

2.5.4.4.2.3 Shear Wave Velocity Profile Selection

Given the similarity between the suspension P-S velocity logging and the seismic CPT results, and that the CPT results only extend to limited depth, the suspension P-S velocity logging results were used as the basis for determination of shear wave velocity profile for the site. The overall recommended velocity profile for the site soils is addressed in Section 2.5.4.7, including the velocity profile for soils below 400 ft depth and bedrock. }

2.5.4.5 Excavation and Backfill

2.5.4.5.1 {Source and Quantity of Backfill and Borrow

A significant amount of earthwork is anticipated in order to establish the final site grade and to provide for the final embedment of the structures. It is currently estimated that approximately 3.5 million cubic yards (cyd) of materials will be moved during earthworks to establish the site grade. The cut/fill operations are essentially balanced.

The materials that will be excavated as part of the site grading are primarily the surficial soils belonging to the Stratum I Terrace Sand. To evaluate these soils for construction purposes, 20 test pits were excavated at the site, as shown in Figure 2.5.4-3. The maximum depth of the test pits was limited to 10 ft. Results of laboratory testing on the bulk samples collected from the test pits for moisture-density and other indices are summarized in Table 2.5.4-26, with the details included in Appendix 2.5-A. The results clearly indicate that there are both plastic and non-plastic soils included in Stratum I soils, including material designated as fill. These soils are predominantly non-plastic. A similar observation was made from the borings that extended deeper than the test pits. Their composition consists of a wide variety of soils, including poorly-graded sand to silty sand, well graded sand to silty sand, clayey sand, silty sand, clay, clay of high plasticity, and silt of high plasticity, based on the USCS. The highly plastic or clay portion of these soils will not be suitable for use as structural fill, given the high percentage of fines (average 59 percent) and the average natural moisture content nearly twice the optimum value of 10 percent. The remaining sand or sandy portion will be suitable; however, these materials are typically fine (sometimes medium to fine) sand in gradation, and likely moisture-sensitive that may require moisture-conditioning. Additionally, at this time, it is very likely that the suitable portion of the excavated soils will be used for site grading purposes, with very little, if any, remaining to be used as structural fill. Therefore, structural fill will be obtained, either through the development of a source on site or from off-site borrow source(s), with consideration to crushed stone materials meeting the requirements of Maryland State Highway Administration coarse aggregate. Structural fill issues will be the subject of a comprehensive investigation during the detailed design stage in order to identify suitable sources of borrow fill that qualify for use as structural backfill. At this time, it is estimated that about 2 million cyd of structural backfill will be needed. Although the source is unidentified at this stage, suitable structural fill will be sought to satisfy several requirements, including material(s) that is sound, durable, well-graded sand or sand and gravel, with maximum 25 percent fines content; and free of organic matter, trash, and deleterious materials. Once identified, the material(s) will be sampled and tested in the laboratory to establish its static and dynamic properties. Chemical tests will also be performed on the candidate backfill material(s). If found feasible, a flyash-cement backfill may be considered as an alternative to structural soil fill, and similarly tested for static and dynamic properties. Once available, the results will be evaluated to verify that they meet the design requirements for structural fill.

2.5.4.5.2 Extent of Excavations, Fills, and Slopes

In the area of planned CCNPP Unit 3, the current ground elevations range from approximately elevation 50 ft to elevation 120 ft, with an approximate average elevation 88 ft, as shown in Figure 2.5.4-1. The planned finished grade in CCNPP Unit 3 powerblock area ranges from about elevation 75 ft to elevation 85 ft; with the centerline of Unit 3 planned at approximately Elevation 85 ft. Earthwork operations will be performed to achieve the planned site grades, as shown on the grading plan in Figure 2.5.4-27. All safety-related structures are contained within the outline of CCNPP Unit 3, except for the water intake structures that are located near the existing intake basin, also shown in Figure 2.5.4-27. A listing of the Category I structures with relevant foundation information is as follows, noting that they are only approximate at the time.

	Foundation elevation (ft)
Reactor Building	44
Safeguards Buildings	44
Fuel Building	44
Emergency Diesel Power Generating Building	79
ESWS Cooling Towers	63
UHS Makeup Water Intake Structure	-25

Foundation excavations will result in removing about 2 million cyd of materials, as currently estimated. The extent of all excavations, backfilling, and slopes for Category I structures are shown in Figures 2.5.4-28 through 2.5.4-32. These sections are taken at locations identified in Figures 2.5.4-1 and 2.5.4-2. These figures illustrate that excavations for foundations of Category I structures will result in removing Stratum I Terrace Sand and Stratum IIa Chesapeake Clay/Silt in their entirety, and will extend to the top of Stratum IIb Chesapeake Cemented Sand, except in the UHS Makeup Water Intake Structure area. In the UHS Makeup Water Intake Structure area, the foundations will likely be supported on Stratum IIc soils, given the interface proximity of Strata IIb and IIc.

The depth of excavations to reach Stratum IIb is approximately 40 ft to 45 ft below the final site grade in the Powerblock area. Since foundations will be deriving support from these soils, variations in the top of this stratum was evaluated, reflected as elevation contours for top of Stratum IIb in CCNPP Unit 3 and in CLA1 areas, as shown in Figure 2.5.4-33. This figure shows that the variation in top elevation of these soils is very little, approximately 4 ft or less (about 1 percent) within each major foundation area. This information will be used to develop the final excavation plan for the foundations. The extent of excavations to final subgrade, however, will be determined during construction based on observation of the actual soil conditions encountered during construction and their suitability for foundation support. Once subgrade suitability in Stratum IIb Cemented soils is confirmed, the excavations will be backfilled with compacted structural fill to the foundation level of structures. Subsequent to foundation construction, the fill would be extended to the final site grade, or near the final site grade, depending on the details of the final civil design for the project. Compaction and quality control/quality assurance programs for backfilling are addressed in Sections 2.5.4.5.3.

Permanent excavation and fill slopes, created due to site grading, are addressed in Section 2.5.5. Temporary excavation slopes, such as those for foundation excavation, would be graded on an inclination of at least 2:1 horizontal:vertical (H:V) or even extended to inclination 3:1 H:V, if found necessary during detailed design based on temporary loading conditions, depending on observations of their performance during construction, and having a factor of safety for stability of at least 1.30 for static conditions. These slopes are currently shown as 3:1 H:V in Figures 2.5.4-28 through 2.5.4-31, with the actual slope inclination to be determined later as described above.

Excavation for the ESWS Cooling Towers will be different than that for CCNPP Unit 3 structures, as shown in Figure 2.5.4-32. Given the proximity of this excavation to the Chesapeake Bay, this excavation will be made by installing a sheetpile cofferdam that will not only provide excavation support but also aid with the dewatering needs. This is addressed further in Section 2.5.4.5.4.

2.5.4.5.3 Compaction Specifications

Once structural fill sources are identified, as discussed in Section 2.5.4.5.1, several samples of the material(s) will be obtained and tested for indices and engineering properties, including moisture-density relationships. For foundation support and backfill against walls, structural fill will be compacted to minimum 95 percent of its maximum dry density, as determined based on the Modified Proctor compaction test procedure (ASTM, 2002c). The fill would be compacted to within 3 percent of its optimum moisture content.

Fill placement and compaction control procedures will be addressed in a technical specification that will be prepared during the detailed design stage of the project. It will include requirements for suitable fill, sufficient testing to address potential material variations, and in-place density testing frequency, e.g., a minimum of one test per 10,000 square ft of fill placed. It will also include requirements for an on-site testing laboratory for quality control (gradation, moisture-density, placement, compaction, etc.) and requirements to ensure that the fill operations conform to the earthwork specification. The soil testing firm is required to be independent of the earthwork contractor and to have an approved quality program. A sufficient number of laboratory tests are required to be performed to ensure that variations in the fill material are accounted for. A test fill program will also be included for the purposes of determining an optimum number of passes, lift thickness, and other relevant data for optimum achievement of the specified compaction.

2.5.4.5.4 Dewatering and Excavation Methods

Groundwater control will be required during construction. Groundwater conditions and dewatering are addressed in Section 2.5.4.6.

Given the soil conditions, excavations will be performed using conventional earth-moving equipment. Most excavations should not present any major difficulties. Blasting is not anticipated. The more difficult excavations would have been in Stratum IIb Cemented Sand, due to the cemented nature and proximity to groundwater, but the cemented portions are not planned to be excavated, except where minor excavations may be needed due to localized conditions or due to deeper foundation elevations such as at the UHS Makeup Water Intake Structure area. Excavations in localized, intermittent cemented soils may require greater excavating effort, such as utilizing hoe-rams or other ripping tools; however, these zones are very limited in thickness, with probably only occasional need for expending additional efforts. Excavations for the CCNPP Unit 3 powerblock foundations will be open cut. Upon reaching the final excavation levels, all excavations will be cleaned of any loose materials, by either removal or compaction in place. All final subgrades will be inspected and approved prior to being covered by backfill or concrete. The inspection and approval procedure(s) will be addressed in the foundation and earthworks specifications that will be developed during the detailed design stage of the project. These specifications will include measures, such as proof-rolling, excavation and replacement of unsuitable soils, protection of surfaces from deterioration, etc.

As discussed in Section 2.5.4.5.2, excavation for the UHS Makeup Water Intake Structure will require the installation of a sheetpile cofferdam. The sheetpile structure is anticipated to extend from the ground surface to a depth of about 50 ft. The full scope of the sheetpile cofferdam will be developed during the detailed design stage of the project. Excavation of soils in this area should not present any major difficulties given their compactness.

Foundation rebound (or heave) will be monitored in excavations for selected Category I structures. Rebound estimates are addressed in Section 2.5.4.10. Monitoring program specifications will be developed during the detailed design stage of the project. The specification document will address issues, such as the installation of a sufficient quantity of instruments in the excavation zone, monitoring and recording frequency, and evaluation of the magnitude of rebound and settlement during excavations and foundation construction. }

2.5.4.6 Groundwater Conditions

2.5.4.6.1 {Groundwater Conditions

The groundwater data collection and monitoring program is still in progress subsequent to the installation of observation wells during the CCNPP subsurface investigation. Details of available groundwater conditions at the site are given in Section 2.4.12. Based on available information through March 2007, the shallow (surficial) groundwater level in CCNPP Unit 3 and CLA1 areas ranges from approximately elevation 73 to elevation 85 ft, or an approximate average elevation 80 ft. This evaluation was used as the design groundwater elevation in the geotechnical calculations, as opposed to the design groundwater elevation of 73 ft as discussed in Section 2.4.12. The value used in the geotechnical calculations is bounded by the DCD value. Similarly, the groundwater level associated with the deeper hydrostatic surface was found to range from approximately elevation 34 ft to elevation 42 ft, with an approximate average elevation 39 ft. The shallow groundwater should have little to no impact on the stability of foundations, as the site grading and excavation plans will implement measures to divert these flows way from excavations, e.g., through runoff prevention measures and/or ditches. There are no Category I foundations are planned within the upper water-bearing soils. The deeper groundwater condition, within the cemented sands, could adversely impact foundation soil stability during construction if not properly controlled, resulting in loss of density, bearing, and equipment trafficability.

2.5.4.6.2 Dewatering During Construction

Temporary dewatering will be required for groundwater management during construction. Analysis of the groundwater conditions at the site is ongoing at this time, given continued groundwater monitoring, as addressed in Section 2.4.12. Nonetheless, a groundwater control implementation program will be needed in order to address the groundwater conditions at the site during excavations for CCNPP Unit 3 foundations. Groundwater control associated with seepage in the shallow (upper) zones will be controlled through site grading and/or a system of drains and ditches, as previously discussed. The deeper groundwater regime will require a more positive control, including a series of sumps and pumps strategically located in the excavation to effectively collect and discharge the seepage that enters the excavation, in addition to ditches, drains, or other conveyance systems. It will be required that the groundwater level in excavations be maintained a minimum of 3 ft below the final excavation level. A groundwater dewatering specification will be developed as part of the detailed design for the project.

Temporary dewatering will also be required for the excavation of the Ultimate Heat Sink Makeup Intake Structure. A sheetpile cofferdam will be designed to aid with the dewatering needs; however, some level of groundwater control will still be required to maintain a relatively "dry" excavation during construction. As a minimum, sumps will be installed to control and/or lower the groundwater level inside the cofferdam. Full details of the dewatering requirements will be developed during the detailed design stage of the project.

2.5.4.6.3 Analysis and Interpretation of Seepage

Analysis of the groundwater conditions at the site is ongoing at this time, given continued groundwater monitoring that is still in progress, as addressed in Section 2.4.12. A groundwater model, based on information currently available, has been prepared for the overall groundwater conditions at the site and is addressed in detail in Section 2.4.12. The groundwater program and milestones are provided in Section 2.4.12

2.5.4.6.4 Permeability Testing

Testing for permeability of the site soils was performed using Slug tests, as discussed in Section 2.5.4.3. A detailed description of the tests and the results is provided in Section 2.4.12. A summary of the hydraulic conductivity values is presented in Table 2.5.4-21.

2.5.4.6.5 History of Groundwater Fluctuations

A detailed treatment of the groundwater conditions is provided in Section 2.4.12.}

2.5.4.7 Response of Soil and Rock to Dynamic Loading

{The SSE spectra and its specific location at a free ground surface reflect the seismic hazard in terms of PSHA and geologic characteristics of the site and represent the site-specific ground motion response spectrum. These spectra would be expected to be modified as appropriate to develop ground motion for design considerations. Detailed descriptions on response of site soils and rocks to dynamic loading are addressed in Section 2.5.2.

2.5.4.7.1 Site Seismic History

The seismic history of the area and the site, including any prior history of seismicity, evidence of liquefaction or boils, is addressed in Sections 2.5.1.1.4.4.5 and 2.5.1.2.6.4.

2.5.4.7.2 P- and S-Wave Velocity Profiles

Given the depth to bedrock of about 2,500 ft and the depth of velocity measurements during the CCNPP Unit 3 subsurface investigation, additional studies were performed to complete the soil column profile for the CCNPP Unit 3 site.

2.5.4.7.2.1 Subsurface Conditons in the Upper 400 Feet

Geophysical measurements in the upper 400 ft were made during the CCNPP Unit 3 subsurface investigation and are addressed in Section 2.5.4.4.2. The average shear wave velocity and Poisson's ratio profiles for the upper approximately 400 ft of the site, as obtained from the CCNPP Unit 3 subsurface investigation, are shown in Figures 2.5.4-23 and 2.5.4-25, respectively.

2.5.4.7.2.2 Subsurface Conditions Below 400 Feet

It is known that sediments at the site extend below the maximum depth of the CCNPP Unit 3 subsurface investigation. With the maximum depth of the subsurface exploration at 400 ft, additional subsurface information was sought to characterize the site conditions below this depth, including bedrock.

<u>Soil Shear Wave Velocity Profile</u>

In seeking available resources, various geologic records were reviewed and communication made with staff at the Maryland Geological Survey, the United States Geological Survey, and the Triassic-Jurassic Study Group of Lamont-Doherty Earth Observatory, Columbia University. The results of this work, and associated references, are addressed in Section 2.5.1. In summary, a soil column profile was prepared, extending from the ground surface to the top of rock, as shown in Figure 2.5.1-34. Soils below 400 ft consist of Coastal Plain sediments of Eocene, Paleocene, and Cretaceous eras, extending to an estimated depth of about 2,500 ft below the ground surface. These soils contain sequences of sand, silt, and clay. Given their geologic age, they are expected to be competent soils, consolidated to at least the weight of the overlying soils.

Several available geologic records were also reviewed in order to obtain information on both the depth to bedrock and the bedrock type, as addressed in Section 2.5.1. Accordingly, the estimated depth to bedrock in the proximity of the site is about 2,555 ft, which is consistent with the depth of 2,500 ft reported in the CCNPP Units 1 and 2 UFSAR (BGE, 1982) and as shown in Figure 2.5.4-20. Top of rock elevation at the CCNPP site is estimated, and adopted, at approximately elevation -2,446 ft which corresponds to a depth of about 2,531 feet. Regional geologic data were also researched for information on bedrock type. This revealed various rock types in the region, including Triassic red beds and Jurassic diabase, granite, schist, and gneiss. However, only granitoid rocks (metamorphic gneiss, schist, or igneous granitic rocks), similar to those exposed in the Piedmont, could be discerned as the potential regional rock underlying the CCNPP Unit 3 site. For the purpose of rock response to dynamic loading, granitoid was considered as the predominant rock type at the CCNPP site.

With the geology established below a depth of 400 ft, velocity profiles also needed to be established. The velocity data were found through a research of available geologic information for the area. From the Maryland Geological Survey data, two sonic profiles were discovered for wells in the area that penetrated the bedrock, one at Chester, MD (about 38 miles north the site, (USGS, 1983) and another at Lexington Park, MD (about 13 miles south of the site, (USGS, 1984); their locations relative to the site are shown in Figure 2.5.4-34. These two sonic profiles were digitized and converted to shear wave velocity, based on a range of Poisson's ratios for the soil and the rock. The two Vs profiles for Chester and Lexington Park are plotted versus elevation, with the superimposed measured velocity profile from the upper 400 ft at the CCNPP site, as shown in Figures 2.5.4-35 and 2.5.4-36.

The bottom of the <u>measured</u> Vs profile in the upper 400 ft fits well with the Chester data for which a soil's Poisson's ratio=0.4 was used (Figure 2.5.4-35), whereas, in the case of Lexington Park data (Figure 2.5.4-36), the bottom of the <u>measured</u> data in the upper 400 ft fits well with the profile for which the soil's Poisson's ratio=0.45 was used. Geologically, the soils at the two sites are quite comparable (refer to Section 2.5.1 for more details on site geology). The reason(s) for the different "fits" is not clear. However, based on actual Poisson's ratio measurement at another deep Coastal Plain site (SNOC, 2006), where suspension P-S velocity logging measurements extended to a depth of over 1,000 ft, a Poisson's ratio of 0.4 was adopted to represent the soil conditions at the CCNPP site, given the geologic similarity of the soils at both sites.

If a Poisson's ratio of 0.4 is used to convert the Chester sonic log to a shear wave velocity log, this shear wave velocity log fits well with the bottom of the site Vs profile measured with suspension logging at comparable elevations (Figure 2.5.4-35). A similarly good fit is obtained for the Lexington Park data when a Poisson's ratio of 0.45 is used (Figure 2.5.4-36). Although geologically the soils at the Chester and Lexington Park sites are quite comparable (refer to Section 2.5.1 for more details on site geology), there are reasons why the soils at the elevation of the bottom of the site profile could have slightly different Poisson's ratio values, e.g., the Lexington Park soils may be more cohesive than the Chester soils. Nevertheless, a single Poisson's ratio value was needed for below the bottom of the measured profile for the CCNPP site. Based on actual Poisson's ratio measurements at another deep Coastal Plain site (SNOC, 2006), where suspension P-S velocity logging measurements extended to a depth of over 1,000 ft, a Poisson's ratio of 0.4 was adopted to represent the soil conditions at the CCNPP site, given the geologic similarity of the soils at CCNPP site and the other Coastal Plain site.

Both profiles (particularly the Chester profile) include significant "peaks," giving a visual impression that the difference in the two profiles may be large. To further look at the variation in these two profiles based on the adopted Poisson's ratio of 0.4, both profiles were averaged over 100-ft <u>intervals</u> along the entire depth to "smooth" the peaks. The original profiles for the two sites (based on a Poisson's ratio of 0.4) and the 100-ft interval average for the two measurements are shown in Figure 2.5.4-37. A comparison of the two 100-ft interval averages show that once the effect of the "peaks" are removed, the two profiles are relatively similar for the same Poisson's ratio of 0.4. Finally, an average of the 100-ft interval data for both sites was taken, as also shown in Figure 2.5.4-37. This latter profile was compared with an available measured profile in deep Coastal Plain soils (SNOC, 2006); its similarity to the measured profile is indicative of its appropriateness for the geologic setting, as shown in Figure 2.5.4-38. Accordingly, based on measured data in the upper 400 ft and data obtained from available literature in areas surrounding the CCNPP site, the recommended shear wave velocity profile in soils at the CCNPP Unit 3 site is shown in Figure 2.5.4-39. This profile is later compared to the profile used for CCNPP Units 1 and 2.

<u>Bedrock Shear Wave Velocity Profile</u>

Similar to the soil profiles addressed above, two velocity profiles were also available for bedrock, based on the sonic data from Chester (USGS, 1983) and Lexington Park (USGS, 1984) sites. Rock was encountered at different depths at these two sites; however, the elevation difference in top of rock is only 11 ft between the two sites. The bottom portions of Figures 2.5.4-35 and 2.5.4-36 (near the soil-rock interface) are enlarged for clarity and are shown in Figures 2.5.4-40 and 2.5.4-41 for the Poisson's ratios shown.

A comparison of the Vs profiles in bedrock for the two sites reveals different velocity responses, regardless of the Poisson's ratio values considered. The Chester profile is somewhat transitional and does not approach 9,200 ft/sec at termination of measurements. The Lexington Park profile is rather abrupt, and is in excess of 9,200 ft/sec. The difference in these two responses is found in the geologic description of the bedrock at the two sites. At Chester, the bedrock is described as more the typical, regional metamorphic rock (granitic, schist, or gneiss). At Lexington Park, the bedrock is described as an intrusive diabase. Based on further evaluation of regional bedrocks, as addressed in Section 2.5.1, the following description was established for the CCNPP Unit 3 site: bedrock is probably granitoid rock, less likely to be sandstone or shale, even less likely to be diabase. Accordingly, the Lexington Park profile (that is for diabase rock) was excluded from further consideration.

Closer examination of the Chester bedrock velocity results reveal that the velocities are rather "insensitive" to the assumption of Poisson's ratio, as is evident in Figure 2.5.4-40. For all practical purposes, the assumption of Poisson's ratio of 0.2, 0.25, or 0.3 for the bedrock renders identical velocity profiles. The responses also follow a particular velocity gradient. For a Poisson's ratio of 0.3 for the rock, one could assume a bedrock velocity starting at some value at the soil-rock interface, transitioning to the 9,200 ft/sec at some depth. This approach was followed, as shown in Figure 2.5.4-42, showing the Vs profile versus elevation in bedrock. From this figure, starting at Vs of 5,000 ft/sec at the soil-rock interface, the 9,200 ft/sec velocity is reached within about 20 ft depth into rock. Many variations were tried (varying the starting velocity at soil-rock interface, varying the slope of transitioning velocity profile, transition in "slope" or in "step," different Poisson's ratios, etc.); the end result appeared relatively unchanged, i.e., the 9,200 ft/sec velocity is achieved within a short distance of penetrating the rock. On this basis, the "stepped" velocity gradient shown in Figure 2.5.4-42 was adopted to define the velocity profile for the rock. The recommended velocity profile for bedrock begins with Vs=5,000 ft/sec at the soil-rock interface, as indicated from the sonic data and also shown in Figure 2.5.4-42, transitioning to 9,200 ft/sec in steps shown in Figure 2.5.4-42.

Both the soil and bedrock velocity profile are reflected in an overall site velocity profile for the CCNPP site, as shown in Figure 2.5.4-43. It should be noted that the top of rock elevation shown in Figure 2.5.4-43 was adjusted to conform to the estimated rock elevation for the CCNPP Unit 3 site, or elevation -2,446 ft (refer to Section 2.5.1). Figure 2.5.4-43 is considered the design shear wave velocity profile for the CCNPP Unit 3 site. A companion figure shows the Poisson's ratios that were measured in the upper 400 ft and those estimated below 400 ft in Figure 2.5.4-44. The numerical values of velocity steps for the entire profile are given in Table 2.5.4-27.

A comparison was made of the adopted Vs and Poisson's ratio profiles described above (Figures 2.5.4-43 and 2.5.4-44) with those used for the original design of CCNPP Units 1 and 2 (as shown in Figure 2.5.4-20). The average values for both CCNPP Units 1 and 2 and from the CCNPP Unit 3 investigation are summarized below, after being "weighted" with respect to a common depth. The weighting included obtaining an average value for each parameter over a particular depth (in this case 1,000 ft) for comparison purposes.

	Average CCNPP Units 1 and 2		Average CCNPP Unit 3	
	Vs (ft/sec)	μ	Vs (ft/sec)	μ
Upper ≈1,000 ft	1,500	0.44	1,900	0.44
Below ≈1,000 ft to Bedrock	3,400	0.35	2,500	0.40
Bedrock	10,000	0.15	9,200	0.30

The average Vs (weighted) values in the upper 1,000 ft for the CCNPP Units 1 and 2 is about 1,500 ft/sec, compared to a weighted average Vs adopted for the CCNPP Unit 3 of 1,900 ft/sec over the same depth. For the soils below 1,000 ft, the CCNPP Units 1 and 2 Vs is reported as 3,400 ft/sec, compared to a weighted average Vs adopted for the CCNPP Unit 3 of about 2,500 ft/sec over the same depth. For bedrock, the CCNPP Units 1 and 2 used Vs=10,000 ft/sec at the top of bedrock, compared to a "transitional" Vs adopted for the CCNPP Unit 3, starting at 5,000 ft/sec, transitioning to 9,200 ft/sec with depth.

The differences between the CCNPP Units 1 and 2 UFSAR (BGE, 1982) and CCNPP Unit 3 subsurface investigation values may be attributed to a variety of factors, including measurement techniques and available technology at the time of measurement, assumptions in data reduction, and available geologic references at the time, among many others. It should be noted that the original 1967 investigation relied primarily on refraction survey and results of a 1943 geophysical survey several miles south of the site to define the soil-rock column profile (reference to both the 1967 and 1943 work are contained in the CCNPP Units 1 and 2 UFSAR (BGE, 1982)); only one measurement in a boring at the site to a depth of about 148 ft provided uphole measurements. Conversely, the CCNPP Unit 3 subsurface investigation used 10 suspension P-S velocity logging sets of measurements at the site, a more advanced technology for velocity measurements than 1960s technology, extending to depths of about 400 ft, including deriving the deeper velocities from actual borehole sonic measurements as close as 13 miles from the site. Similarly, the Poisson's ratios adopted in the CCNPP Unit 3 subsurface investigation derivation of velocity profiles below 400 ft were based on actual suspension P-S velocity logging measurements by others in similar Coastal Plain geology. Equally, the geologic references adopted for estimation of the CCNPP Unit 3 subsurface investigation shear wave velocity profile are recent, building on prior decades of geologic knowledge in the area. On these bases, the shear wave velocity profile adopted for the CCNPP Unit 3 subsurface investigation phase is considered a closer reflection of the site dynamic characterization.

2.5.4.7.3 Dynamic and Static Laboratory Testing

Dynamic testing, consisting of RCTS tests, to obtain data on shear modulus and damping characteristics of the soils is currently in progress and will likely not be completed until the Fall of 2007. In the interim, shear modulus degradation and damping ratio curves from the available literature were used for material characterization. Once the RCTS results become available, an evaluation will be made, comparing the laboratory tests and the adopted values, to verify that they meet the project requirements. Should the results prove to be substantially different, such that they are likely to alter the seismic characterization of the site, a revised set of data will be adopted and the calculations will be repeated. Updates as necessary to the application will be performed. In absence of RCTS test results, descriptions of shear modulus degradation parameters adopted for seismic soil characterization are presented below.

2.5.4.7.3.1 Shear Modulus Degradation Curves for Soils

In absence of actual data for the site soils, generic EPRI curves were adopted from EPRI TR-102293 (EPRI, 1993). EPRI "sand" curves were used for predominately granular soils and "clay" curves were used for predominately clay soils based on estimated PI values. The EPRI "sand" curves cover a depth range up to 1,000 ft. Since soils at the CCNPP site extend beyond 1,000 ft, similar curves were extrapolated from the EPRI curves, extending beyond the 1,000-ft depth, to characterize the deeper soils. For instance, the "1,000-2,000 ft" curve was extrapolated by "off-setting" this curve by the amount shown between the "250-500 ft" and "500-1,000 ft" curves in EPRI TR-102293 (EPRI, 1993). EPRI curve selection for the upper 400 ft of the site soils was based on available soil characterization data from the site investigation. Below 400 ft, the geologic profile that was prepared (Figure 2.5.1-34) was used as a basis for the soil profiles, including engineering judgment to arrive at the selected EPRI curves. The developed EPRI (shear modulus and damping ratio) curves for the CCNPP Unit 3 site are shown in Figure 2.5.4-45. These curves are shown being extended beyond the 1-percent shear strain provided in EPRI TR-102293 (EPRI, 1993), only to aid with the randomization process. In reality, the extended portions will not be used in the final analyses due to the very low strain levels. It should be noted that the damping ratio curves will be truncated at 15 percent, consistent with the maximum damping values that will be used for the site response analysis. Tabulated values of shear modulus reduction and damping ratios are presented in Table 2.5.4-28.

2.5.4.7.3.2 Shear Modulus Degradation Curves for Rock

The two velocity profiles for the Chester and Lexington Park sites (Figures 2.5.4-40 and 2.5.4-41), indicate that "hard" rock (identified with Vs = 9,200 ft/sec) is present at these two site. Hard rocks typically exhibit an elastic response to loading, with little, if any, change is stiffness properties. For the range of shear strains anticipated in the analysis (10^{-4} to 1 percent range), essentially no shear modulus reduction is expected; therefore, for rocks at the site, the estimated shear moduli should remain unaffected, given the relatively high velocity observed from the area rocks.

Hard rocks are considered to have damping, but it is not strain dependent. A damping ratio of 1 percent has been used for bedrock at other sites, e.g., for the Vogtle Early Site Permit application (SNOC, 2006) in order to obtain compatibility with soils above bedrock. Experience on similar work has indicated that using damping ratios of 0.5 percent, 1 percent, 2 percent, and 5 percent produces essentially identical results (Dominion, 2006). Therefore, for the CCNPP Unit 3, a damping ratio of 1 percent was adopted for the bedrock. Bedrock shear modulus was considered to remain constant, i.e., no degradation, in the shear strain range of 10^{-4} percent to 1 percent. The groundwater level of elevation 80 ft was also adopted for the analyses.

Other material parameters that were used for dynamic analysis included material density and soil Plasticity Index. The soil unit weights for the upper 400 ft were obtained from the laboratory test results and site characterization. Those below a depth of 400 ft were estimated based on an approximate correlation of available laboratory data with Gamma-Gamma density measurements available from USGS (USGS, 1983). The values are shown in Table 2.5.4-29. The rock unit weight was estimated from the available literature (Deere, 1966)), as 162 pcf. The Plasticity Index values were used for the selection of appropriate shear modulus and damping ratio curves for the clay soils. Indices for soils in the upper 400 ft of the site were selected and based on available laboratory data. For deeper soils, they were estimated and based on descriptions of the soils in the available literature (USGS, 1983) (USGS, 1984).

2.5.4.7.3.3 Dynamic Properties of Structural Fill

As stated in Section 2.5.4.5.2, all Category I structures will be supported on structural fill, which is in turn supported on Stratum IIb Chesapeake Cemented Sand. Material parameters, static or dynamic, are not available at this time, because the backfill source has yet to be determined. In absence of this information, it is assumed that material parameters for the structural backfill will be similar to parameters for Stratum I Terrace Sand, and therefore, measurements available for Terrace Sand soils were adopted to represent the fill and used in the analyses. Once the structural fill is identified and tested for characterization, a comparison will be made between the assumed parameters and actual data to verify that it meets the project requirements. Should the results prove to be substantially different, such that they are likely to alter the seismic characterization of the site, a new set of data will be adopted based on the test results, and the calculations will be repeated.

2.5.4.7.4 Shear Modulus Estimation

With shear wave velocity and other parameters established, the low strain soil and rock shear modulus values can be estimated from the following equation (Bowles, 1996):

$$Gmax = \gamma \cdot (Vs)2/g \qquad \text{Eq. 2.5.4-15}$$

where, γ=total unit weight, Vs=shear wave velocity, and g=acceleration of gravity. The shear wave velocity data are given in Table 2.5.4-27. The unit weight data are given in Table 2.5.4-29. Strain compatible shear modulus values are estimated during the analysis using Eq. 2.5.4-15.

2.5.4.7.5 Acceleration Time History for Sail-Structure Interaction Analysis

A spectrum-compatible acceleration-time history was developed for use with the velocity profile described in Section 2.5.4.7.2. This acceleration-time history was chosen based on the probabilistic seismic hazard deaggregation information described in Section 2.5.2.

The development of the single horizontal component spectrum-compatible time history is based on the mean 10^{-4} uniform hazard target spectrum described in Section 2.5.2. The spectrum compatible time history was developed for the frequency range of 100 Hz to 0.5 Hz.

Using the site-specific soil column extended to the ground surface and the amplification factor, and the performance-based hazard methodology utilized to develop the SSE (refer to Sections 2.5.2.5 and 2.5.2.6), a zero depth peak ground acceleration of 0.084g associated with a magnitude M5.5 earthquake was computed. These parameters apply to analysis of liquefaction and seismic stability of the soils.

For reconciliation of site specific design parameters affecting the SSE analysis results, refer to Sections 3.7.1 and 3.7.2.}

2.5.4.8 Liquefaction Potential

{The potential for soil liquefaction at the CCNPP Unit 3 site was evaluated following NRC Regulatory Guide 1.198 (NRC, 2003c). The soil properties and profiles utilized are those described in Section 2.5.4.2.

2.5.4.8.1 Previous Liquefaction Studies

Two liquefaction studies are cited in the CCNPP Units 1 and 2 UFSAR (BGE, 1982), as follows. The same reference cites a horizontal ground acceleration of 0.08 g and a Richter magnitude of 4 to 5 for the OBE case, and a horizontal ground acceleration of 0.15 g and a Richter magnitude of 5 to 5.5 for the SSE case.

2.5.4.8.1.1 Liquefaction Potential of Units 1 and 2

CCNPP Units 1 and 2 UFSAR (BGE, 1982) reports that the liquefaction potential at the site was evaluated using data from standard penetration test borings, laboratory test results, in-place density determinations, and geologic origin of the site soils. The results showed that the site soils did not possess the potential to liquefy. Quantitative values for the factor of safety against liquefaction were not given.

2.5.4.8.1.2 Liquefaction Potential of Diesel Generator Building

CCNPP Units 1 and 2 UFSAR (BGE, 1982) also reports on results of a liquefaction study for the siting of the Diesel Generator Building in the North Parking area as a part of CCNPP Units 1 and 2 development. This liquefaction evaluation was performed on data from standard penetration test borings, resulting in computed factors of safety from 1.3 to 2.4, with a median value of 1.8. On this basis, it was determined that the site of the Diesel Generator Building had adequate factor of safety against liquefaction (Bechtel, 1992).

2.5.4.8.2 Soil and Seismic Conditions for CCNPP Unit 3 Liquefaction Analysis

Preliminary assessments of liquefaction for the CCNPP Unit 3 soils were based on observations and conclusions contained within CCNPP Units 1 and 2 UFSAR (BGE, 1982). The site soils that were investigated for the design and construction of CCNPP Units 1 and 2 did not possess the potential to liquefy. Given the relative uniformity in geologic conditions between existing and planned units, the soils at CCNPP Unit 3 were preliminarily assessed as not being potentially liquefiable for similar ground motions, and were further evaluated for confirmation, as will be described later in this subsection. Based on this assessment, it was determined that aerial photography as outlined in Regulatory Guide 1.198 (NRC, 2003c) would not add additional information to the planning and conduct of the subsurface investigation; therefore, was not conducted.

Given the relative uniformity in top and bottom elevations of various soil strata at the site, as indicated in the subsurface profiles in Figures 2.5.4-5 through 2.5.4-9, a common stratigraphy was adopted for the purpose of establishing soil boundaries for liquefaction evaluation. The adopted stratigraphy was that shown in Figure 2.5.4-6 for its location relative to Category I structures and including the deepest borings located on this profile. Only soils in the upper 400 ft of the site were evaluated for liquefaction, based on available results from the CCNPP Unit 3 subsurface investigation. Soils below a depth of 400 ft are considered geologically old and sufficiently consolidated. These soils are not expected to liquefy, as will be further discussed in Section 2.5.4.8.4.

As described in Section 2.5.4.7.5, the resulting peak ground acceleration for the site was found to be 0.084g associated with a magnitude M5.5 earthquake. For conservatism, a peak ground acceleration of 0.125g and an earthquake magnitude of 6.0 were adopted and used for the liquefaction analysis.

2.5.4.8.3 Liquefaction Evaluation Methodology

Liquefaction is defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress (Youd, 2001). The prerequisite for soil liquefaction occurrence (or lack thereof) are the state of soil saturation, density, gradation and plasticity, and earthquake intensity. The present liquefaction analysis employs state-of-the-art methods provided in Youd (Youd, 2001) for evaluating the liquefaction potential of soils at the CCNPP Unit 3 site. Given the adequacy of these methods in assessing liquefaction of the site soils, and the resulting factors of safety which will be discussed later in this subsection, probabilistic methods were not used.

In brief, the present state-of-the-art method considers evaluation of data from SPT, Vs, and CPT data. Initially, a measure of stress imparted to the soils by the ground motion is calculated, referred to as the cyclic stress ratio (CSR). Then, a measure of 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 a ratio of cyclic resistance ratio and cyclic stress ratio. Details of the liquefaction methodology and the relationships for calculating CSR, CRR, FOS, and other intermediate parameters such as the stress reduction coefficient, magnitude scaling factor, accounting for non-linearity in stress increase, and a host of other correction factors, can be found in Youd (Youd, 2001). A magnitude scaling factor (MSF) of 1.97 was used in the calculations based on the adopted earthquake magnitude and guidelines in Youd (Youd, 2001). Below are examples of liquefaction resistance calculations using the available SPT, Vs, and CPT data in the powerblock area of CCNPP Unit 3 and the adjoining CLA1 area. Calculations were performed mainly using spreadsheets, supported by spot hand-calculations for verification.

2.5.4.8.3.1 FOS Against Liquefaction Based on SPT Data

The equivalent clean-sand $CRR_{7.5}$ value, based on SPT measurements, was calculated following recommendations in Youd (Youd, 2001), based on corrected SPT N-values $(N_1)_{60}$, as recommended in Youd (Youd, 2001), including corrections based on hammer-rod combination energy measurements at the site. It is noted that soils at CCNPP site include $(N_1)_{60}\geq 30$; Youd (Youd, 2001) indicates that clean granular soils with $(N_1)_{60}\geq 30$ are considered too dense to liquefy and are classified as non-liquefiable. Similarly, corrections were made for the soils fines contents, based on average fines contents provided in Table 2.5.4-12 and the procedure recommended in Youd (Youd, 2001).

The collected raw (uncorrected) SPT N-values are shown in Figure 2.5.4-46. SPT data from 41 borings located in Unit 3 power block area and in CLA1 are shown in this figure and were used for the liquefaction FOS calculations, or over 2,000 SPT N-value data points. An example of a FOS calculation for a SPT N-value=8 from Boring B-330 at elevation 25.5 ft was hand-calculated for verification and found to conform to the spreadsheet calculations. The SPT value for this sample calculation is identified in Figure 2.5.4-46.

For completeness, all data points, including data for clay soils and data above the groundwater level, were included in the FOS calculation, despite their known high resistance to liquefaction. The SPT N-values shown in Figure 2.5.4-46 were mostly taken at 5-ft intervals. SPT in the deepest borings (B-301 and B-401) extended to about 400 ft below the ground surface. The calculated FOS associated with each of the SPT values in Figure 2.5.4-46 is shown in Figure 2.5.4-47. Also, the FOS=2.25 hand calculated for the SPT value in Boring B-330 at elevation 25.5 ft is shown. Figure 2.5.4-47 additionally shows a demarcation line for FOS=1.1 (FOS=1.1 is discussed at the end of this subsection).

Of the over 2,000 SPT N-value data points for which FOS values were calculated, all but 7 points resulted in FOS>1.1. The 7 points with FOS<1.1 amount to less than 0.5 percent of all the data points evaluated; in other words, over 99.5 percent of the calculated FOS values exceeded 1.1. The FOS<1.1 are highlighted within a "dotted" inset in Figure 2.5.4-47 and are re-plotted for clarity to a higher scale in Figure 2.5.4-48. They range from 0.80 to 1.09. An examination of each FOS is as follows.

Boring	Ground elevation (ft)	El of FOS < 1.1	Value of FOS < 1.1	Overlying Structure	Structure BOF elevation (ft)	Disposition of Soils in the Area with FOS < 1.1	
B-305	72.0	63.0	0.93	Safeguard Bldg.	43.6	Soils will be removed to elevation 38± ft during excavation for Safeguard Bldg.	√
B-314	52.8	50.9	0.80	RadWaste Bldg.	47.1	Soils will be removed to at least elevation 47± ft during excavation for RadWaste Bldg	√
B-331	68.3	66.1	0.94	Turbine Bldg.	45.0	Soils will be removed to at least elevation 45 ft during excavation for Turbine Bldg. foundation	√
B-404	67.9	27.9	0.82	CLA1	N.A.	No structures planned	
B-419	55.3	53.1, 48.8 & 30.3	1.06, 0.81& 1.09	CLA1	N.A.	No structures planned	

For √, see comments below. N.A.=Not Applicable

From the above list, it is noted that all soils indicating FOS<1.1 are either at elevations that will eventually be lowered during construction which would result in the removal of these soils (as indicated by √), or are at locations where no structures are planned. Hence, the low FOSs should not be a concern for these samples.

2.5.4.8.3.2 FOS Against Liquefaction Based on Vs Data

Similar to the FOS calculations for the SPT values, equivalent clean-sand $CRR_{7.5}$ values, based on Vs measurements, were calculated following recommendations in Youd (Youd, 2001). Similarly, corrections were made for the soils fines contents, based on average fines contents provided in Table 2.5.4-12 and the procedure recommended in ASCE (ASCE, 2000). It is noted that soils at CCNPP site include soils with normalized shear wave velocity (V_{s1}) exceeding a value of 215 m/sec. Clean granular soils with $V_{s1} \geq 215$ m/sec are considered too dense to liquefy and are classified as non-liquefiable (Youd, 2001). The limiting upper value of V_{s1} for liquefaction resistance is referred to as V_{s1}^*; the latter varies with fines content and is 215 m/sec and 200 m/sec for fines contents of ≤5 percent and ≥35 percent, respectively. As such, when values of $V_{s1} \geq V_{s1}^*$, the soils were considered too dense to liquefy, and therefore, the maximum CRR value of 0.5 was used in the FOS calculations.

Shear wave velocity data from the P-S logging measurements were used for the FOS calculations. The collected raw (uncorrected) V_s data are shown in Figure 2.5.4-49, which is from all the 10 suspension P-S velocity logging boreholes in CCNPP Unit 3 and in CLA1 areas. Suspension P-S velocity logging measurements were made at 0.5-m intervals (~1.6-ft). The two deepest measurements (at borings B-301 and B-401) extended to about 400 ft below the ground surface. Approximately 1,400 Vs data points were used for the FOS calculations. An example of a FOS calculation for Vs=590 ft/sec from Boring B-423 at elevation 80.6 ft was hand-calculated for confirmation. This Vs value is identified in Figure 2.5.4-49.

For completeness, all data points, including data for clay soils, were included in the calculation, despite their known high resistance to liquefaction. The calculated FOS associated with each of the Vs values shown in Figure 2.5.4-49 is shown in Figure 2.5.4-50. Also, the FOS=2.2 hand calculated for the Vs value in Boring B-423 at elevation 80.6 ft is shown. Figure 2.5.4-50 additionally shows a demarcation line for FOS=1.1.

The results show that all calculated FOSs exceeded 1.1; almost all are at least 4.0, with a few scattered values at about 2.0. The high calculated FOS values are the result of V_{s1} values typically exceeding the limiting V_{s1}^* values, indicating no potential for liquefaction, and therefore, a maximum CRR=0.5 was used in the calculations. The effect of CRR=0.5, as applicable to $V_{s1} \geq V_{s1}^*$ cases, is observed in the rather consistent FOS values shown in Figure 2.5.4-50.

2.5.4.8.3.3 FOS Against Liquefaction Based on CPT Data

The CPT testing at the CCNPP Unit 3 site included the measurement of both commonly measured cone parameters (tip resistance, friction, and pore pressure) and shear wave velocity. The evaluation of liquefaction based on both the commonly measured parameters and shear wave velocity is addressed herein. The CCNPP Unit 3 site CPT data was reviewed and correlated with the applicable SPT data and compared with guidelines in Robertson (Robertson, 1988). As discussed in subsections 2.5.4.2.1.1 through 2.5.4.2.1.3, this review .process verified the CPT data by correlation to the CCNPP Unit 3 site-determined SPT values and data published for relevant soil parameters.

The equivalent clean-sand CRR7.5 value, based on CPT tip measurements, was calculated following recommendations in Youd (Youd, 2001), based on normalized clean sand cone penetration resistance (qc1N)cs and other parameters such as the soil behavior type index, Ic.

Cone tip resistance values, q_c, from all 27 CPT soundings in CCNPP Unit 3 powerblock and CLA1 areas are shown in Figures 2.5.4-51 and 2.5.4-52. The CPT soundings encountered repeated refusal in the cemented sand layer, and could only be advanced deeper after pre-drilling through these soils, indicative of their high level of resistance to liquefaction. The deepest CPT sounding (C-407) penetrated 142 ft below the ground surface, encountering refusal at that depth, terminating at approximately elevation -80 ft. Tip resistance measurements were made at 5-cm intervals (~2-in). Approximately 5,200 tip resistance measurements were made in the soundings in CCNPP Unit 3 powerblock and CLA1 areas, and were used for the FOS calculations. An example of a FOS calculation for a tip resistance value of 36.8 tsf in C-408 at elevation 76.4 ft was hand-calculated for confirmation. This value is identified in Figure 2.5.4-52.

For completeness, all data points, including data for clay soils, were included in the calculation, despite their known high resistance to liquefaction. The calculated FOS associated with each of the tip resistance values shown in Figure 2.5.4-52 are shown in Figure 2.5.4-53. Also, the

FOS=1.52 hand-calculated for the tip resistance value of 36.8 tsf in CPT C-408 at elevation 76.4 ft is shown. Figure 2.5.4-53 additionally shows a demarcation line for FOS=1.1.

Of the over 5,000 data points for which FOSs were calculated, about 100 points indicated FOS<1.1, or approximately 2 percent; in other words, 98 percent of the data points resulted in FOS>1.1. The points with FOS<1.1 are highlighted within a "dotted' inset on Figure 2.5.4-53 and are re-plotted for clarity to a higher scale in Figure 2.5.4-54. An examination of each of these FOSs is as follows.

Boring	Ground elevation (ft)	El range of FOS<1.1	Range of FOS<1.1	Overlying Structure	Structure BOF elevation (ft)	Disposition of Soils in the Area with FOS<1.1	
C-304	60.9	60.1 – 60.0	0.93–1.04	Emergency Power Generating Bldg.	78.6	Soils will be removed to elevation 40± ft* in excavation for Emergency Power Generating Building.	√
C-308	84.3	61.4	1.03	ESWS Cooling Towers	62.6	Soils will be removed to elevation 38± ft* in excavation for ESWS Cooling Towers	√
C-314	80.1	78.1 – 64.7	0.82–1.08	Transformers	N.K.	Soils will be removed to elevation 45± ft in excavation for Turbine Bldg.	√
C-311	73.9	72.8–70.5	1.05	Turbine Bldg.	45.0	Soils will be removed to elevation 45 ft* in excavation for Turbine Bldg. foundations	√
C-313	79.9	78.8 – 67.5	1.05–1.07	Transformers	N.K.	Soils will be removed to elevation 65± ft in excavation for Turbine Bldg.	√
C-402	73.1	72.5 – 70.8	0.81–1.05	CLA1	N.A.	No structures planned	
C-406	43.9	41.9 – 29.0	0.72–1.08	CLA1	N.A.	No structures planned	

For √ and * see comments below. N.K.=Not Known N.A.=Not Applicable

From the above list, it is noted that all soils that indicated FOS<1.1 are either within elevations that will eventually be lowered during construction which will result in the removal of these soils (as indicated by √) or are at locations where no structures are planned. Excavation for the Emergency Power Generating Building, the ESWS Cooling Towers, and the Turbine Building (as indicated by *) will extend to the noted elevations for deriving support for their foundations from the Chesapeake Cemented Sand. Nevertheless, it is noted that the CPT-based CRR relationship was intended to be conservative, not necessarily to encompass every data point; therefore, the presence of a few data points beyond the CRR base curve is acceptable (Youd, 2001).

Shear wave velocity measurements were made in 7 of the CPT soundings in Unit 3 and CLA1 areas at the locations shown in Figure 2.5.4-2. As noted earlier, the CPT soundings encountered repeated refusal in the cemented sand layer, and they could only be advanced deeper after pre-drilling through these soils. Shear wave velocity measurements from the seismic cone were compared to similar measurements using the P-S logging method. The average results are shown in Figure 2.5.4-26. By observation, the two independent measurements are comparable. Given that Vs data from the suspension P-S velocity logging method resulted in high values of FOS against liquefaction, as described in this subsection, similar results are expected from the seismic CPT data, and therefore, separate calculations were not made for the CPT Vs results.

2.5.4.8.4 Liquefaction Resistance of Soils Deeper Than 400 Feet

Liquefaction evaluation of soils at the CCNPP Unit 3 site was focused on soils in the upper 400 ft. The site soils, however, are much deeper, extending to approximately 2,500 ft below the ground surface. Geologic information on soils below a depth of 400 ft was gathered from the available literature, indicating that these soils are from about 50 to over 100 million years old, as shown in Figure 2.5.1-34. Liquefaction resistance increases markedly with geologic age, therefore, the deeper soils are geologically too old to be prone to liquefaction. Additionally, their compactness and strength are only anticipated to increase with depth, compared with the overlying soils. The Pleistocene soils have more resistance than Recent or Holocene soils and pre-Pleistocene sediments are generally immune to liquefaction (Youd, 2001). Additionally, liquefaction analyses using shear wave velocity values of about 2,000 ft/sec near the 400-ft depth did not indicate any potential liquefaction at that depth, with the FOSs exceeding 4.0. With shear wave velocities increasing below the 400-ft depth, in the range of about 2,200 ft/sec to 2,800 ft/sec as indicated in Figure 2.5.4-39, high resistance to liquefaction would be expected from these deeper soils. On this basis, liquefaction of soils at the CCNPP Unit 3 site below a depth of 400 ft is not considered possible.

2.5.4.8.5 Concluding Remarks

A liquefaction analysis was performed using procedures outlined in Youd (Youd, 2001). Over 2,000 SPT data points were analyzed from 41 test borings, from which 99.5 percent of the calculated FOSs exceeded 1.1. Over 1,400 Vs data points from 10 suspension P-S velocity logging boreholes were analyzed; the calculated FOS for the overwhelming majority exceeded 4.0, with few values in the 2.0 range. All values exceeded 1.1. Finally, over 5,000 CPT data points from CPT soundings were evaluated. Approximately 98 percent of the calculated FOSs exceeded 1.1. An examination of the remaining 2 percent with FOS<1.1 revealed that the affected soils will either be removed during construction or are at locations where no structures are planned.

It is evident, from the collective results, that soils at the CCNPP Unit 3 site are so consolidated, geologically old, and sometimes even cemented that they are not susceptible to liquefaction due to acceleration levels from the anticipated earthquakes. A very limited portion of the data at isolated locations indicated potentially liquefiable soils, however, this indication cannot be supported by the overwhelming percentage of the data that represent these soils. Moreover, the state-of-the-art methodology used for the liquefaction evaluation was intended to be conservative, not necessarily to encompass every data point; therefore, the presence of a few data points beyond the CRR base curve is acceptable (Youd, 2001). Additionally, in the liquefaction evaluation, the effects of age, overconsolidation, and cementation were ignored, which tend to increase resistance to liquefaction. Finally, the earthquake acceleration and magnitude levels adopted for the liquefaction analysis are conservative. More importantly, there is no documented liquefaction case for soils in the State of Maryland (USGS, 2000). Therefore, liquefaction should not be a concern. A similar conclusion was arrived at for the original CCNPP Units 1 and 2 (BGE, 1982).

A significant level of site grading is anticipated at the CCNPP Unit 3 site during construction. This primarily results in the removal of geologically younger materials (the upper soils) from the higher elevations, and the placement of dense compacted fill in lower elevations, further improving the liquefaction resistance of soils at the site.

It is noted that limited man-made fill may be present at the CCNPP Unit 3 site at isolated locations. These soils will be removed during construction, further improving the liquefaction resistance of soils at the site.

2.5.4.8.6 Regulatory Guide 1.198

Before and during the foregoing evaluation, guidance contained in NRC Regulatory Guide 1.198 (NRC, 2003c) was used. The liquefaction evaluation conforms closely to the NRC Regulatory Guide 1.198 guidelines.

Under "Screening Techniques for Evaluation of Liquefaction Potential," NRC Regulatory Guide 1.198 (NRC, 2003c) lists the most commonly observed liquefiable soils as fluvial-alluvial deposits, eolian sands and silts, beach sands, reclaimed land, and uncompacted hydraulic fills. The geology at the CCNPP site includes fluvial soils and man-made fill at isolated locations. The liquefaction evaluation included all soils at the CCNPP site. The man-made fill, which is suspected only at isolated locations, will be removed during the site grading operations. In the same section, NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that clay to silt, silty clay to clayey sand, or silty gravel to clayey gravel soils can be considered potentially liquefiable. This calculation treated all soils at the CCNPP Unit 3 site as potentially liquefiable, including the fine-grained soils. The finer-grained soils at the CCNPP Unit 3 site contain large percentages of fines and/or are plastic and are, therefore, considered non-liquefiable, as also indicated by the calculated FOSs for these soils. In fact, all soils at the CCNPP Unit 3 site contain some percentage of fines and exhibit some plasticity, which tends to increase their liquefaction resistance. The same section of NRC Regulatory Guide 1.198 (NRC, 2003c) confirms that potentially liquefiable soils that are currently above the groundwater table, are above the historic high groundwater table, and cannot reasonably be expected to become saturated, pose no potential liquefaction hazard. In the liquefaction analyses, the groundwater level was taken at elevation 80 ft. This water level may be a "perched" condition, situated above Stratum IIa Chesapeake Clay/Silt, with the actual groundwater level near the bottom of the same stratum in the Chesapeake Cemented Sand, or at about an average elevation 39 ft. Despite the adopted higher groundwater level (a higher piezometric head of more than 40 ft), the calculated FOS overwhelmingly exceeded 1.1. The site historic groundwater level is not known, however, it is postulated that the groundwater level at the site has experienced some fluctuation due to pumping from wells in the area and climatic changes. Groundwater levels at the site are not expected to rise in the future given the relief and topography of the site, promoting drainage. Similarly, NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that potentially liquefiable soils may not pose a liquefaction risk to the facility if they are insufficiently thick and of limited lateral extent. At the CCNPP Unit 3 site, the soil layers are reasonably thick and uniformly extend across the site, except where they have been eroded, yet the FOSs overwhelmingly exceeded

1.1. Soils identified as having FOS<1.1, regardless of the thickness, will be removed during grading operations or are located where no structures are planned.

Under "Factor of Safety Against Liquefaction," NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that FOS≤1.1 is considered low, FOS≈1.1 to 1.4 is considered moderate, and FOS ≥ 1.4 is considered high. A FOS=1.1 appears to be the lowest acceptable value. On the same issue, the Committee on Earthquake Engineering of the National Research Council (CEE, 1985) states that "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." This, and a minimum FOS=1.1 in NRC Regulatory Guide 1.198 (NRC, 2003c), are consistent with the FOS=1.1 adopted for the assessment of FOSs for the CCNPP Unit 3 site soils, considering the conservatism adopted in ignoring the cementation, age, and overconsolidation of the deposits, as well as the seismic acceleration and magnitude levels. Such level of conservatism in the evaluation, in conjunction with ignoring the geologic factors discussed above, justifies the use of FOS=1.1 for liquefaction assessment of the CCNPP site soils.}

2.5.4.9 Earthquake Design Basis

{Section 2.5.2.6 describes the development of the horizontal Safe Shutdown Earthquake (SSE) ground motion for the CCNPP Unit 3 site. The selected SSE ground motion is based on the risk-consistent/performance-based approach of NRC Regulatory Guide 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion" with reference to NUREG/CR-6728 and ASCE/SEI 43-05 (refer to Section 2.5.2.6 for references). Any deviation from the guidance provided in Regulatory Guide 1.208 is discussed in Section 2.5.2. Horizontal ground motion amplification factors are developed in Section 2.5.2.5 using site-specific data and estimates of near-surface soil and rock properties presented in Section 2.5.4. These amplification factors are then used to scale the hard rock spectra, presented in Section 2.5.2.4, to develop Uniform Hazard Spectra (UHS), accounting for site-specific conditions using Approach 2A of NUREG/CR-6769. Horizontal SSE spectra are developed from these soil UHS, using the performance-based approach of ASCE/SEI 43-05, accepted by Regulatory Guide 1.208. The SSE motion is defined at the free ground surface of a hypothetical outcrop at the base of the foundation. Section 2.5.2.6 also describes vertical SSE ground motion, which was developed by scaling the horizontal SSE by a frequency-dependent vertical-to-horizontal (V:H) factor, presented in Section 2.5.2.6.}

2.5.4.10 Static Stability

{The area of planned Unit 3 will be graded to establish the final site elevation, which is currently estimated will be at about elevation 85 ft at the center of the unit. The Reactor, Safeguard, and Fuel Buildings are seismic Category I structures and will be supported on a common basemat. The common basemat has an irregular shape, estimated to be approximately 64,400 square ft, or about 322 ft x 200 ft in plan dimensions if an approximate rectangular configuration is considered. All Category I structures' size and depth ranges are summarized below.

Category I Structure	Estimated Foundation elevation (ft)	Estimated Final Site Grade elevation (ft)	Estimated Foundation Depth (ft)*	Estimated Footing Size (ft x ft)
Reactor	44	85	41	322 x 200
ESWS Cooling Towers	63	81–82	18–19	147 x 96
Emergency Power Generating Building	79	82	3	131 x 93
UHS Water Intake Makeup Structure	-25	10	35	78 x 47

* approximate value, below respective final site grade

Structures locations and designations are shown in Figure 2.5.4-2. Other major structures in the power block area are the Auxiliary Building, RadWaste Building, and the Turbine Building, which are Category II structures.

Construction of the Reactor basemat requires an excavation of about 41 ft (from approximately elevation 85 ft). The resulting rebound (heave) in the ground due to the removal of the soils is expected to primarily take place in Stratum IIc Chesapeake Clay/Silt soils. A rebound of about 2 in. is estimated due to excavation for the Reactor basemat, and is expected to take place concurrent with the excavation. Ground rebound will be monitored during excavation.

2.5.4.10.1 Bearing Capacity

The U.S. EPR DCD includes the following COL item in Section 2.5.4.10.1:

> A COL applicant that references the U.S. EPR design certification will verify that site-specific foundation soils beneath the NI basemat have the capacity to support bearing pressures with a function of safety of 3.0 under static conditions.

This COL item is addressed in the following sections.

2.5.4.10.1.1 Bearing Condition of Units 1 and 2 Soils

CCNPP Units 1 and 1 UFSAR (BGE, 1982) provides an evaluation of the site soils for bearing purposes for CCNPP Units 1 and 2. It indicates that the upper (Pleistocene Age) soils are capable of supporting light loads, on the order of 2 to 3 kips per square foot (ksf) for a small amount of settlement. The lower (Miocene Age) soils are described as being capable of supporting heavy loads, on the order of 15 ksf to 20 ksf with slight consolidation.

The CCNPP Units 1 and 2 Turbine Building, Auxiliary Building, Containments, Turbine Generators, and Circulating Water Systems are supported on mat foundations on the Miocene soils. Site grading prior to foundation construction resulted in significant ground unloading. The following is a summary of pertinent information (BGE, 1982).

Structure	Contact Pressure (ksf)	Foundation elevation (ft)	Average Ground elevation (ft)	Average Excavation Unloading (ksf)
Containment Structure Mat	8	-1	60 to 75	6.6 to 8.4
Auxiliary Building Mat	8	-14 to -19	70	8.3 to 8.85
Turbine Pedestal Mat	5	---	---	---
Turbine Building Column Footings	5	-11	40 to 60	4.9 to 7.3
Intake & Discharge Structure Mat	2.5	-27 to -30	20 to 80	4.05 to 10.8

It is also reported in CCNPP Units 1 and 1 UFSAR (BGE, 1982) that elastic expansion of the soils occurred as a result of the excavations, producing "slight upward movement." No magnitude, however, is given. Reference is also made to downward movement of the soils as the foundation load was applied, resulting in a "small" movement and "was complete when construction was completed." No magnitude, however, is given.

2.5.4.10.1.2 Bearing Capacity of CCNPP Unit 3 Structures

The ultimate (gross) bearing capacity of a footing, q_{ult}, supported on homogeneous soils can be estimated by (Vesic, 1975):

$$q_{ult} = cN_c\zeta_c + \gamma' D_f N_q \zeta_q + 0.5\gamma B N_\gamma \zeta_\gamma \qquad \text{Eq. 2.5.4-16}$$

where, c=undrained shear strength for clay material (c_u) or cohesion intercept for (c, ϕ) material,

$\gamma' D_f$ = effective overburden pressure at base of foundation,

γ' = effective unit weight of soil,

D_f = depth from ground surface to base of foundation,

B = width of foundation,

N_c, N_q, and N_γ are bearing capacity factors (defined in Vesic, 1975), and

ζ_c, ζ_q, and ζ_γ are shape factors (defined in Vesic, 1975).

The ultimate bearing capacity, q_u, of a footing supported on a strong sandy layer underlain by weaker soil (a 2-layer system) can be estimated by (Meyerhof, 1978):

$$q_u = q_b + \gamma_1 H^2\left(1+\frac{B}{L}\right)\left(1+\frac{2D_f}{H}\right)\left(\frac{K_s \tan\phi_1}{B}\right) - \gamma_1 H \le q_t \qquad \text{Eq. 2.5.4-17}$$

where, $q_b = c_2 N_{c2}\zeta_{c2} + \gamma_1(D_f + H)N_{q2}\zeta_{q2} + 0.5\gamma_2 B N_{\gamma 2}\zeta_{\gamma 2}$ Eq. 2.5.4-18A

$q_t = c_1 N_{c1}\zeta_{c1} + \gamma_1 D_f N_{q1}\zeta_{q1} + 0.5\gamma_1 B N_{\gamma 1}\zeta_{\gamma 1}$ Eq. 2.5.4-18B

K_s = punching shear coefficient, defined in Meyerhof (Meyerhof, 1978)

H = depth to the lower layer

The footing factors in Eq. 2.5.4-18, are defined as follows:

Layer	Effective Unit Weight	Soil Friction	shear strength	Bearing Capacity factors	Shape factors
Top (strong layer)	γ_1	ϕ_1	c_1	N_{c1}, N_{q1}, $N_{\gamma 1}$	ζ_{c1}, ζ_{q1}, $\zeta_{\gamma 1}$
Bottom (weak layer)	γ_2	ϕ_2	c_2	N_{c2}, N_{q2}, $N_{\gamma 2}$	ζ_{c2}, ζ_{q2}, $\zeta_{\gamma 2}$

For each of the Category I structures under consideration, the bearing capacity of the foundations was estimated using two methods, i.e., (1) considering a layered system (Meyerhof, 1978), assuming a strong layer (Stratum IIb Chesapeake Cemented Sand) over a "weak" layer (Stratum IIc Chesapeake Clay/Silt), and (2) considering homogenous soils (Vesic, 1975), assuming Stratum IIc Chesapeake Clay/Silt soils are present under the foundation in entirety. This assumption provides a lower-bound estimate of the bearing capacity.

It is noted that the Reactor, Safeguard, and Fuel Buildings, which are on a common basemat, will essentially derive support from Stratum IIb Chesapeake Cemented Sand. All other structures, except the UHS Water Intake Structure, will be supported on compacted structural fill resting on Stratum IIb Chesapeake Cemented Sand. The UHS Water Intake Structure will essentially derive support from Stratum IIc Chesapeake Clay/Silt soils. No Category I structure will be supported on Stratum I Terrace Sand or Stratum IIa Chesapeake Clay/Silt.

The subsurface conditions and material properties were described in Section 2.5.4.2. Material properties, conservatively designated for the various strata, were used for foundation evaluation, as shown in Table 2.5.4-12. In absence of actual data, the following properties for compacted fill were taken: a unit weight of 120 pcf, an angle of internal friction of 32 degrees, and a modulus of elasticity of 500 tsf. Location of structures, relative to the subsurface conditions, are shown in Figures 2.5.4-28 through 2.5.4-32. An average groundwater level at elevation 80 ft was used for foundation evaluation. For the case of the UHS Makeup Water Intake Structure where the ground surface was below elevation 80 ft, the groundwater elevation was considered to be at the ground surface.

A summary of the estimated allowable bearing pressures, using both the layered and the homogeneous soils assumptions, including recommended values, are as follows. A factor of safety of 3.0 was applied to obtain the allowable values.

Category I Structure	Allowable Bearing Pressure (Layered System) (ksf)	Lower-Bound Allowable Bearing Pressure (ksf)	Recommended Max. Bearing Pressure (ksf)
Essential Service Water System (ESWS) Cooling	13 - 14	8.0	13

Tower (UHS)			
Emergency Power Generating Building (EDGB)	14 - 15	7.8	13
Common Basemat	24	8.3	20
UHS Makeup Water Intake Structure	---	8.0	8

Actual foundation pressures for the Category I structures are not available at this time, however, they were approximated for comparison with the allowable values above, as follows.

ESWS Cooling Tower (UHS)	7
(EDGB)	5
Common Basemat	15
UHS Makeup Water Intake Structure	6

The recommended maximum bearing pressures well exceed the estimated foundation pressures. Traditionally, a factor of safety of 3.0 has been found acceptable for foundation design, although lower factors of safety (1.7 to 2.5) have been suggested for mat foundations (Bowles, 1996). A factor of safety of 3.0 was used in the bearing capacity evaluations. A comparison of the recommended maximum bearing pressures with the estimated foundation pressures suggest that the final factor of safety may even be higher than 3.0. Additionally, the recommended bearing pressures are comparable with estimates of bearing capacity identified in the CCNPP Units 1 and 2 UFSAR (BGE, 1982); the notable difference is in the estimate of "actual" foundation pressure of 15 ksf for the Common Basemat and the "contact pressure" of 8 ksf for the Containment Structure Mat of CCNPP Units 1 and 2.

The site-specific foundation soils beneath the NI basemat have been verified to have the capacity to support the bearing pressures with a factor of safety of 3.0 under static conditions.

2.5.4.10.2 Settlement

The pseudo-elastic method of analysis was used for settlement estimates. This approach is suitable for the overconsolidated soils at the site. The analysis is based on a stress-strain model that computes settlement of discrete layers:

$$\delta = \Sigma(\Delta p_i \times \Delta h_i)/E_i \qquad \text{Eq. 2.5.4-19}$$

where, δ = settlement

i = 1 to n, where n is the number of soil layers

p_i = vertical applied pressure at center of layer i

h_i = thickness of layer i

Ei = elastic modulus of layer i

The stress distribution below the rectangular foundations is based on a Boussinesq-type distribution for flexible foundations (Poulos, 1974). The computation extends to a depth where the increase in vertical stress (Δp) due to the applied load is equal to or less than 10 percent of the applied foundation pressure. The Boussinesq-type vertical pressure under a rectangular footing, σ_z, is as follows (Poulos, 1974):

$$\sigma_z = (p/2\pi)(\tan^{-1}(lb/(zR_3)) + (lbz/R_3)(1/R_1^2 + 1/R_2^2)) \qquad \text{Eq. 2.5.4-20}$$

where,
l = length of footing
b = width of footing
z = depth below footing at which pressure is computed
$R_1 = (l^2 + z^2)^{0.5}$
$R_2 = (b^2 + z^2)^{0.5}$
$R_3 = (l^2 + b^2 + z^2)^{0.5}$

Settlement estimates were made following the preceding relationships and using available soils properties given in Table 2.5.4-12. Settlement estimates were made for all Category I structures, for the estimated "actual" foundation pressures given in this subsection. They are as follows.

Category I Structure	Est. "Actual" Foundation Pressure (ksf)	Est. Foundation Settlement (in.)		
		Center	Edge	Average
ESWS Cooling Tower (UHS)	7	5	3	4
EDGB	5	4	2	3
Common Basemat	15	10	6	8
UHS Makeup Water Intake Structure	6	2	1	1.5

The settlement magnitudes are discussed later. However, net foundation pressures and, therefore, the final estimate of settlements due to net loading, will be somewhat different than those above, given the planned site grading and excavation for foundations.

The planned site grading will result in removing as much as 23 ft of soil in the area of the Emergency Power Generating Building-South (1UBP and 2UBP, shown in Figure 2.5.4-2) and in adding as much as 17 ft of fill to the Emergency Power Generating Building-North (3 UBP and 4 UBP shown in Figure 2.5.4-2). Additionally, foundations will rest as much as 3 ft to 41 ft below the final site grade for the Emergency Power Generating Building and the Common Basemat, respectively, resulting in further changes in the net foundation loading. Net foundation pressures were estimated, based on available grading information, as follows.

Category I Structure[(1)]	Approx. [Average] Existing Site Grade elevation (ft)	Approx. Final Grade elevation (ft)	Approx. Foundation elevation (ft)	Est. "Actual" Foundation Pressure (ksf)	Est. Net Foundation Pressure (ksf)
ESWS Cooling Tower North(URB3&4) ESWS Cooling Tower-South(URB1&2)	60 - 95 [80] 90 - 120 [100]	81 82	63	7	6 4
EDGB-North(UBP3&4) EDGB-South(UBP1&2)	55 - 70 [65] 105 – 115 [105]	82 82	79	5	7 2
Common Basemat	70 – 110 [90]	85	44	15	11
UHS Makeup Water Intake Str.	10 [10]	10	-25	6	4

(1) Refer to Figure 2.5.4-2 for locations

Estimated settlements corresponding to the net foundation pressures are given below. It is noted, however, that the magnitude of estimated settlements essentially remain unchanged, given only slight change in foundation pressures.

Category I Structure	Est. Net Foundation Pressure (ksf)	Est. Foundation Settlement (in.)		
		Center	Edge	Average
ESWS Cooling Tower-North	6	5	3	4
ESWS Cooling Tower-South	4	3	2	2
EDGB-North	7	5	3	4
EDGB-South	2	2	1	1
Common Basemat	11	7	5	6
UHS Makeup Water Intake Structure.	4	1	1	1

The average total settlement estimates above are in the range of about 1 to 4 in. except for the Common Basemat which is about 6 in. for the 11 ksf loading case and about 8 in. for the 15 ksf loading case. The maximum total settlement (at center of Common Basemat) is estimated to be about 10 in. resulting from the 15 ksf loading. Generally acceptable total and differential settlements for mat foundations supported on clays are typically in the range of 2.5 in. and 1.5 in., respectively, although tolerable total settlements as high as 4 in. have been suggested for mat foundations (Bowles, 1996). Differential settlement, however, is more critical than total settlement. Acceptable tilt for foundations is on the order of 1/300 (Bowles, 1996), although values as low as 1/750 have been stated for foundations that support machinery sensitive to settlement (Das, 1990).

From the above estimates, average foundation settlement for the UHS Makeup Water Intake Structure is within the acceptable range. Similar estimates for the Emergency Power Generating Building and the ESWS Cooling Towers vary by a factor of 2 to 4, depending on the structure location whose loading is impacted by the site grading and the magnitude of initial foundation loading. For the Common Basemat, an average settlement of about 8 in. was estimated for the 15 ksf loading. Differential settlements were estimated as the difference in settlement values at the center and edge of foundations. The estimated values are as follows: 1 in. to 2 in. for the ESWS Cooling Towers, 1 in. to 2 in. for the Emergency Power Generating Building Building, 2 in. to 4 in. for the Common Basemat, and practically zero for the UHS Makeup Water Intake Structure. From these values, tilt was estimated at about 1/600 for the ESWS Cooling Towers, 1/550 for the EDGB, and in the range of 1/600 to 1/1,200 for the Common Basemat foundations. Estimates of tilt for all structures, including the Common Basemat, are well within the acceptable limit of 1/300, however, they exceed the 1/750 for the special case of sensitive machinery, although the difference is not substantial. It is noted that the tabulated settlement estimates are based on the assumption of a flexible foundation; they do not take into account the effects of a thick, highly reinforced foundation mat which tends to mitigate differential settlements.

The estimated total settlement for the Common Basemat is largely impacted by the extreme foundation size and loading. It is expected that settlements will largely be taking place concurrent with construction; therefore, a majority of the settlements will have taken place prior to placing the equipment, piping, and the final finishes. Hence, post-construction total and differential settlements are expected to be lower than the values noted herein, particularly after accounting for foundation mat rigidity. This issue will be the subject of an in-depth study during the final geotechnical investigation for the detailed design phase of the project, including additional testing specifically focused on enhancing the available data on compressibility of the foundation soils. Further, this issue will be a common focus among foundation designers and engineers during detailed design of the Common Basemat to 1) provide a final estimate of settlement based on further engineering and modeling and 2) develop appropriate measures for control and monitoring of this foundation during construction. The final results will be incorporated into the foundation design requirements.

In general, the estimated foundation settlements are larger than those indicated for CCNPP Units 1 and 2, although no estimates or measured values are available for Units 1 and 2, as discussed in Section 2.5.4.10.1. The difference in settlement between the two areas is not due to differing soil conditions, as the soils are comparable. Rather, they are largely due to the difference in magnitude of net loading imposed by these structures on the soils, and foundation size. The influence of the larger and heavier Common Basemat for Unit 3 extends deeper, thereby influencing a larger volume of soils. As noted earlier, additional testing will be performed during the detailed design phase of the project, specifically focused on compression characteristics of Unit 3 soils, particularly in Stratum IIc Chesapeake Clay/Silt which is the dominant foundation material, and settlement magnitudes will be re-evaluated. Regardless of the final settlement estimates, however, all foundations will be designed to safely tolerate the anticipated total and differential settlements. Additionally, engineering measures will be incorporated into design for control of differential movements between adjacent structures, piping, and appurtenances sensitive to movement, consistent with final settlement estimates.

This includes the development and implementation of a monitoring plan that supplies information throughout construction and post-construction on ground heave, settlement, pore water pressure, foundation pressure, building tilt, and other necessary data. This information will provide a basis for comparison with estimated conditions and for projections of future performance.

2.5.4.10.3 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant below-ground walls. Seismic earth pressure diagrams are structure-specific and are, therefore, only addressed generically herein. Specific earth pressure diagrams will be developed for specific structures that will be based upon each structure's final configuration. Passive earth pressures are not addressed; they are ignored for conservatism for general purpose applications. The following soil properties were assumed for the backfill; an angle of shearing resistance of 30 degrees and a total unit weight of 120 pcf. These values were arbitrarily taken for the purpose of this generic exercise; actual values will be used during detailed design based on actual material properties. A surcharge pressure of 500 psf was assumed as well; however, this will be evaluated on a case-by-case basis that will be based on actual conditions during detailed design. Lateral pressures due to compaction are not included; these pressures will be controlled by compacting backfill with light equipment near structures.

Earthquake-induced horizontal ground accelerations are addressed by the application of $k_h \cdot g$. Vertical ground accelerations ($k_v \cdot g$) are considered negligible and were ignored (Lambe, 1969). A seismic acceleration of 0.125g was adopted for developing the generic earth pressure diagrams. Backgrounds on seismic accelerations are discussed in Section 2.5.4.8.2.

2.5.4.10.3.1 Static Lateral Earth Pressures

The static active earth pressure, p_{AS}, is estimated using (Lambe, 1969):

$$p_{AS} = K_{AS} \cdot \gamma \cdot z \qquad \text{Eq. 2.5.4-21}$$

where K_{AS}=Rankine coefficient of static active lateral earth pressure
γ =unit weight of backfill
z=depth below ground surface

The Rankine coefficient, K_{AS} , is calculated from

$$K_{AS} = \tan^2 (45- \phi'/2) \qquad \text{Eq. 2.5.4-22}$$

where,ϕ'=angle of shearing resistance of the backfill, in degrees.

The static at-rest earth pressure, p_{0S}, is estimated using (Lambe, 1969):

$$p_{0S} = K_{0S} \cdot \gamma \cdot z \qquad \text{Eq. 2.5.4-23}$$

where, K_{0S}=coefficient of at-rest static lateral earth pressure and is given by

$$K_{0S} = 1\text{-}\sin \phi' \qquad \text{Eq. 2.5.4-24}$$

Hydrostatic groundwater conditions are considered for active and at-rest static conditions. The lateral hydrostatic pressure is calculated by:

$p_W = \gamma_W \cdot z_w$ Eq. 2.5.4-25

where, p_w=hydrostatic lateral earth pressure
z_w=depth below ground water table
γ_W=62.4 pcf

2.5.4.10.3.2 Seismic Lateral Earth Pressures

The active seismic pressure, p_{AE}, is given by the Mononobe-Okabe equation (Whitman, 1991), represented by

$p_{AE} = \Delta K_{AE} \cdot \gamma \cdot (H-z)$ Eq. 2.5.4-26

where, ΔK_{AE}=coefficient of active seismic earth pressure = K_{AE}-K_{AS}
K_{AE}=Mononobe-Okabe coefficient of active seismic earth thrust
Eq. 2.5.4-27
γ =unit weight of backfill at depth z
z=depth below the top of the backfill
H=below-grade height of wall
$K_{AE}=\cos^2(\phi'-\theta)/\{\cos^2\theta\cdot[1+(\sin\phi' \sin(\phi'-\theta)/\cos(\theta))^{0.5}]^2\}$ Eq. 2.5.4-27
$\theta=\tan^{-1}(k_h)$

ΔK_{AE} may be estimated as $3/4 \cdot k_h$ for k_h values less than about 0.25g, regardless of the angle of shearing resistance of the backfill.

The at-rest seismic conditions are reported to be two to three times as large as the active earth pressures calculated by the Mononobe-Okabe equation (Whitman, 1991). Given that most below-grade walls actually yield to some extent, the actual "at-rest" seismic pressures may not be as high as previously indicated (Whitman, 1991). Thus, the "at-rest" seismic earth pressures will be taken as twice the active values, or, $\Delta K_{oE} = 2 \cdot \Delta K_{AE}$.

For well-drained backfills, seismic groundwater pressures need not be considered (Ostadan, 2004). Since granular backfill(s) will be used for the project, only hydrostatic pressures are taken into consideration, as given in Eq. 2.5.4-25. It is noted that seismic groundwater thrust greater than 35 percent of the hydrostatic thrust can develop for cases when k_h>0.3g (Whitman, 1990). Given the relatively low seismicity at the CCNPP Unit 3 site (k_h<0.3g), seismic groundwater considerations can be ignored.

2.5.4.10.3.3 Lateral Earth Pressures Due to Surcharge

Lateral earth pressures as a result of surcharge applied at the ground surface at the top of wall, p_{sur}, are calculated as follows:

$p_{sur} = K \cdot q$ Eq. 2.5.4-28

where, K=earth pressure coefficient; K_{AS} for active; K_0 for at-rest; ΔK_{AE} or ΔK_{oE} for seismic loading depending on the nature of loading, and q=uniform surcharge pressure.

2.5.4.10.3.4 Sample Earth Pressure Diagrams

Using the relationship outlined above and assumed backfill properties, sample earth pressures were estimated. Sample earth pressure diagrams are provided in Figures 2.5.4-55 and 2.5.4-56 for a wall height of 41 ft, level ground surface, and with groundwater level at 5 ft below the surface. The backfill is taken as granular soils, with ϕ'=30 degrees and γ=120 pcf. The horizontal ground acceleration is taken as 0.125g. A permanent uniform surcharge load of 500 psf is also included. Actual surcharge loads, backfill properties, and structural configurations are not known at this time; they will be developed as part of the detailed design stage. Actual earth pressure evaluations will be performed at that time for the design of below-grade walls, based on actual project conditions.

2.5.4.10.4 Selected Design Parameters

The field and laboratory test results are discussed in Section 2.5.4.2. The parameters employed for the bearing capacity, settlement, and earth pressure evaluations are based on the material characterization addressed in Section 2.5.4.2, and as summarized in Table 2.5.4-12. The parameters reflected in this table were conservatively chosen, as discussed in Section 2.5.4-2. In fact, some of the parameters may have been chosen over-conservatively, e.g., elastic moduli, resulting in apparently large settlement predictions for the Common Basemat. This issue will be the subject of further review and engineering. The groundwater level was chosen at elevation 80 ft, whereas this could be a "perched" condition only. The factor of safety utilized for bearing capacity of soils typically exceeds 3.0, whereas a value of 3.0 is commonly used. An angle of shearing resistance of 30 degrees was used for characterization of a structural backfill for earth pressure evaluations, which is considered conservative for granular fill compacted to 95 percent Modified Proctor compaction. Similarly, a seismic acceleration of 0.125g and a magnitude 6.0 earthquake were used in the evaluations, which are higher than the 0.084g zero depth peak ground acceleration and 5.5 magnitude indicated by the seismic analyses, therefore resulting in conservative estimates.}

2.5.4.11 Design Criteria

Section 3.8.5 provides criteria, references, and design methods used in static and seismic analysis and design of foundations, including an explanation of computer programs used in the analyses and a description of soil loads on subsurface facilities.

2.5.4.12 Techniques to Improve Subsurface Conditions

(Major structures will derive support from the very dense cemented soils or compacted structural backfill. Given the planned foundation depths and soil conditions at these depths, as shown in Figures 2.5.4-28 through 2.5.4-32, no special ground improvement measures are warranted. Ground improvement will be limited to excavation of unsuitable soils, such as suspected fill or loose/soft soils, and their replacement with structural backfill. It will also include proof-rolling of foundation subgrade for the purpose of identifying any unsuitable soils for further excavation and replacement, which further densifies the upper portions of the subgrade. In absence of subsurface conditions at the site that require ground improvement, ground control, i.e., maintaining the integrity of existing dense or stiff foundation soils, will be the primary focus of earthworks during foundation preparation. These measures will include such steps as groundwater control, use of appropriate measures and equipment for excavation and compaction, subgrade protection, and other similar measures.

Given that the project detailed design phase is yet to begin, and that further explorations are needed to complement the geotechnical investigation that was performed for the COL, ground conditions may be present outside the powerblock area that could warrant specialty ground modification measures. These conditions will be evaluated on a case-by-case basis, given the subsurface conditions and structural requirements, as warranted.}

2.5.4.13 References

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Table 2.5.4-1 Summary of Field Testing Quantities
(Page 1 of 1)

Drill Rig	Standard	Quantity
Test Borings	ASTM D1586/1587	145
Observation Wells	ASTM D5092	40
CPT Soundings	ASTM D5778	50*
Suspension P-S Velocity Logging	EPRI TR-102293	10
Test Pits	N/A	20
Field Electrical Resistivity Arrays	ASTM G57/IEEE 81	4
SPT Hammer Energy Measurements	ASTM D4633	5

Note:

* Not including additional off-set soundings performed

Table 2.5.4-2 Summary Thickness of Various Soil Strata
(Page 1 of 1)

Stratum I Terrace Sand			Chesapeake			Nanjemoy
From Existing Ground Surface (ft)		Below elevation 85 (ft)	Stratum IIa Clay/Silt (ft)	Stratum IIb Cemented Sand (ft)	Stratum IIc Clay/Silt (ft)	Stratum III Sand (ft)
CCNPP Unit 3						
Maximum	51	20	35	73	190*	>101*
Minimum	2	0	4	57	190*	>101*
Average	21	14	20	66	190*	>101*
Construction Laydown Area 1 (CLA1)						
Maximum	59	27	26	65	190*	>119*
Minimum	2	11	8	24	190*	>119*
Average	34	18	20	58	190*	>119*
CCNPP Unit 3 and CLA1 Combined						
Maximum	59	27	35	73	190	>119
Minimum	2	0	4	24	190	>101
Average	27	16	20	55	190	>110
Cooling Tower Area						
Maximum	69	38	31	65	>13*	---
Minimum	2	5	5	>1	>13*	---
Average	30	25	19	>30	>13*	---
Switchyard Area						
Maximum	53	27	36	>63	---	---
Minimum	7	14	4	>8	---	---
Average	30	21	22	>31	---	---
Entire Site						
Maximum	69	38	36	73	190	>119
Minimum	2	0	4	28	190	>101
Average	28	19	20	60	190	>110

Note:

* Data based on a single boring

Table 2.5.4-3 Summary Termination Elevation of Various Soil Strata (Page 1 of 1)

	Stratum I Terrace Sand (ft)	Chesapeake			Nanjemoy
		Stratum IIa Clay/Silt (ft)	Stratum IIb Cemented Sand (ft)	Stratum IIc Clay/Silt (ft)	Stratum III Sand (ft)
CCNPP Unit 3					
Maximum	80	56	3	-208*	---
Minimum	47	38	-31	-208*	---
Average	66	47	-19	-208*	---
Construction Laydown Area 1 (CLA1)					
Maximum	74	50	-8	-211*	---
Minimum	35	27	-23	-211*	---
Average	63	45	-14	-211*	---
CCNPP Unit 3 and CLA1 Combined					
Maximum	80	56	3	-208	---
Minimum	35	27	-31	-211	---
Average	65	46	-17	-209	---
Cooling Tower Area					
Maximum	66	46	-24	---	---
Minimum	46	26	-24	---	---
Average	56	36	-24	---	---
Switchyard Area					
Maximum	71	67	---	---	---
Minimum	58	30	---	---	---
Average	64	42	---	---	---
Entire Site					
Maximum	80	72	-8	-208	---
Minimum	7	21	-31	-211	---
Average	61	42	-18	-209	---

* Data based on a single boring

Note: Only data from borings that fully penetrated each stratum was considered for determination of the maximum, minimum, and average termination elevations shown. For instance, a termination elevation for Stratum III is not provided since no boring reached the bottom of this stratum, as indicated by "---" .

Table 2.5.4-4 Summary of Measured (Uncorrected) SPT N-Values for Various Soil Strata (Page 1 of 1)

		Stratum I Terrace Sand (blows/ft)	Chesapeake			Nanjemoy
			Stratum IIa Clay/Silt (blows/ft)	Stratum IIb Cemented Sand (blows/ft)	Stratum IIc Clay/Silt (blows/ft)	Stratum III Sand (blows/ft)
CCNPP Unit 3	Maximum	70	46	100	100	100
	Minimum	0	1	4	12	34
	Average	10	9	45	23	64
CLA1	Maximum	43	45	100	39	100
	Minimum	0	1	0	10	28
	Average	12	9	45	20	56
CCNPP Unit 3 and CLA1 Combined	Maximum	70	46	100	100	100
	Minimum	0	1	0	10	28
	Average	11	9	45	21	61
Switchyard Area	Maximum	27	19	100	---	---
	Minimum	2	4	7	---	---
	Average	9	10	35	---	---
Cooling Tower Area	Maximum	49	26	100	25	---
	Minimum	0	1	9	19	---
	Average	12	10	38	23	---
Entire Site	Maximum	70	46	100	100	100
	Minimum	0	1	0	10	28
	Average	11	10	41	22	61

Note: A cut off SPT N-value of 100 blows/ft is shown whenever SPT refusal (50 blows/6" or less) was measured or the linearly extrapolated N-value exceeded 100 blows/ft.

Table 2.5.4-5 Summary of Hammer-Rod Energy Measurements (Page 1 of 1)

Drill Rig	Measurement in Boring No.	ETR range (%)	Average ETR (%)	Energy Adjustment (ETR% / 60%)
Failing 1500	B-401	67-88	78	1.30
CME 550X ATV	B-403	73-92	84	1.40
CME 750 ATV	B-404	78-90	87	1.45
CME 75 Truck	B-409	69-90	84	1.40
Deidrich D50 ATV	B-744	73-84	81	1.35

Note:

ETR= Percentage of theoretical hammer energy measured in the field

Table 2.5.4-6 Summary of Adjusted SPT N-Values Based on Energy Measurements (Page 1 of 1)

Stratum	Adjusted Minimum N-value (blows/ft)	Adjusted Maximum N-value (blows/ft)	Adjusted Average N-value (blows/ft)	Adopted N-value for Engineering Purposes (blows/ft)
I – Terrace Sand	0	91	16	15
IIa – Ches. Clay/Silt	1	64	13	10
IIb – Ches. Cemented sand	0	100	48	45
IIc – Ches. Clay/Silt	14	100	29	25
III – Nanjemoy Sand	36	100	72	70

Note: Adjusted values are for "Entire Site" shown in Table 2.5.4-4.

Table 2.5.4-7 Summary of Laboratory Tests and Quantities
(Page 1 of 1)

Identification and Index Testing		Quantity	Standard/Method Used	Regulatory Guide 1.138 Recommended
	Unified Soil Classification System (USCS)	NA	ASTM D2487 (ASTM, 2006a) ASTM D2488 (ASTM, 2006d)	ASTM D2487-00
	Sieve and Hydrometer Analysis	398	ASTM D422 (ASTM, 2002a) ASTM D6913 (ASTM, 2004b)	ASTM D422-63(98)
	Atterberg Limits	330	ASTM D4318 (ASTM, 2005b)	ASTM D4318--00
	Natural Moisture Content	812	ASTM D2216 (ASTM, 2005c)	ASTM D2216-98
	Specific Gravity	77	ASTM D854 (ASTM, 2006b)	ASTM D854-00
	Organic Content	9	ASTM D2974 (ASTM, 2000d)	ASTM D2974-00
Compaction and Strength Tests				
	Moisture-Density Relationship		ASTM D1557 (ASTM, 2002c)	ASTM D1557-00
	California Bearing Ratio	12	ASTM D1883 (ASTM, 2005d)	
	Unconfined Compression	22	ASTM D2166 (ASTM, 2006c)	ASTM D2166-98
	Unconsolidated-Undrained Triaxial Compression	38	ASTM D2850 (ASTM, 2003)	ASTM D2850-95 (99)
	Consolidated-Undrained Triaxial compression	10	ASTM D4767 (ASTM, 2004c)	ASTM D4767-95
	Direct Shear	19	ASTM D3080 (ASTM, 2004d)	ASTM D3080-98
Compressibility Tests	Consolidation	50	ASTM D2435 (ASTM, 2004e)	ASTM D2435--96
Chemical Testing – Soils				
	pH	77	ASTM D4972 (ASTM, 2001b)	*
	Chloride	77	EPA 300.0 (EPA, 1993)	*
	Sulfate	77	EPA 300.0 (EPA, 1993)	*
	Cation Exchange Capacity	NA	(ECL, 2007)	Not Specified
Proctor Compaction		28	ASTM D1587	Not Specified
Unit Weight		78	Not specified	Not specified
Resonant Column Torsional Shear (RCTS)			Not specified	Not specified

* Regulatory Guide 1.138 states that Manual of Soil Laboratory Testing Volume 1, 1992 information on the most widely used clinical test for soils and groundwater.
Results of Cation Exchange Capacity tests are addressed with the groundwater chemistry data in Subsection 2.4.13.

Table 2.5.4-8 Summary Average Values of Laboratory Index Properties (Page 1 of 2)

Atterberg Limits

Stratum	No. of Tests	Average LL (%)	Average PI (%)	Average WC (%)	USCS Classification**	Adopted PI for Engineering Purposes (%)
I - Terrace Sand	31	NP	NP	15	SP-SM, SC, SM, CL, SW-SM, CH, ML, MH	NP
IIa- Chesapeake Clay/Silt	67	57	35	32	CH, MH, CL, SM, SC-SM, OH	35
IIb- Chesapeake Cemented Sand	67	46	22	34	SM, ML, MH, CH, CL, SP-SM, SC, OH	20
IIc- Chesapeake Clay/Silt	88	94	44	54	MH, CH, SM, CL, OH	45
III- Nanjemoy Sand	7	59	27	30	SC, SM, CH, MH	30

Fines Content (% Passing No. 200 Sieve)

Stratum	No. of Tests	Average Fines Content (%)	Adopted Value for Engineering Purposes (%)
I -Terrace Sand	85	19	20
IIa - Chesapeake Clay/Silt	72	77	75
IIb - Chesapeake Cemented Sand	115	24	25
IIc - Chesapeake Clay/Silt	82	54	50
III - Nanjemoy Sand	10	19	20

Unit Weight

Stratum	No. of Tests	Average Unit Weight (pcf)	Adopted Value for Engineering Purposes (pcf)
I - Terrace Sand	3	120	120
IIa - Chesapeake Clay/Silt	40	116	115
IIb - Chesapeake Cemented Sand	16	118	120
IIc - Chesapeake Clay/Silt	19	107	110
III – Nanjemoy Sand	0	N.A.	120*

Notes:

N.A. – Not Available
* – Estimated
** – Classification legend on Page 2.

Table 2.5.4-8 Summary Average Values of Laboratory Index Properties (Page 2 of 2)

Group Symbols	Typical Names
SW	Well graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, poorly graded sand-silt mixtures
SC	Clayey sands, poorly graded sand-clay mixtures
ML	Inorganic silts and very find sands, rock flour, silty or clayey fine sands with slight plasticity
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OH	Organic clays of medium to high plasticity

Table 2.5.4-9 Summary Laboratory Strength Results
(Page 1 of 2)

Summary - Direct Shear Test Results

Boring	elevation (ft)	USCS	ϕ' (deg.)	c' (tsf)	Boring	elevation (ft)	USCS	ϕ' (deg.)	c' (tsf)
Stratum I – Terrace Sand					**Stratum IIb – Chesapeake Cemented Sand**				
B-743	78.1	CL	29.2	0.3	B-724	21.5	OL	27.5	0.6
Stratum IIa – Chesapeake Clay/Silt					B-440	3.3	SC	30.3	0.4
B-319	67.4	CL	24.9	0.4	B-420	-2.9	SC	34	0.2
B-320	65.9	SC	26	0.2	**Maximum**			34	0.6
B-735	61.2	CH	27.2	0.4	**Minimum**			27.5	0.2
B-319	57.7	CH	20.8	0.7	**Average**			31	0.4
B-326	57.6	OH	19	0.4					
B-433	57.0	CH	20.2	0.7	**Stratum IIc – Chesapeake Clay/Silt**				
B-320	56.4	CH	21.6	0.7					
B-316	52.6	CL	30.1	0.3	B-313	-44.0	CL	29	0.8
B-427	50.8	OH	29.2	0.4	B-307	-61.1	SC	35	0
B-737	51.0	CH	22.7	0.4	B-423	-78.9	MH	18.5	1.7
B-413	47.9	CH	31.4	0.5	B-401	-102.3	CH	18.9	2.3
Maximum			31.4	0.7	**Maximum**			35	2.3
Minimum			19	0.2	**Minimum**			18.5	0
Average			25	0.5	**Average**			25	1.6

Table 2.5.4-9 Summary Laboratory Strength Results
(Page 2 of 2)

Summary – CIU-bar Test Results

			Effective		Total	
Boring	elevation (ft)	USCS	ϕ' (deg.)	c' (tsf)	ϕ (deg.)	c (tsf)
Stratum IIa – Chesapeake Clay/Silt						
B-320	65.9	SC	27.9	0.3	13.3	0.6
B-317	63.9	CL	31	0.2	17	0.4
B-316	52.6	CL	32.1	0.5	12.5	1.0
B-414	51.2	CH	20	0.7	10.4	1.0
B-433	47	CH/CL	19.3	0.3	8.3	0.4
B-317	43.9	CL	33.5	0.3	19.5	0.6
		Maximum	33.5	0.7	19.5	1.0
		Minimum	19.3	0.2	8.3	0.4
		Average	27	0.4	14	0.7
Stratum IIb – Chesapeake Cemented Sand						
B-328	10.8	OH	34.6	0.0	13.4	1.7
B-423	6.6	SP-SC	27	0.8	14.1	2.3
B-321	-4.8	SM	30	0.5	20	1.0
		Maximum	34.6	0.8	20	2.3
		Minimum	27	0.5	13.4	1.0
		Average	31	0.5	16	1.7
Stratum IIc – Chesapeake Clay/Silt						
B-420	-65.9	OH	29.1	1.0	15.4	1.5

Table 2.5.4-10 Summary Consolidation Properties
(Page 1 of 1)

Laboratory Testing

Stratum	No. of Tests		C_r (1)	C_c (1)	e_o	Pp' (tsf)	OCR
Stratum I – Terrace Sand	2	Maximum	0.018	0.146	0.82	6	4.5
		Minimum	0.018	0.071	0.78	4	3.7
		Average	0.018	0.108	0.80	5	4.1
Stratum IIa – Ches. Clay/Silt	25	Maximum	0.126	0.915	1.95	18.5	12.5
		Minimum	0.018	0.071	0.78	4	1.2
		Average	0.054	0.526	1.09	9.1	5.6
Stratum IIb – Ches. Cemented Sand	9	Maximum	0.137	1.092	1.73	14.2	11.6
		Minimum	0.005	0.109	0.70	1.1	0.4
		Average	0.033	0.396	1.05	9	5.2
Stratum IIc Ches. Clay/Silt	14	Maximum	0.152	2.052	2.80	23	5.9
		Minimum	0.004	0.276	0.93	7	1.2
		Average	0.041	0.905	1.53	15.5	3.3

CPT Data Interpretation

Stratum	Min. OCR	Max. OCR	Average OCR
Stratum I - Terrace Sand	0.6	10	5.3
Stratum IIa – Ches. Clay/Silt	0.6	10	5.9
Stratum IIb – Ches. Cemented Sand	0.8	10	7.1
Stratum IIc Ches. Clay/Silt	1.2	10	9.2

Average Values Adopted for Engineering Purposes

Stratum	OCR	Pp' (tsf)
Stratum I - Terrace Sand	4	4
Stratum IIa – Ches. Clay/Silt	4	6
Stratum IIb – Ches. Cemented Sand	3	8
Stratum IIc Ches. Clay/Silt	3	14

Notes:
C_r = recompression index
C_c = compression index
e_o = void ratio
Pp' = preconsolidation pressure
OCR = overconsolidation ratio
(1) values are void ratio-based

Table 2.5.4-11 High Strain Elastic and Shear Moduli Estimation (Page 1 of 1)

High Strain Elastic Modulus (E)

Relationship	Stratum I Terrace Sand (tsf)	Chesapeake			Nanjemoy
		Stratum IIa Clay/Silt (tsf)	Stratum IIb Cemented Sand (tsf)	Stratum IIc Clay/Silt (tsf)	Stratum III Sand (tsf)
E = 18 N	270	---	810	---	1,260
$E_u = 450\ s_u$	---	450	---	900	1,800
$E_{.375\%} = f\ (V_s)$	302	---	1,134	---	1,879
$E_{.375\%} = f\ (PI)$	---	1,766	---	2,477	---
$E_{.375\%} = f\ (s_u)$	---	580	---	1,160	2,080

Adopted E-Values for Engineering Purposes

E (tsf)	280	510	970	1,030	1,750

High Strain Shear Modulus (G)

Relationship	Stratum I Terrace Sand (tsf)	Chesapeake			Nanjemoy
		Stratum IIa Clay/Silt (tsf)	Stratum IIb Cemented Sand (tsf)	Stratum IIc Clay/Silt (tsf)	Stratum III Sand (tsf)
$G_{.375\%} = f\ (V_s)$	116	---	436	---	723
$G_{.375\%} = f\ (PI)$	---	609	---	853	---
$G_{.375\%} = f\ (s_u)$	---	200	---	400	---
$G_{.375\%}=E/[2(1+\mu)]$	108	176	373	355	673

Adopted G-Values for Engineering Purposes

G (tsf)	110	180	400	370	700

Table 2.5.4-12 Summary Average Soils Engineering Properties[1]
(Page 1 of 1)

Parameter	Stratum				
	I Terrace Sand	IIa Chesapeake Clay/Silt	IIb Chesapeake Cemented Sand	IIc Chesapeake Clay/Silt	III Nanjemoy Sand
Average thickness, feet	20	20	60	190	>110
USCS symbol (Predominant class. underlined)	SP-SM, SM, SP, SC	CH, MH, CL, SM, SC-SM, OH	SM, SC, SP-SM, SP , OH	MH, CH, SM, CL, OH	SC, SM, MH, CH
Natural water content (WC), %	15	32	34	54	30
Moist unit weight □$_{moist}$), pcf	120	115	120	110	120
Fines content , %	20	75	20	50	20
Liquid limit (LL), %	NP	57	46	94	60
Plasticity index (PI), %	NP	35	20	45	30
Measured SPT N-value, bpf	11	10	41	22	61
Adjusted SPT N_{60}-value, bpf	15	10	45	25	70
Shear Wave Velocity, ft/sec	790	1,100	1,530	1,250	1,970
Undrained shear strength (s_u), tsf	N/A[2]	1.0	N/A[2]	2.0	4.0
Friction angle (Ø'), degree Cohesion (c'), tsf	32 0	26 0.4	34 0	27 1.0	40 0
Elastic modulus (high strain) (E_s), tsf	280	510	970	1,030	1,750
Shear modulus (high strain) (G_s), tsf	110	180	400	370	700
Coefficient of Subgrade Reaction (k_1), tcf (for 1-ft. sq. area)	75	75	300	150	N/A[2]
Earth Pressure Coefficients					
Active (K_a)	0.3	0.4	0.3	0.4	N/A[2]
Passive (K_p)	3.3	2.6	3.5	2.6	N/A[2]
At Rest (K_0)	0.5	0.8	0.5	0.7	N/A[2]
Coefficient of Sliding	0.40	0.35	0.45	0.40	N/A[2]
Consolidation Properties					
C_c [C_r] (void ratio-based)	0.108 [0.018]	0.526 [0.054]	0.396 [0.033]	0.905 [0.041]	N/A[2]
Void Ratio, e	0.80	1.09	1.05	1.53	N/A[2]
P_p', tsf [OCR]	4 [4]	6 [4]	8 [3]	14 [3]	N/A[2]

Notes.

(1) The values tabulated above are for use as guideline only. Reference should be made to specific boring and CPT logs and laboratory test results for appropriate modifications at specific locations and for specific calculations.

(2) N/A indicates that the properties were either not measured or are not applicable.

Table 2.5.4-13 Summary Undrained Shear Strength for Cohesive Soils (Page 1 of 1)

From Correlation with SPT N-Values

Stratum	SPT N-Value (blows/ft)	S_u (tsf)
Stratum IIa – Ches. Clay/Silt	10	0.63
Stratum IIc – Ches. Clay/Silt	25	1.6
Stratum III – Nanjemoy Clayey Sand	70	4.4*

From Laboratory UU and UC Tests

Stratum	Max. S_u (tsf)	Min. S_u (tsf)	Average S_u (tsf)
Stratum IIa – Ches. Clay/Silt	2.4	0.3	1.1
Stratum IIc – Ches. Clay/Silt	5.2	0.2	2.2

From Correlation with CPT Results

Stratum	Max. S_u (tsf)	Min. S_u (tsf)	Average S_u (tsf)
Stratum IIa – Ches. Clay/Silt	9.3	0.7	1.6
Stratum IIc – Ches. Clay/Silt	9.6	1.4	4.7

Adopted Values for Engineering Purposes

Stratum	S_u (tsf)
Stratum IIa – Ches. Clay/Silt	1.0
Stratum IIc – Ches. Clay/Silt	2.0
Stratum III – Nanjemoy Clayey Sand	4.0*

Note:

* Assuming “undrained” behavior

Table 2.5.4-14 Summary Soils Chemical Test Results
(Page 1 of 1)

From CCNPP Unit 1 and 2 Exploration

	Stratum IIa Chesapeake Clay/Silt	Stratum IIb Chesapeake Cemented Sand
pH (unit)		
- min. value	6.2	7.1
- max. value	7.1	8.0
- average value	6.7	7.5
Sulfate (ppm)		
- min. value	1,800	600
- max. value	2,000	600
- average value	1,900	600
Chloride (ppm)		
- min. value	20	10
- max. value	110	60
- average	60	57

From CCNPP Subsurface Investigation

	No. of Tests		pH (CaCl)	pH (H_2O)	Sulfate (%)	Chloride (ppm)
Stratum I Terrace Sand	21	Maximum	6.7	7.6	2.570	48.6
		Minimum	2.6	2.7	0.001	<10
		Average	4.6	5.5	0.236	<12
Stratum IIa Ches. Clay/Silt	18	Maximum	4.9	5.8	2.590	10.7
		Minimum	2.6	2.5	0.006	<10
		Average	3.1	3.5	0.914	<10
Stratum IIb Ches. Cemented Sand	37	Maximum	7.4	8	3.130	145
		Minimum	2.4	2.5	0.010	<10
		Average	5.7	5.8	0.567	<22
Stratum IIc Ches. Clay/Silt	1		6.6	7	0.196	<10

Table 2.5.4-15 Summary Field Electrical Resistivity Test Results
(Page 1 of 1)

Measured Data ("apparent" values)

Location		R-1	R-2	R-3	R-4	Values in Ohm-m Min.	Max.	Average
Ground Surface El. (ft)		85.5	85.5	89.1	99.4	Min.	Max.	Average
Array Spacing	1.5 ft	1,210	1,520	3,070	471	471	3,070	1,568
	3 ft	2,480	2,410	3,750	640	640	3,750	2,320
	5 ft	3,220	2,780	4,550	660	660	4,550	2,803
	7.5 ft	3,110	2,890	5,440	806	806	5,440	3,062
	10 ft	2,490	2,700	6,240	1,130	1,130	6,240	3,140
	15 ft	1,870	2,780	5,370	1,340	1,340	5,370	2,840
	20 ft	1,570	1,960	4,100	1,790	1,570	4,100	2,355
	30 ft	1,310	2,060	1,960	1,640	1,310	2,060	1,743
	40 ft	739	1,590	1,010	1,280	739	1,590	1,155
	50 ft	314	1,080	415	975	314	1,080	696
	100 ft	45	487	69	463	45	487	266
	200 ft	37	116	38	57	37	116	62
	300 ft	48	76	31	41	31	76	49

Modeled Data ("true" values)

Location	Depth of Layer (ft)	Resistivity (Ohm-m)
R-1	0.5	428
	2.2	12,318
	6.3	966
	15	3,114
	43.1	51
	119.4	17
	N/A	94
R-2	0.5	639
	7.6	3,648
	17.9	2,247
	62.9	1,184
	N/A	68
R-3	2.4	2,952
	10.6	11,930
	59.8	128
	N/A	30
R-4	4.6	494
	13.8	5,040
	39.9	891
	53.2	375
	N/A	36

An Approximate Correlation with Depth

Stratum	Depth Range (ft)
I. Terrace Sand	upper 20 ft
IIa. Ches. Clay/Silt	20 - 40 ft
IIb. Ches. Cem. Sand	40 - 100 ft
IIc. Ches. Clay/Silt	below 100 ft

Table 2.5.4-16 Guidelines for Soil Chemistry Evaluation
(Page 1 of 1)

Soil Corrosiveness

	Range for Steel Corrosiveness				
	Little Corrosive	Mildly Corrosive	Moderately Corrosive	Corrosive	Very Corrosive
Resistivity (ohm-m)	>100	20-100 50-100(>30	10-20 20-50	5-10(7-20	<5 <7
pH		>5.0 and <10		5.0-6.5	<5.0
Chlorides (ppm)		<200		300-1,000	>1,000

Soil Aggressiveness

Recommendations for Normal Weight Concrete Subject to Sulfate Attack			
Concrete Exposure	Water Soluble Sulfate (SO_4) in Soil, Percent	Cement Type	Water Cement Ratio (Maximum)
Mild	0.00-0.10	---	---
Moderate	0.10-0.20	II, IP(MS), IS(MS)	0.5
Severe	0.20-2.0	V[1]	0.45
Very Severe	Over 2.0	V with pozzolan	0.45

Notes:

[1] Or a blend of Type II cement and a ground granulated blast furnace slag or a pozzolan that gives equivalent sulfate resistance.

Table 2.5.4-17 Summary As-Conducted Boring Information
(Page 1 of 4)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
B-301	403.0	-308.5	217024.06	960815.05	94.51	9/15/2006
B-302	200.0	-123.6	217122.24	960766.98	76.41	9/15/2006
B-303	200.0	-112.6	217016.91	960867.69	87.40	9/15/2006
B-304	200.0	-132.0	217188.61	960896.88	68.00	9/15/2006
B-305	151.5	-79.5	217166.25	960686.74	72.01	9/15/2006
B-306	150.0	-31.4	217024.31	960681.82	118.58	9/15/2006
B-307	201.5	-82.2	216955.27	960690.13	119.28	9/15/2006
B-308	150.0	-42.9	216906.69	960771.28	107.10	9/15/2006
B-309	150.0	-49.9	216949.24	960890.70	100.06	9/15/2006
B-310	100.0	-8.4	217081.40	960616.60	91.62	5/15/2006
B-311	150.0	-91.6	217268.61	960771.76	58.43	9/15/2006
B-312	99.5	-44.2	217293.00	960740.00	55.27	5/15/2006
B-313	150.0	-99.3	217372.34	960713.67	50.73	9/15/2006
B-314	100.0	-47.2	217321.89	960654.50	52.78	9/15/2006
B-315	100.0	-34.5	217184.68	960559.43	65.54	9/15/2006
B-316	100.0	8.1	216767.16	960864.35	108.07	9/15/2006
B-317	100.0	-5.6	217094.70	961249.20	94.42	5/15/2007
B-318	200.0	-102.2	217019.30	961227.20	97.82	5/15/2006
B-319	100.0	2.9	216963.62	961123.01	102.87	9/15/2006
B-320	150.0	-43.6	216943.50	961044.10	106.43	5/15/2006
B-321	150.0	-79.3	217152.50	960333.20	70.66	5/25/2006
B-322	100.0	-10.1	217170.03	960202.65	89.87	9/15/2006
B-323	200.0	-92.5	217027.97	960060.86	107.48	9/15/2006
B-324	101.5	3.7	216906.40	960114.44	105.20	9/15/2006
B-325	100.0	-15.0	216948.98	960549.73	84.97	9/15/2006
B-326	100.0	3.1	216859.22	960652.25	103.11	9/15/2006
B-327	150.0	-63.1	216865.70	960573.37	86.92	9/15/2006
B-328	150.0	-73.7	216828.86	960493.21	76.29	9/19/2006
B-329	100.0	-25.2	216800.38	960379.43	74.83	9/19/2006
B-330	100.0	-14.5	216715.40	960523.70	85.46	9/15/2006
B-331	100.0	-31.7	216970.57	960481.79	68.32	9/15/2006
B-332	100.0	-34.6	217127.42	960400.52	65.40	9/15/2006
B-333	98.8	-9.3	216657.04	960386.24	89.49	9/15/2006
B-334	100.0	-13.3	216515.53	960556.61	86.75	9/15/2006
B-335	100.0	-0.5	216732.70	960703.30	99.47	5/15/2006
B-336	100.0	-3.1	216632.91	960750.27	96.87	9/15/2006
B-337	100.0	-28.2	217257.88	960264.41	71.77	9/15/2006
B-338	99.6	-1.6	217121.10	960150.10	97.97	5/25/2006
B-339	100.0	-8.0	217095.21	960211.99	91.96	9/15/2006

Table 2.5.4-17 Summary As-Conducted Boring Information (Page 2 of 4)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
B-340	100.0	-15.4	217171.34	961225.22	84.57	9/15/2006
B-341	100.5	-2.3	217036.40	961104.48	98.16	9/15/2006
B-401	401.5	-329.4	216344.12	961516.81	72.06	9/15/2006
B-402	200.0	-117.8	216405.10	961463.50	82.22	5/15/2006
B-403	200.0	-136.6	216305.80	961562.90	63.41	5/15/2006
B-404	200.0	-132.1	216441.34	961596.49	67.90	9/21/2006
B-405	150.0	-28.0	216487.38	961408.73	122.00	9/15/2006
B-406	150.0	-31.6	216315.62	961352.01	118.36	9/15/2006
B-407	200.0	-118.4	216238.96	961412.45	81.63	9/15/2006
B-408	150.0	-81.6	216261.74	961482.04	68.41	9/15/2006
B-409	150.0	-88.5	216253.80	961614.80	61.55	4/20/2006
B-410	55.0	64.1	216374.30	961323.70	119.05	4/20/2006
B-410A*	98.7	20.4	216381.30	961323.70	119.05	4/20/2006
B-411	150.0	-68.6	216556.31	961517.19	81.45	9/15/2006
B-412	98.9	-6.7	216589.24	961495.42	92.17	9/15/2006
B-413	150.0	-27.1	216694.88	961413.25	122.90	9/15/2006
B-414	100.0	21.2	216630.18	961354.48	121.20	9/15/2006
B-415	98.7	20.6	216480.90	961264.20	119.26	4/20/2006
B-416	100.0	-13.8	216084.50	961596.34	86.22	9/15/2006
B-417	101.5	-52.3	216435.75	961901.11	49.23	9/15/2006
B-418	200.0	-156.3	216340.25	961976.71	43.67	9/22/2006
B-419	100.0	-44.7	216267.83	961895.60	55.29	9/21/2006
B-420	150.0	-87.4	216213.53	961670.44	62.57	9/15/2006
B-421	150.0	-34.4	216497.56	961019.77	115.58	9/15/2006
B-422	100.0	4.0	216478.23	960915.01	104.02	9/15/2006
B-423	201.5	-91.4	216331.76	960850.21	110.14	9/15/2006
B-424	100.0	18.9	216263.30	960818.60	118.92	4/26/2006
B-425	101.5	16.9	216247.50	961274.70	118.43	4/20/2006
B-426	100.0	-16.3	216193.04	961386.57	83.73	9/21/2006
B-427	150.0	-33.7	216164.05	961272.73	116.27	9/19/2006
B-428	150.0	-35.9	216109.19	961210.06	114.11	9/19/2006
B-429	100.0	3.7	216087.85	961119.27	103.66	9/19/2006
B-430	100.0	2.5	216006.88	961193.12	102.48	9/19/2006
B-431	101.5	16.9	216271.10	961177.30	118.43	4/20/2006
B-432	100.0	18.6	216399.00	961139.10	118.62	4/20/2006
B-433	100.0	-2.5	215963.80	961107.50	97.49	4/27/2006
B-434	100.0	5.2	215827.10	961244.30	105.15	5/2/2006
B-435	100.0	7.7	216020.06	961404.74	107.71	9/15/2006
B-436	100.0	8.3	215923.92	961441.55	108.29	9/22/2006

Table 2.5.4-17 Summary As-Conducted Boring Information (Page 3 of 4)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
B-437	100.5	10.1	216521.76	960968.80	110.63	9/15/2006
B-438	6.5	99.5	216414.91	960848.90	105.95	9/28/2006
B-438A	100.0	6.6	216411.98	960867.31	106.59	9/28/2006
B-439	100.0	13.8	216340.49	960948.68	113.80	9/15/2006
B-440	100.0	-43.7	216349.47	961813.66	56.34	9/21/2006
B-701	75.0	-66.3	219485.54	960507.60	8.66	9/21/2006
B-702	50.0	-39.7	218980.62	961183.23	10.33	9/21/2006
B-703	100.0	-54.6	218171.00	960957.01	45.42	9/21/2006
B-704	50.0	-10.4	217991.06	960926.05	39.58	9/21/2006
B-705	50.0	-3.3	217581.30	960917.90	46.75	4/19/2006
B-706	50.0	27.4	217140.14	961339.74	77.42	9/21/2006
B-707	50.0	17.4	217396.98	961481.84	67.38	9/21/2006
B-708	100.0	-62.7	217585.84	961810.64	37.35	9/28/2006
B-709	50.0	-18.8	217642.82	961978.18	31.25	9/28/2006
B-710	75.0	-27.0	217542.51	962136.88	47.96	9/28/2006
B-711	50.0	3.0	216755.70	961743.50	53.01	4/19/2006
B-712	50.0	-7.6	216506.16	961997.56	42.41	9/22/2006
B-713	50.0	8.0	216117.68	962283.16	57.99	9/28/2006
B-714	50.0	66.0	215705.73	962034.37	116.02	10/16/2006
B-715	50.0	36.3	214951.76	962639.59	86.29	10/17/2006
B-716	49.5	32.9	215003.21	961364.57	82.35	10/16/2006
B-717	50.0	40.7	214302.45	962349.27	90.72	10/17/2006
B-718	50.0	67.5	214130.52	961929.05	117.47	10/18/2006
B-719	49.4	25.8	213978.69	961500.20	75.23	10/18/2006
B-720	75.0	-1.5	215674.48	962378.47	73.47	9/28/2006
B-721	100.0	1.3	215545.80	962462.10	101.30	5/4/2006
B-722	73.9	25.9	215386.10	962467.00	99.78	5/4/2006
B-723	75.0	15.0	215108.00	963000.80	90.02	4/28/2006
B-724	100.0	-3.0	214780.00	963106.20	96.97	4/28/2006
B-725	75.0	-16.0	214664.30	963219.40	59.02	4/28/2006
B-726	75.0	3.3	215564.67	961709.57	78.33	10/16/2006
B-727	100.0	4.9	215300.85	961884.98	104.88	10/16/2006
B-728	75.0	37.3	215163.63	961910.05	112.30	10/16/2006
B-729	75.0	42.3	214861.87	962454.60	117.28	10/17/2006
B-730	75.0	40.4	214728.50	962523.84	115.36	10/17/2006
B-731	99.3	16.4	214546.48	962547.88	115.67	10/17/2006
B-732	75.0	15.7	215034.10	961594.70	90.72	5/11/2006
B-733	100.0	-12.1	214866.80	961697.70	87.92	5/11/2006
B-734	75.0	30.7	214589.60	961812.50	105.73	5/9/2006

Table 2.5.4-17 Summary As-Conducted Boring Information (Page 4 of 4)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
B-735	75.0	16.2	214805.48	961021.83	91.20	10/16/2006
B-736	75.0	23.3	214681.67	961154.26	98.29	10/16/2006
B-737	100.0	-36.5	214511.91	961147.40	63.47	10/16/2006
B-738	75.0	12.3	213826.30	961679.62	87.29	10/19/2006
B-739	99.8	0.5	213719.60	961793.32	100.35	10/19/2006
B-740	75.0	-0.7	213605.13	961781.13	74.29	10/19/2006
B-741	75.0	6.4	213760.48	961029.82	81.38	10/18/2006
B-742	100.0	2.4	213472.84	961217.19	102.39	10/18/2006
B-743	75.0	28.6	213315.70	961232.00	103.60	5/9/2006
B-744	100.0	13.3	216377.30	959963.38	113.28	9/29/2006
B-745	75.0	36.7	215971.20	960529.02	111.71	9/29/2006
B-746	75.0	7.8	215743.35	960721.36	82.79	9/29/2006
B-747	75.0	15.3	216176.28	959944.95	90.34	9/29/2006
B-748	100.0	-17.6	216039.74	960288.74	82.40	9/29/2006
B-749	75.0	27.5	215775.08	960332.24	102.53	9/29/2006
B-750	73.9	-1.6	215849.16	959930.06	72.35	9/29/2006
B-751	73.9	18.3	215588.86	960146.20	92.23	9/29/2006
B-752	100.0	-4.2	215489.21	960257.57	95.79	9/29/2006
B-753	40.0	8.8	217831.20	960648.86	48.81	9/21/2006
B-754	50.0	17.0	217369.78	960290.37	67.00	9/21/2006
B-755	40.0	55.0	215923.66	961637.86	94.98	9/22/2006
B-756	50.0	56.9	215504.60	961215.10	106.85	4/21/2006
B-757	40.0	66.9	215135.13	960760.60	106.86	10/16/2006
B-758	40.0	42.6	215133.29	960332.67	82.63	10/16/2006
B-759	100.0	-1.7	214526.25	960025.32	98.35	10/19/2006
B-765	102.0	-4.6	216424.51	959701.22	97.37	9/29/2006
B-766	50.0	58.9	216932.89	959791.50	108.89	9/19/2006
B-768	100.0	-51.6	217116.03	962242.98	48.39	9/28/2006
B-769	50.0	4.2	216589.75	962559.47	54.23	9/28/2006
B-770	50.0	71.6	215466.60	962826.95	121.59	10/18/2006

Note:

*Location and elevation approximated based on offset observed in the field and recorded on Field Checklist.

Table 2.5.4-18 Summary Undisturbed Tube Sample
(Page 1 of 6)

Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
B-301	U. TRUCK	5/25/2006	UD-1	33.5 - 35.5	24	MH
			UD-2	43.5 - 45.3	21	MH
			UD-3	88.5 - 90.5	0	
			UD-4	98.5 - 99.8	6	SM
			UD-5	138.5 - 140.5	4	SC / SM
		5/30/2006	UD-6	158.5 - 159.6	13	13" push, CL with fine sand
			UD-7	168.5 - 170.5	9	CL / MH
			UD-8	183.5 - 184.3	10	MH
B-302	C. ATV	5/30/2006	UD-1	83.5 - 84.9	16	16" push, SM with fine sand, shell
			UD-2	128.5 - 130.5	12	MH
B-303	U. TRUCK	5/9/2006	UD-1	28 - 30	24	CL
				38 - 39.6	19	19" push, SC
B-304	U. ATV	5/30/2006	UD-1	73.5 - 75.5	22	SM
			UD-2	98.5 - 99.5	12	12" push, SC
			UD-3	138.5 - 139.3	10	MH
B-305	C.ATV	7/17/2006	UD-1	12.5 - 14.3	22	CH
			UD-2	19.5 - 21.2	16	MH
			P-3	35 - 37	5	pitcher, cemented sand
			P-4	39.5 - 41.5	22	pitcher, SM
			UD-5	52.5 - 53.5	7	f. sandy silt, shell
			P-6	89.5 - 91.5	8	pitcher, sand
B-306	U. TRUCK	5/5/2006	UD-1	58 - 60	24	CL
		5/5/2006	UD-2	68 - 70	24	CL
B-307	U. TRUCK	5/15/2006	UD-1	123.5 - 124.7	14	SM
			UD-2	178.5 - 180.4	23	MH
B-308	U. TRUCK	5/3/2006	UD-1	43 - 45	24	CL
		5/4/2006	UD-2	53 - 55	16	CL
		5/4/2006	UD-3	63 - 65	0	sand
B-309	C. TRUCK	5/11/2006	UD-1	33.5 - 35.5	23	CL
		5/11/2006	UD-2	43.5 - 45.5	24	CL
		5/11/2006	UD-3	53.5 - 55.5	23	SC
B-310	C. ATV	6/15/2006	UD-1	78.5 - 79.8	15	SC
B-312	C. ATV	5/18/2006	UD-1	10.5 - 12.3	17	21" push, CH
		5/18/2006	UD-2	38.5 - 38.6	0	0.5" push
		5/18/2006	UD-3	98.5 - 99.5	12	12" push, MH
B-313	U. ATV	5/22/2006	UD-1	93.5 - 94.7		CL
			UD-2	123.5 - 124.3		ML

Table 2.5.4-18 Summary Undisturbed Tube Sample
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Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
B-314			UD-1	13.5 - 15.5	12	CH
B-315	C. ATV	5/22/2006	UD-1	23.5 - 25.5	14	CH
B-316	C. TRUCK	5/4/2006	UD-1	43.5 - 45.5	24	CL
		5/4/2006	UD-2	53.5 - 55.5	24	CL
B-317	C. TRUCK	5/5/2006	UD-1	28.5 - 30.5	24	CL
		5/5/2006	UD-2	38.5 - 40.5	24	CH
		5/5/2006	UD-3	48.5 - 50.3	21	SC
B-318	U. ATV	6/3/2006	UD-1	148.5 - 149.1	3	7" push, f. sandy SILT
B-319	U. ATV	5/5/2006	UD-1	33.5 - 35.5	24	MH
		5/5/2006	UD-2	43.5 - 45.5	27	MH
		5/5/2006	UD-3	53.5 - 54.3	10	MH
B-320	C. TRUCK	5/8/2006	UD-1	38.5 - 40.5	24	MH
		5/9/2006	UD-2	48.5 -50	18	18" push, clayey sand
B-321	C. ATV	6/5/2006	UD-1	23.5 - 25	18	CH
		6/6/2006	UD-2	73.5 - 75.5	24	SM
B-322	U. ATV	5/18/2006	UD-1	28.5 - 30.5	28	CL
			UD-2	38.5 - 39.9	27	SM
			UD-3	48.5 - 49.3	9	SC
B-323	U. ATV	6/7/2006	UD-1	83.5 - 84.8	15	MH
			UD-2	178.5 - 179.1	0	MH
B-324			UD-1	60 - 62	24	CH
			P-2	69 - 71	22	SM
			P-3	85.5 - 87.5	5	SM
B-326	U. ATV	5/4/2006	UD-1	33.5 - 35.5	28	CL
		5/4/2006	UD-2	43.5 - 45.5	28	MH
		5/4/2006	UD-3	53.5 - 55.5	27	bottom 2" bent, sandy lean clay
B-327	C. ATV	5/25/2006	UD-1	113.5 - 114.2	9	ML
			UD-2	138.5 - 140.5	10	SM
B-328	C.ATV	6/19/2006	UD-1	63.5 - 65.5	24	SM
			UD-2	93.5 - 94.6	12	SC
			UD-3	123.5 - 124.4	11	ML, shell
B-329	C.ATV	6/13/2006	UD-1	63.5 - 65.3	22	SM
			UD-2	73.5 - 75.5	24	SM
B-330	U. ATV	5/25/2006	UD-1	28.5 - 29.2	0	
B-331	C. ATV	5/24/2006	UD-1	18.5 - 20.5	24	MH
B-332	C. ATV	6/2/2006	UD-1	73.5 - 74.6	13	SM
B-333	U. ATV	5/17/2006	UD-1	28.5 - 30.5	24	MH
			UD-2	38.5 - 40.5	24	CL
			UD-3	48.5 - 48.8	4	SM
B-334	U. TRUCK	5/24/2006	UD-1	23 - 25	24	CL

Table 2.5.4-18 Summary Undisturbed Tube Sample
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Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
			UD-2	33 - 35	13	CL
B-335	U. ATV	5/3/2006	UD-1	31 - 33	24	CL
			UD-2	38.5 - 40.5	24	CH
			UD-3	48.5 - 50.5	24	CL
			UD-4	58.5 - 58.8	3	tube deformed, SPT @ bottom, sand with shell
B-336	U. ATV	5/15/2006	UD-1	33.5 - 35.5	24	CH
			UD-2	43.5 - 45.5	24	CH
			UD-3	53.5 - 55.5	15	SC
B-337	C. ATV	6/7/2006	UD-1	53.5 - 54.6	13	ML
B-338	C.ATV	6/13/2006	UD-1	48.5 - 50.5	24	MH / ML
				94.5 - 95.0	?	not on boring log
				95 - 97	?	not on boring log
			UD-4	98.5 - 99.6	7	SM
B-340	C.TRACK	8/4/2006	P-1	66 - 68	12	SC, cemented
B-341			UD-1	88.5 - 90.5	24	SM
			UD-2	98.5 - 100.5	24	SP-SM
B-401	U.TRUCK	6/20/2006	UD-1	68.5 - 70.5	23	SM
			UD-2	98.5 - 99.8	15	ML
			UD-3	123.5 - 124.8	16	CL
			UD-4	138.5 - 140.5	23	MH
		6/21/2006	UD-5	158.5 -159.3	10	MH
		6/21/2006	UD-6	173.5 - 174.4	11	MH
		6/22/2006	UD-7	198.5 - 200.5	21	ML
		6/22/2006	UD-8	213.5 - 214.6	13	ML
			UD-9	228.5 - 229.6	13	ML
			UD-10	243.5 - 244.4	8	ML
			UD-11	348.5 - 350.5	7	
B-403	C.ATV	6/21/2006	UD-1	63.5 - 64.9	20	SM
			UD-2	98.5 - 99.5	12	ML
			UD-3	123.5 - 124.5	12	ML
B-404	U.ATV	6/23/2006	UD-1	52 - 53.6	18	SP-SM
			UD-2	66 - 67.5	18	SC
			UD-3	83.5 - 85.1	17	SC
B-405	C. TRUCK	5/16/2006	UD-1	58.5 - 60.5	22	CL
			UD-2	68.5 - 70.5	24	CL
B-406	U. TRUCK	5/17/2006	UD-1	63.5 - 65.5	24	CH
			UD-2	73.5 - 75.2	12	21" push, SC

Table 2.5.4-18 Summary Undisturbed Tube Sample
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Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
B-407	U. ATV	5/14/2006	UD-1	53.5 - 54.5	11	12" push, SM with shell
		5/15/2006	UD-2	78.5 - 79	4	tube bent, SM
		5/15/2006	UD-3	128.5 - 129	6	ML with sand
		5/15/2006	UD-4	153.5 - 153.9	5	tube bent, MH
B-409	C.TRUCK	6/22/2006	P-1	35	13	Pitcher, SP
			UD-2	17.5 - 19	24	SC
			UD-3	50 - 52	24	SM
			UD-4	62.5 - 64.5	24	SM
			UD-5	95 -96.6	19	ML, sandy SILT
		6/27/2006	UD-6	137.5 - 139	18	MH
B-410	C. TRUCK	5/1/2006	UD-1	53.5 - 55.5	0	shelby tube lost in hole, not accepted
		5/1/2006	UD-2	60.5 - 62.5	15.5	remnant tube recovered, not accepted
B-410A	C. TRUCK	5/1/2006		53.5 - 55.5	24	CH, not on log
		5/1/2006	UD-2	63.5 - 65.5	7	CH
		5/2/2006	UD-3	73.5 - 75	18	CH, f. sand at bottom
B-411	C.ATV	7/26/2006	UD-1	23 - 25	16	CH
B-413	U. TRUCK	5/15/2006	UD-1	73 - 75	24	CL
B-414	U. TRUCK	5/11/2006	UD-1	58 - 60	24	CL
		5/11/2006	UD-2	68 - 70	24	CL
B-418	U.ATV	6/28/2006	UD-1	?	0	
B-420	U. TRUCK	6/6/2006	UD-1	63.5 - 65.5	24	SM
		6/7/2006	UD-2	128.5 - 130.3	22	CL
B-421	C. TRUCK	5/10/2006	UD-1	48.5 - 50.5	24	ML
		5/10/2006	UD-2	58.5 - 60.5	24	CL
B-422	C. ATV	5/4/2006	UD-1	38.5 - 40.5	24	CL
		5/4/2006	UD-2	48.5 - 50.5	23	CH
		5/4/2006	UD-3	58.5 - 59.3	8	CH / SC
B-423			UD-1	103.5 - 105.3	21	SM
			UD-	113.5 - 113.8	0	
			UD-2	158.5 - 160.1	19	CL
			UD-3	178.5 - 179.8	16	MH
			UD-4	188.5 - 189.2	8	MH
B-425	U. TRUCK	5/1/2006	UD-1	57 - 59	24	CH
		5/1/2006	UD-2	65 - 67	24	CH
		5/1/2006	UD-3	75 - 77	24	CH
B-427	C. TRUCK	5/2/2006	UD-1	63.5 - 65.5	24	CH
		5/2/2006	UD-2	73.5 - 74.8	15	SC

Table 2.5.4-18 Summary Undisturbed Tube Sample
(Page 5 of 6)

Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
B-428	U. TRUCK	5/2/2006	UD-1	57 - 59	21	CH, bottom 10" bent
		5/2/2006	UD-2	60 - 62	24	CL, bent
		5/2/2006	UD-3	63 - 65	20	CL, bottom 10" bent
		5/2/2006	UD-4	66 - 68	24	CL, bottom 5" bent
		5/2/2006	UD-5	69 - 71	7	CL, bottom 3" bent
B-429	U. ATV	5/1/2006	UD-1	45 - 47	24	CH
		5/1/2006	UD-2	53.5 - 55.5	0	
		5/1/2006	UD-3	58.5 - 60	18	SC
B-430	C. ATV	5/1/2006	UD-1	30 - 32	10	ML
		5/1/2006	UD-2	38.5 - 39.2	5	SC
		5/1/2006	UD-3	48.5 - 50.1	18	MH
		5/1/2006	UD-4	58.5 - 59.3	18	ML
B-433	C. TRUCK	5/17/2006		28.5 - 30.5	24	not on log
		5/17/2006	UD-2	38.5 - 40.5	24	CL
		5/17/2006	UD-3	48.5 - 48.8	4	CL from log
B-434	C. ATV	5/9/2006	UD-1	43.5 - 45.5	6.5	CL
		5/9/2006	UD-2	53.5 - 55	18	CH
		5/10/2006	UD-3	63.5 - 64.3	14	CH
B-436	C. ATV	5/9/2006	UD-1	48.5 - 50.5	18	CL
B-437	U.TRUCK	7/10/2006	UD-1	13.5 - 15.5	23	SM
			UD-2	98.5 - 100.5	22	SM
B-438a			UD-1	93.5 - 95.5	14	SM
B-440	U. ATV	6/6/2006	UD-1	51 - 53	24	SM
			UD-2	58.5 - 58.6	0	
B-701	C.TRUCK	6/28/2006	UD-1	43.5 - 44.9	17	ML
B-703			UD-1	18.5 - 20.5	19	CH
			UD-2	73.5 - 75.5	10	SM
B-708	U. ATV	5/9/2006	UD-1	78.5 - 79.5	12	12" push, sand
B-714			UD-1	48 - 50	24	SC
B-722	U.ATV	7/18/2006	UD-1	13 - 15	24	SM
B-723	C.TRACK	6/1/2006	UD-1	28.5 - 30.2	20	SP-SC
			UD-2	38.5 - 40.5	24	CL
B-724	C. TRACK	6/5/2006	UD-1	73.5 - 75.5	21	SM
B-725	C. TRACK	6/6/2006	UD-1	63.5 - 65.5	24	SM
B-726	C.TRACK	8/1/2006	UD-1	10.5 - 12.5	0	No Recovery
		8/1/2006	UD-2	23.5 - 25.5	19.5	CH
B-727	C. ATV	5/10/2006	UD-1	48.5 - 50.5	22	
		5/11/2006	UD-2	63.5 - 65.5	20	24" push
B-728	C. ATV	5/11/2006	UD-1	53.5 - 55.5	23	CH
B-729	C. TRUCK	5/19/2006	UD-1	68.5 - 70.5	24	CH
B-730	C. TRUCK	5/18/2006	UD-1	53.5 - 55.5	0	No Recovery
			UD-2	68.5 - 70.5	24	CH
B-731	C. TRACK	5/31/2006	UD-1	58.5 - 60.5	24	SM
B-732	C.TRACK	6/8/2006	UD-1	15 - 17	24	SM
B-733	C. TRACK	6/8/2006	UD-1	23.5 - 25.5	24	CL
			UD-2	88.5 - 90.5		CH/MH
B-734	C. TRACK	6/7/2006	UD-1	48.5 - 50.5	24	CL

Table 2.5.4-18 Summary Undisturbed Tube Sample (Page 6 of 6)

Boring	Drill Rig	Date	Sample No.	Depth (ft)	Rec (in.)	Field Remarks
B-735	C.TRACK	6/28/2006	UD-1	28 - 30	24	sand
B-737	C.TRACK	7/19/2006	UD-1	10.5 - 12.5	24	SC / CL
B-739		6/15/2006	UD-1	51- 52	12	SC
			UD-2	83.5 - 84	5	CL
			UD-3	96 - 96.8	9	SP-SM
B-742			UD-1	´78.5 - 78.6	0	
			UD-2	88.5 - 88.8	3	SM, sample placed in jar
B-743	U.ATV	7/10/2006	UD-1	23.5 - 25.5	21	SM
			UD-2	38 - 40	0	
B-746	C. TRACK	7/18/2006	UD-1	13.5 -15.5	24	SM
B-748	C.TRACK	7/17/2006	UD-1	13.5 - 15.5	24	ML
B-749	C. TRUCK	5/23/2006	UD-1	43.5 - 45.5		
B-750	C.TRACK	7/10/2006	UD-1	28.5 - 30.5	0	
			UD-2	48.5 - 49.5	11	clayey sand, shells
B-751	C. TRUCK	5/22/2006	UD-1	33.5 - 35.5		
			UD-2	43.5 - 45.5		
B-752	C.TRACK	7/5/2006	UD-1	58 - 59.5	18	clay
B-759			UD-1	56.5 - 57	0	
			UD-2	66 - 68	24	CH
			UD-3	98 - 98.5	5	SC, tube bent
B-765	C. TRACK	7/12/2006	P-	70 - 72	8	cemented fine sandy silt, trace clay, trace shells
			P-	100 - 102	20	clayey fine sandy silt
B-768	C.TRUCK	6/20/2006	UD-1	43.5 - 45.3	20	SM
			UD-2	73.5 - 75.5	24	SM

Total Tubes Attempted: 217

Table 2.5.4-19 Summary As-Conducted CPT Information (Page 1 of 2)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927)		Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey	Remarks		
			North	East			Pre-Drill	Seismic	Dissipation
C-301	52.3	42.5	217041.78	960820.13	94.84	9/15/2006		√	
C-302	61.7	29.3	217088.90	960833.77	90.94	9/15/2006			√
C-302-2*	55.3	39.2	217026.56	960817.55	94.51	7/26/2006			
C-302-2a*	138.0	-43.5	217026.56	960817.55	94.51	7/26/2006	√ 85 ft		√
C-303	25.4	36.2	217230.60	960804.00	61.58	4/24/2006			
C-303a*	47.1	14.5	217230.60	960804.00	61.58	7/25/2006	√ 45 ft		
C-303a-1*	71.4	-9.8	217230.60	960804.00	61.58	7/25/2006	√ 50 ft		
C-303b*	123.4	-61.8	217230.60	960804.00	61.58	7/25/2006	√ 80 ft		√
C-304	26.7	34.2	217235.29	960606.73	60.95	9/15/2006		√	√
C-305	74.3	41.6	216876.50	960961.50	115.91	4/24/2006			
C-306	56.9	40.4	217042.12	961184.89	97.31	9/15/2006			√
C-306a*	102.5	-5.2	217038.92	961181.69	97.31	7/27/2006	√ 80 ft		
C-307	75.3	42.4	216853.68	961079.64	117.64	9/15/2006		√	
C-308	48.2	36.1	217129.90	960263.70	84.33	5/1/2006		√	
C-309	70.1	36.0	217045.62	960110.76	106.04	9/15/2006			√
C-311	34.9	39.0	216869.75	960488.16	73.97	9/15/2006			
C-312	56.4	43.3	216799.20	960596.36	99.75	9/15/2006			
C-313	37.2	42.7	216757.92	960336.75	79.93	9/15/2006			
C-314	39.5	40.6	216531.40	960493.83	80.09	9/15/2006			
C-401	28.1	39.4	216384.26	961574.09	67.46	9/15/2006		√	
C-401-2a*	81.9	-14.4	216381.06	961570.89	67.46	7/27/2006	√ 55 ft	√	
C-401-2b*	131.2	-63.8	216381.06	961570.89	67.46	7/27/2006	√ 85 ft	√	√
C-402	34.5	38.7	216333.85	961494.18	73.13	9/15/2006			√
C-403	43.8	39.2	216517.33	961511.47	82.96	9/15/2006			
C-404	80.1	39.2	216524.30	961308.90	119.21	4/20/2006		√	√
C-405	40.0	35.5	216163.49	961666.32	75.54	9/15/2006			
C-406	15.6	28.3	216380.92	961901.51	43.89	9/28/2006			√
C-407	32.3	30.9	216159.20	961732.20	63.23	6/22/2006		√	√
C-407-2a*	96.3	-33.1	216161.50	961726.70	63.23	7/28/2006	√ 50 ft	√	√
C-407-b*	142.4	-79.2	216161.50	961726.70	63.23	7/31/2006	√ 95 ft	√	√
C-408	77.4	40.8	216396.64	961001.81	118.18	9/15/2006		√	
C-408a*	98.3	19.9	216398.76	960999.69	118.18	7/24/2006	√ 98 ft		
C-408-2a*	123.7	-5.5	216393.81	961004.64	118.18	7/31/2006	√ 105 ft	√	
C-409	80.5	38.6	216288.45	960760.56	119.12	9/15/2006			√
C-411	80.4	36.2	216178.94	961178.21	116.60	9/19/2006			√
C-412	76.8	37.5	216093.75	961306.66	114.31	9/28/2006			
C-413	13.6	86.3	216045.53	961037.78	99.90	9/28/2006			
C-414	62.5	39.9	215893.42	961201.10	102.36	9/28/2006			√
C-415	20.0	36.6	216305.70	961857.40	56.63	5/26/2006			
C-701	29.5	-18.6	219262.19	960933.61	10.95	9/21/2006			√
C-701a*	28.1	-17.1	219265.39	960936.81	10.95	7/21/2006			
C-702	20.3	-9.0	218720.05	961033.95	11.34	9/21/2006			
C-703	32.6	35.2	217361.27	961165.03	67.82	10/17/2006			√
C-704	48.2	-2.9	217500.74	961710.02	45.36	9/28/2006			
C-705	34.0	-2.9	217637.26	961983.10	31.08	9/28/2006			

Table 2.5.4-19 Summary As-Conducted CPT Information (Page 2 of 2)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927)		Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey	Remarks		
			North	East			Pre-Drill	Seismic	Dissipation
C-706	50.0	55.2	216958.95	961494.86	105.28	9/21/2006			
C-707	19.5	20.8	216308.12	962079.42	40.35	9/22/2006			
C-708	50.0	62.9	215658.28	961962.86	112.97	10/16/2006			
C-709	50.0	61.7	215027.59	962824.89	111.73	10/18/2006			
C-710	21.2	85.0	214875.83	961187.31	106.15	10/16/2006			
C-711	34.9	65.6	214222.13	962176.75	100.54	10/17/2006			
C-712	29.7	29.4	213909.83	961370.06	59.05	10/18/2006			√
C-713	41.8	21.3	215855.86	962296.57	63.11	9/28/2006			
C-714	85.1	24.2	214920.30	963057.62	109.32	10/18/2006			√
C-715	57.3	33.6	215445.62	961798.99	90.85	10/16/2006			
C-716	20.5	75.7	214432.49	962659.44	96.21	10.17/2006			
C-717	66.6	35.8	214698.14	961692.58	102.35	10/16/2006			√
C-718	34.1	33.6	214343.71	961205.59	67.67	10/16/2006			
C-719	12.0	78.2	214025.30	961636.90	90.21	10/18/2006			
C-720	70.7	28.0	213593.77	961134.09	98.66	10/18/2006			√
C-721	52.0	35.6	216157.88	960330.47	87.62	9/29/2006			
C-722	38.4	36.1	215478.76	960648.26	74.52	10/16/2006			
C-723	68.7	28.9	215988.18	959760.36	97.60	9/29/2006			√

* Location and elevation approximated based on offset observed in the field and recorded on Field Checklist

Table 2.5.4-20 Summary As-Conducted Observation Well Information (Page 1 of 2)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Elevation (ft), Top of Concrete at Base of Well Head Protector	Elevation (ft), Ground Water Level Measuring Point (V-Notch)	Date of As Built Survey
OW-301	80.0	14.5	217048.02	960814.47	94.51	94.78	96.27	9/15/2006
OW-313A	57.5	-6.5	217367.31	960705.30	51.03	51.31	53.20	9/15/2006
OW-313B	110.0	-59.3	217372.34	960713.67	50.73	51.16	53.54	9/15/2006
OW-319A	35.0	68.1	216962.56	961116.12	103.13	103.31	104.91	9/15/2006
OW-319B	85.0	18.5	216957.32	961125.02	103.53	103.85	105.35	9/19/2006
OW-323	43.5	63.5	217034.46	960057.07	106.96	107.55	109.69	9/19/2006
OW-328	72.0	4.3	216828.86	960493.21	76.29	76.55	77.85	9/19/2006
OW-336	74.0	23.1	216643.18	960746.61	97.11	97.50	99.07	9/16/2006
OW-401	77.5	-6.1	216348.86	961530.99	71.38	71.91	73.49	9/21/2006
OW-413A	50.0	73.2	216703.14	961418.81	123.15	123.51	125.04	9/15/2006
OW-413B	125.0	-2.1	216694.88	961413.25	122.90	123.25	124.85	9/15/2006
OW-418A	40.0	3.7	216340.41	961966.46	43.66	44.31	45.83	9/22/2006
OW-418B	92.0	-48.3	216340.25	961976.71	43.67	44.13	45.77	9/22/2006
OW-423	43.0	68.1	216339.99	960882.24	111.12	111.67	113.16	9/15/2006
OW-428	50.0	63.9	216105.21	961212.38	113.92	114.32	115.92	9/19/2006
OW-436	50.0	58.1	215922.47	961446.87	108.13	108.53	110.39	9/22/2006
OW-703A	49.0	-5.0	218171.23	960967.72	44.02	44.44	45.65	9/21/2006
OW-703B	80.0	-34.4	218171.67	960958.91	45.57	45.97	47.53	9/21/2006
OW-705	52.0	-4.3	217566.62	960917.18	47.71	47.77	50.22	9/15/2006
OW-708A	34.0	3.4	217586.23	961803.52	37.44	37.82	39.61	9/28/2006
OW-711	50.0	2.9	216748.48	961741.61	52.92	53.26	55.31	9/22/2006
OW-714	50.0	66.0	215705.73	962034.37	116.02	116.32	117.98	10/16/2006
OW-718	43.0	75.5	214133.58	961924.87	118.53	118.96	120.41	10/18/2006
OW-725	60.0	-2.0	214649.30	963212.73	58.04	58.38	59.94	10/18/2006
OW-729	42.0	76.9	214872.58	962445.93	118.88	119.44	121.11	10/17/2006
OW-735	72.0	19.2	214805.48	961021.83	91.20	91.81	93.44	10/16/2006
OW-743	55.0	48.7	213320.62	961234.01	103.65	104.05	105.89	10/18/2006
OW-744	50.0	47.5	216405.37	960089.41	97.50	97.96	99.81	9/29/2006

Table 2.5.4-20 Summary As-Conducted Observation Well Information (Page 2 of 2)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	Coordinates (ft), Maryland State Plane (NAD 1927) East	Ground Surface Elevation (ft) (NGVD 1929)	Elevation (ft), Top of Concrete at Base of Well Head Protector	Elevation (ft), Ground Water Level Measuring Point (V-Notch)	Date of As Built Survey
OW-752A	37.0	58.3	215482.18	960250.12	95.30	95.73	97.00	9/29/2006
OW-752B	97.0	-1.2	215489.21	960257.57	95.79	96.09	97.41	9/29/2006
OW-754	44.0	23.0	217369.78	960290.37	67.00	67.21	68.85	9/15/2006
OW-756	42.0	64.6	215497.07	961212.39	106.56	107.07	108.77	10/16/2006
OW-759A	35.0	62.8	214536.47	960055.02	97.78	98.05	99.69	10/19/2006
OW-759B	90.0	8.3	214526.25	960056.32	98.35	98.72	100.14	10/19/2006
OW-765A	29.0	68.4	216424.51	959701.22	97.37	97.92	99.60	9/29/2006
OW-765B	102.0	-5.2	216420.42	959693.64	96.82	97.19	98.47	9/29/2006
OW-766	50.0	58.9	216932.89	959791.50	108.89	109.32	110.72	9/19/2006
OW-768A	42.0	6.5	217106.06	962238.98	48.48	48.96	49.84	9/28/2006
OW-769	42.0	12.2	216589.75	962559.47	54.23	54.39	56.43	9/28/2006
OW-770	42.0	79.6	215466.60	962826.95	121.59	121.79	123.08	10/18/2006

Table 2.5.4-21 In-Situ Hydraulic Conductivity (Slug) Test Results (Page 1 of 1)

Location	Screened Interval Depth (ft)	USCS Soil Classification	Hydraulic Conductivity (feet/sec)
OW-301	65 – 75	SP	$1.58X10^{-4}$
OW-313A	40 – 50	SM, ML	$7.50X10^{-6}$
OW-313B	95 – 105	CL, ML, MH	$2.74X10^{-7}$
OW-319A	20 – 30	SP-SM, SC, CH, CL	$2.89X10^{-6}$
OW-319B	70 – 80	SM	$3.42X10^{-5}$
OW-323	30 – 40	SP, SP-SM	$6.24X10^{-5}$
OW-328	60 – 70	SM, OH	$3.79X10^{-6}$
OW-336	60 – 70	SP-SM, SM	$2.10X10^{-5}$
OW-401	63 – 73	SM	$6.77X10^{-6}$
OW-413A	35 – 45	SP-SM	$1.21X10^{-5}$
OW-413B	110 – 120	SP-SM, SM	$2.78X10^{-6}$
OW-418A	25 – 35	SP-SM	$4.41X10^{-6}$
OW-418B	75 – 85	SC, SM	$2.16X10^{-7}$
OW-423	28 – 38	SP-SM, SM, SC	$6.86X10^{-5}$
OW-428	35 – 45	SM, SC	$1.19X10^{-5}$
OW-436	29 – 39	SC, SM	$2.80X10^{-6}$
OW-703A	35 – 45	SM	$1.34X10^{-5}$
OW-703B	68 – 78	SM, ML	$1.08X10^{-6}$
OW-705	40 – 50	SC, SM	$4.99X10^{-6}$
OW-708	22 – 32	SM	$2.56X10^{-5}$
OW-711	35 – 45	SM	$6.04X10^{-6}$
OW-714	38 – 48	SP-SM, SC	$2.81X10^{-6}$
OW-718	30 – 40	SP-SM	$4.44X10^{-6}$
OW-725	48 – 58	SM	$7.54X10^{-6}$
OW-735	60 – 70	SP-SM, SM	$5.48X10^{-5}$
OW-743	40 – 50	SP-SM, SM	$6.23X10^{-7}$
OW-744	38 – 48	CL, SC, SM	$1.07X10^{-6}$
OW-752A	25 – 35	CH, SM	$7.03X10^{-5}$
OW-752B	85 – 95	SP-SM	$3.35X10^{-6}$
OW-754	32 – 42	CL, SM	$5.29X10^{-6}$
OW-756	30 – 40	SP-SM, SP-SC	$2.01X10^{-4}$
OW-759A	20 – 30	SM, SC, MH	$4.64X10^{-7}$
OW-759B	75 – 85	SM, SP, SP-SM	$1.17X10^{-6}$
OW-765A	17 – 27	SP-SM	$1.00X10^{-5}$
OW-765B	82 – 92	SM	$1.36X10^{-6}$
OW-766	20 – 30	SP-SM	$1.10X10^{-6}$
OW-768	30 – 40	SM	$5.29X10^{-6}$
OW-769	32 – 42	SM, SC	$1.74X10^{-6}$

Table 2.5.4-22 Summary As-Conducted Test Pit Information
(Page 1 of 1)

Location	Depth (ft)	Termination [bottom] Elevation (ft)	Coordinates (ft), Maryland State Plane (NAD 1927) North	East	Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
TP-B307	6.7	112.7	216957.53	960690.62	119.35	9/19/2006
TP-B314	9.0	43.8	217320.35	960658.25	52.78	9/15/2006
TP-B315	8.5	57.3	217182.50	960563.12	65.80	9/15/2006
TP-B334	10.0	77.0	216515.64	960560.94	87.03	9/19/2006
TP-B335	8.0	91.6	216730.79	960706.97	99.64	9/19/2006
TP-B407	7.0	74.3	216391.76	961465.02	81.25	9/21/2006
TP-B414	6.5	114.3	216631.18	961530.95	120.83	9/15/2006
TP-B415	6.5	112.4	216490.91	961298.37	118.92	9/15/2006
TP-B423	8.0	97.9	216414.95	960849.03	105.86	9/19/2006
TP-B434	8.5	96.7	215825.90	961244.18	105.24	9/22/2006
TP-B435	10.0	97.7	216020.06	961404.74	107.71	9/19/2006
TP-B715	8.5	79.7	214964.18	962637.77	88.16	10/17/2006
TP-B716	8.8	88.3	214983.83	961289.79	97.13	10/16/2006
TP-B717	8.0	82.5	214297.68	962346.36	90.53	10/17/2006
TP-B719	8.0	64.3	213966.93	961493.94	72.28	10/18/2006
TP-B727	7.0	97.3	215299.14	961883.13	104.33	10/16/2006
TP-B744	6.5	106.8	316377.30	959963.38	113.28	9/29/2006
TP-B758	9.0	73.6	215133.29	960332.67	82.63	10/16/2006
TP-C309	8.0	100.5	217020.05	960105.24	108.45	9/19/2006
TP-C723	7.0	89.8	215989.07	959754.78	96.75	9/29/2006

Table 2.5.4-23 Summary Field Electrical Resistivity Information (Page 1 of 1)

Location	Coordinates (ft), Maryland State Plane (NAD 1927)		Ground Surface Elevation (ft) (NGVD 1929)	Date of As Built Survey
	North	East		
R-1	215837.30	960255.80	85.45	5/3/2006
R-2	215837.30	960255.80	85.45	5/3/2006
R-3	216622.50	960406.80	89.12	5/2/2006
R-4	215915.40	961114.00	99.40	4/27/2006

Table 2.5.4-24 Geophysical Data from CCNPP Units 1 and 2 UFSAR (Page 1 of 1)

	SURFICIAL SEDIMENTS (PLEISTOCENE) COMPRESSIONAL		UNCONSOLIDATED SEDIMENTS (TERTIARY) COMPRESSIONAL		INTERMEDIATE SEDIMENTS (CRETACEOUS)[a] COMPRESSIONAL		BASEMENT ROCK COMPRESSIONAL	
STATION	WAVE VELOCITY (fps)	THICKNESS (ft)	WAVE VELOCITY (fps)	THICKNESS (ft)	WAVE VELOCITY (fps)	THICKNESS (ft)	WAVE VELOCITY (fps)	THICKNESS (ft)
Solomons Shoal[b]	–	–	5900	3080	–	–	15,170	3130
Solomons Deed[b]	–	–	6080	1070	6980	1900	18,100	3080
Site[c]	2200	40	5500	–	–	–	–	–
Site[c]	–	–	5900	–	–	–	–	–

Notes:

(a) These measurements refer to a "masked" arrival and the results are questionable.

(b) Adapted from Ewing and Worzel (Reference 3).

(c) Measurements by Dames & Moore.

Table 2.5.4-25 Shockscope Data from CCNPP Units 1 and 2 UFSAR
(Page 1 of 1)

BORING	DEPTH (ft)	CONFINING PRESSURE (lbs/ft^2)	COMPRESSIONAL WAVE VELOCITY (fps)
DM-2	5	0 2000 4000 6000	1,000 1,200 1,400 1,700
DM-9	15	0 2000 4000 6000	1,200 1,300 1,500 1,700
DM-1	30	0 2000 4000 6000	1,400 1,500 1,800 2,100
DM-10	68	0 2000 4000 6000	2,600 2,600 3,200 3,200
DM-10	111	0 2000 4000 6000	2,600 2,600 3,000 3,000
DM-10	156	0 2000 4000 6000	1,800 1,800 1,900 1,900
DM-10	211	0 2000 4000 6000	1,600 1,700 1,700 1,700
DM-10	256	0 2000 4000 6000	2,100 2,100 2,200 2,200
DM-10	271	0 2000 4000 6000	2,000 2,200 2,300 2,600

Table 2.5.4-26 Summary Laboratory Test Results on Bulk Soil Samples (Page 1 of 1)

Non-Plastic Soils

Location	Depth (ft)	USCS	WC (%)	% Material Passing		Mod. Proctor Compaction		CBR	
				#4	#200	Max dry density (pcf)	Opt. WC (%)	Unsoaked	Soaked
TP-B307	4.5	SP-SM	2.3	100	5.8	109.3	10.5	14.8	4.4
TP-B315	6.0	SP-SM	5.4	99.8	9.7	114.9	11.4	11.6	18.9
TP-B334	3.0	SM	7.4	100	13.9	116.3	9.3		
TP-B334	6.0	SM	14.5	100	13.2	129.8	8.0		
TP-B335	5.0	SM	8.9	100	24.6	130.5	7.6	36.2	18.0
TP-B407	4.5	SW-SM	7.1	97.8	9.0	118.9	8.8	14.8	17.0
TP-B414	6.0	SP-SM	6.0	100	6.4	105.4	11.9		
TP-B415	3.0	SP	10.2	99.8	3.5	116.7	9.8	11.1	4.7
TP-B435	5.0	SM	6.0	100	13.2	119.1	8.9		
TP-B435	7.0	SP-SM	4.6	99.2	8.3	123.9	8.9	26.8	33.7
TP-B715	5.5	SP-SM	4.8	99.1	11.0	110.7	11.8		
TP-B716	6.0	SP-SM	3.8	99.0	6.0	116.3	9.4		
TP-B717	7.0	SP-SM	3.4	97.4	6.4	123.8	10.2	17.2	23.1
TP-B719	7.0	SM	26.7	100	44.3	119.6	10.0	41.3	29.0
TP-B727	6.0	SM	10.3	100	30.1	130.5	6.8		
TP-B758	2.0	SP-SM	6.0	99.2	8.4	121.0	8.8		
TP-B758	7.5	SM	11.8	97.4	31.1	127.3	8.9	11.3	4.4
TP-C309	2.0	SP	4.3	98.8	3.7	111.2	13.9		
TP-C309	7.0	SP-SM	8.7	100	7.8	112.3	9.8		
TP-C723	6.0	SP-SM	4.6	98.8	7.5	113.8	6.8		
Min.	2		2.3	97.4	3.5	105.4	6.8	11.1	4.4
Max.	7.5		26.7	100	44.3	130.5	13.9	41.3	33.7
Average:	**5**		**8**	**99**	**13**	**119**	**10**	**21**	**17**

Plastic Soils

Location	Depth (ft)	USCS	WC (%)	LL	PL	PI	% Material Passing		Mod. Proctor Compaction		CBR	
							#4	#200	Max dry density (pcf)	Opt. WC (%)	Unsoaked	Soaked
TP-B314	4.0	CH	37.0	71	24	47	100	93.1	114.6	15.5		
TP-B335	3.0	CL	19.0	30	20	10	100	65.3	128.8	9.9		
TP-B423	5.0	CL	16.0	24	16	8	100	51.1	123.4	10.8		
TP-B434	2.0	CL	21.0	25	18	7	99.8	59.8	127.1	10.1	9.3	3.2
TP-B435	9.0	SC	6.7	34	17	17	100	14.1	130.2	7.3	34.4	41.8
TP-B719	0.5	CL	23.9	35	22	13	100	84.5	118.4	13.5		
TP-B744	1.5	CL	18.0	25	17	8	100	64.2	131.2	8.0		
TP-C723	2.5	SC	12.0	30	15	15	100	39.5	132.8	7.3	26.8	17.2
Min.	0.5		6.7	24	15	7	99.8	14.1	114.6	7.3	9.3	3.2
Max.	9		37	71	24	47	100	93.1	132.8	15.5	34.4	41.8
Average:	**3**		**19**	**34**	**19**	**16**	**100**	**59**	**126**	**10**	**24**	**21**

Table 2.5.4-27 Design Vs Profile for CCNPP Unit 3 Subsurface Seismic Evaluation
(Page 1 of 1)

Unit	Soil	Depth Range (ft)		El Range (ft)		Vs (feet/sec)
I	Terrace SAND	0	25	+85	+60	790
II-a	Chesapeake CLAY/SILT	25	40	+60	+45	1,100
Ii-b-1	Chesapeake Cemented SAND	40	55	+45	+30	1,450
II-b-2	Chesapeake Cemented SAND	55	70	+30	+15	1,800
II-b-3	Chesapeake Cemented SAND	70	85	+15	0	1,130
II-b-4	Chesapeake Cemented SAND	85	100	0	-15	1,740
II-c	Chesapeake CLAY/SILT	100	285	-15	-200	1,250
III-a-1	Nanjemoy Cemented CLAY/SILT	285	305	-200	-220	1,790
III-a-2	Nanjemoy Cemented CLAY/SILT	305	315	-220	-230	2,330
III-a-3	Nanjemoy Cemented CLAY/SILT	315	355	-230	-270	2,030
III-b-1	Nanjemoy SAND	355	402	-270	-317	1,930
III-b-2	Nanjemoy SAND	402	456	-317	-371	2,200
IV	Marboro CLAY	456	471	-371	-386	2,200
V	Aquia-Brighseat SAND	471	631	-386	-546	2,200
VI-1	Patapsco SAND	631	1,085	-546	-1,000	2,200
VI-2	Patapsco SAND	1,085	1,585	-1,000	-1,500	2,330
VI-3	Patapsco SAND	1,585	1,731	-1,500	-1,646	2,500
VII-1	Patuxent/Arundel CLAY	1,731	2,085	-1,646	-2,000	2,550
VII-2	Patuxent/Arundel CLAY	2,085	2,531	-2,000	-2,446	2,800
VIII-1	Granitoid Bedrock	2,531	2,531	-2,446	-2,446	5,000
VIII-2	Granitoid Bedrock	2,531	2,541	-2,446	2,456	7,000
VIII-3	Granitoid Bedrock	2,541	2,551	-2,456	-2,466	9,200
VIII-4	Granitoid Bedrock	2,551	3,085	-2,466	-3,000	9,200

Table 2.5.4-28 Summary Shear Modulus and Damping Ratios for the CCNPP Unit 3 Seismic Evaluation (Page 1 of 2)

Depth 0-25 ft (Terrace Sand)			Depth 25-40 ft (Chesapeake Clay/Silt)			Depth 40-100 ft (Ches. Cemented Sand)			Depth 100-285 ft (Ches. Clay/Silt)			Depth 285-355 ft (Nanjemoy Cemented Clay/Silt)		
Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}
1.E-04	1	1.4	1.E-04	1	1.5	1.E-04	1	1	1.E-04	1	2	1.E-04	1	1.5
3.E-04	1	1.5	3.E-04	1	1.5	3.E-04	1	1	3.E-04	1	2	3.E-04	1	1.5
1.E-03	0.98	1.8	1.E-03	1	1.6	1.E-03	1	1.2	1.E-03	1	2	1.E-03	1	1.6
3.E-03	0.914	2.8	3.E-03	0.97	2.05	3.E-03	0.97	1.64	3.E-03	0.995	2.13	3.E-03	0.97	2.05
1.E-02	0.75	5	1.E-02	0.878	3.21	1.E-02	0.87	2.8	1.E-02	0.955	2.75	1.E-02	0.878	3.21
3.E-02	0.509	9.3	3.E-02	0.685	5.77	3.E-02	0.68	5.49	3.E-02	0.832	4.38	3.E-02	0.685	5.77
1.E-01	0.27	15.3	1.E-01	0.413	10.64	1.E-01	0.43	10.2	1.E-01	0.59	8	1.E-01	0.413	10.64
3.E-01	0.116	21.9	3.E-01	0.208	16.22	3.E-01	0.22	16.5	3.E-01	0.34	13.16	3.E-01	0.208	16.22
1.E+00	0.04	27	6.E-01	0.115	18.65	1.E+00	0.09	22.9	6.E-01	0.22	16.15	6.E-01	0.115	18.65
3.E+00	0.02	30	1.E+00	0.075	19	3.E+00	0.05	27	1.E+00	0.15	17.56	1.E+00	0.075	19

Table 2.5.4-28 Summary Shear Modulus and Damping Ratios for the CCNPP Unit 3 Seismic Evaluation (Page 2 of 2)

Depth 355-456 ft (Nanjemoy Sand)			Depth 456-471 ft (Marlboro Clay)			Depth 471-631 ft (Aquia/Brightseat Sand)			Depth 631-1,731 ft (Patapsco Sand)			Depth 1,731-2,351 ft (Patuxent/Arundel Clay)		
Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}	Cyclic Shear Strain (%)	G/G_{max}	D/D_{max}
1.E-04	1	0.7	1.E-04	1	1.5	1.E-04	1	0.6	1.E-04	1	0.55	1.E-04	1	1.5
3.E-04	1	0.8	3.E-04	1	1.5	3.E-04	1	0.6	3.E-04	1	0.55	3.E-04	1	1.5
1.E-03	1	0.8	1.E-03	1	1.6	1.E-03	1	0.6	1.E-03	1	0.55	1.E-03	1	1.6
3.E-03	0.988	1.12	3.E-03	0.97	2.05	3.E-03	0.99	0.81	3.E-03	1	0.77	3.E-03	0.97	2.05
1.E-02	0.93	1.8	1.E-02	0.878	3.21	1.E-02	0.95	1.2	1.E-02	0.96	1.15	1.E-02	0.878	3.21
3.E-02	0.791	3.53	3.E-02	0.685	5.77	3.E-02	0.852	2.5	3.E-02	0.88	2.1	3.E-02	0.685	5.77
1.E-01	0.57	7.1	1.E-01	0.413	10.64	1.E-01	0.65	5.3	1.E-01	0.71	4.2	1.E-01	0.413	10.64
3.E-01	0.321	12.78	3.E-01	0.208	16.22	3.E-01	0.41	10.27	3.E-01	0.47	8.45	3.E-01	0.208	16.22
1.E+00	0.15	19.3	6.E-01	0.115	18.65	1.E+00	0.2	16.7	1.E+00	0.265	14.5	6.E-01	0.115	18.65
3.E+00	0.09	23	1.E+00	0.075	19	3.E+00	0.1	20.1	3.E+00	0.16	17.4	1.E+00	0.075	19

Table 2.5.4-29 Material Density and PI Adopted for the CCNPP Unit 3 Seismic Evaluation
(Page 1 of 1)

Unit	Soil	Est. Total Unit Weight (pcf)	Est. PI
I	Terrace SAND	120	NP
II-a	Chesapeake CLAY/SILT	119	30
II-b	Chesapeake Cemented SAND	122	NP
II-c	Chesapeake CLAY/SILT	115	50
III-a	Nanjemoy Cemented CLAY/SILT	115	30
III-b	Nanjemoy SAND	120	NP
IV	Marlboro CLAY	115	30
V	Aquia/Brightseat SAND	115	NP
VI	Patapsco SAND	115	NP
VII	Patuxent/Arundel CLAY	115	30

NP = Non-Plastic

PI = Plastic Index

Note:

The information in this table was prepared during the early stages of the geotechnical investigation, based on limited laboratory test results. These values are slightly different than those compiled from the complete laboratory investigation as summarized in Table 2.5.4-8.

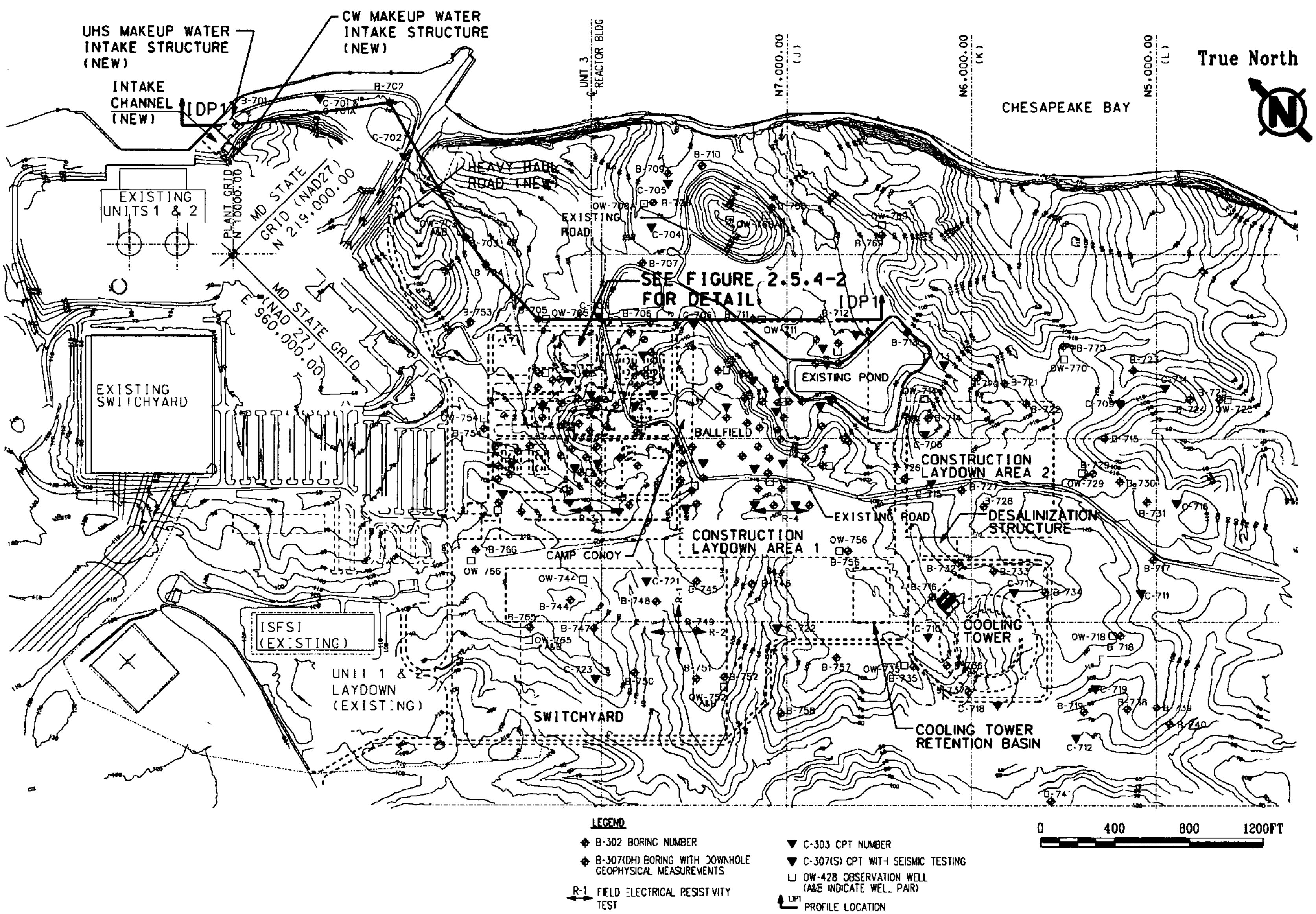
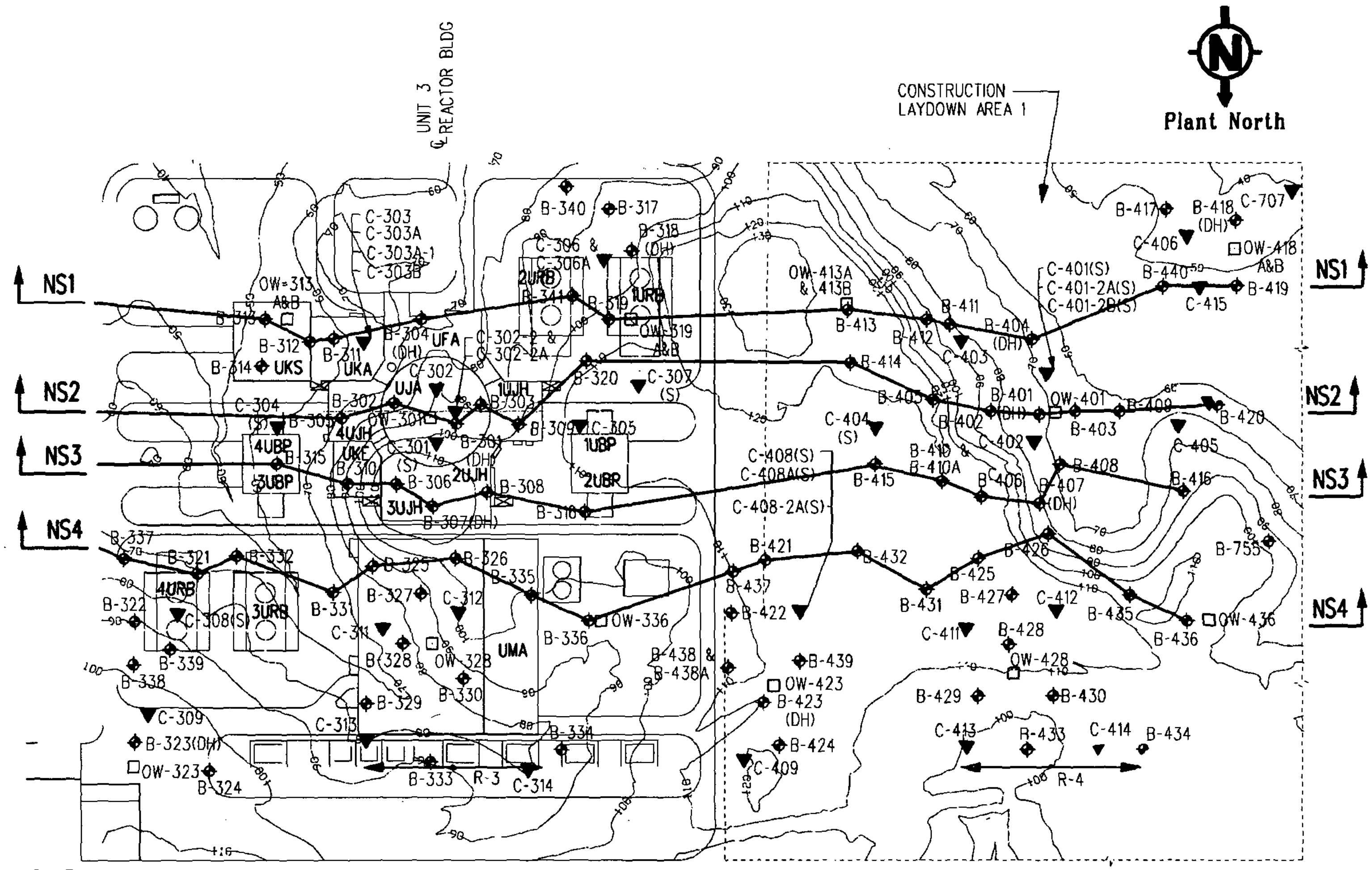


FIGURE 2.5.4-1 **Rev. 0**

SUBSURFACE INVESTIGATION LOCATION PLAN

CCNPP UNIT 3 FSAR



LEGEND

1-4 UBP	EMERGENCY DIESEL GENERATOR BUILDING
UFA	FUEL BUILDING
UJA	REACTOR BUILDING
1-4 UJH	SAFEGUARD BUILDING
UKA	NUCLEAR AUXILIARY BUILDING
UKE	ACCESS BUILDING
UKS	RADIOACTIVE WASTE BUILDING
UMA	TURBINE BUILDING
1-4 URB	ULTIMATE HEAT SINK

B-302 BORING NUMBER
B-307(DH) BORING WITH DOWNHOLE GEOPHYSICAL MEASUREMENTS
C-303 CPT NUMBER
C-307(S) CPT WITH SEISMIC TESTING
OW-428 OBSERVATION WELL (A&B INDICATE WELL PAIR)
R-1 FIELD ELECTRICAL RESISTIVITY TEST
NS1 PROFILE LOCATION

0 100 200 300FT

FIGURE 2.5.4-2 Rev. 0

SUBSURFACE INVESTIGATION LOCATION PLAN (POWERBLOCK)

CCNPP UNIT 3 FSAR

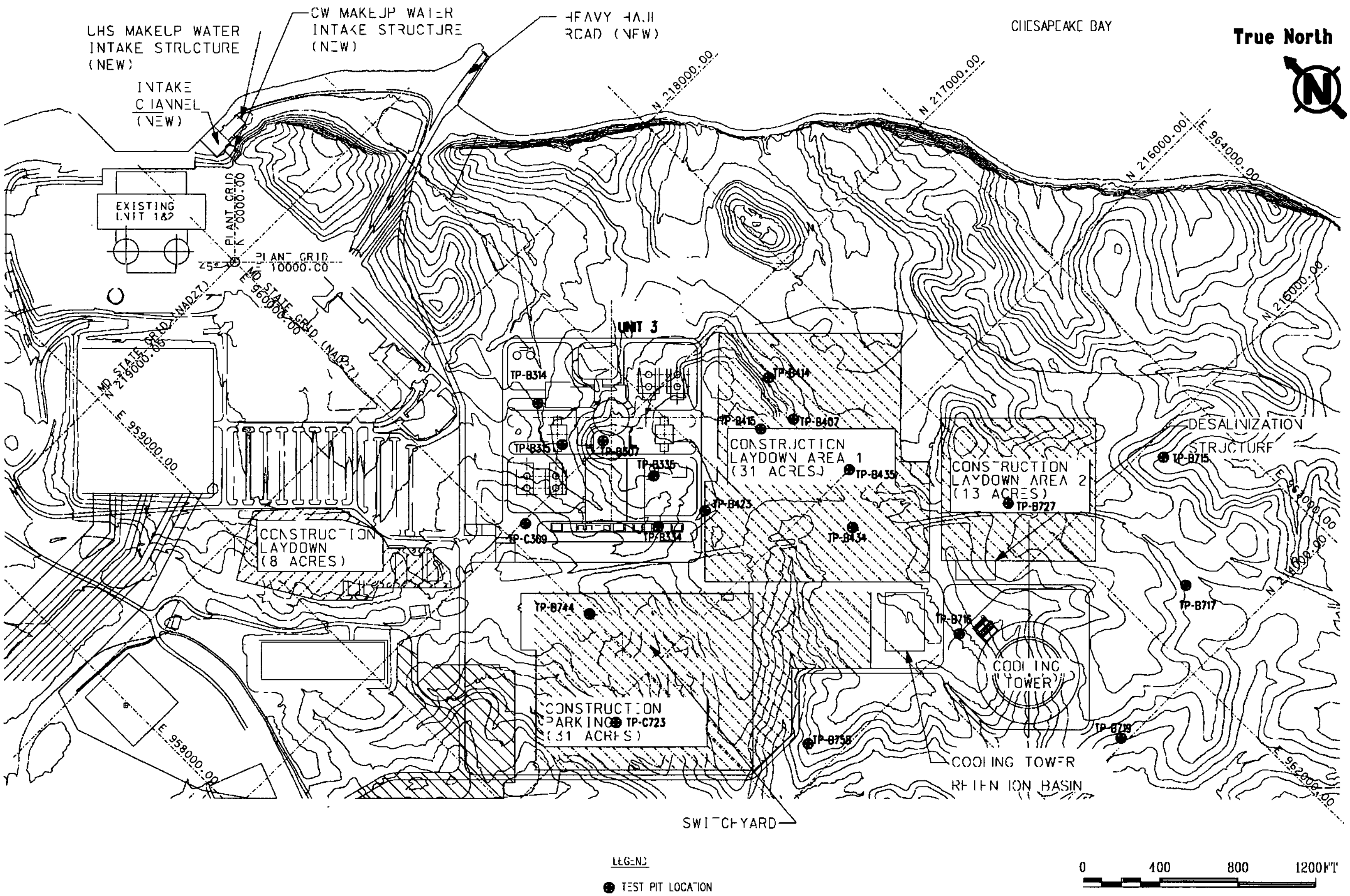
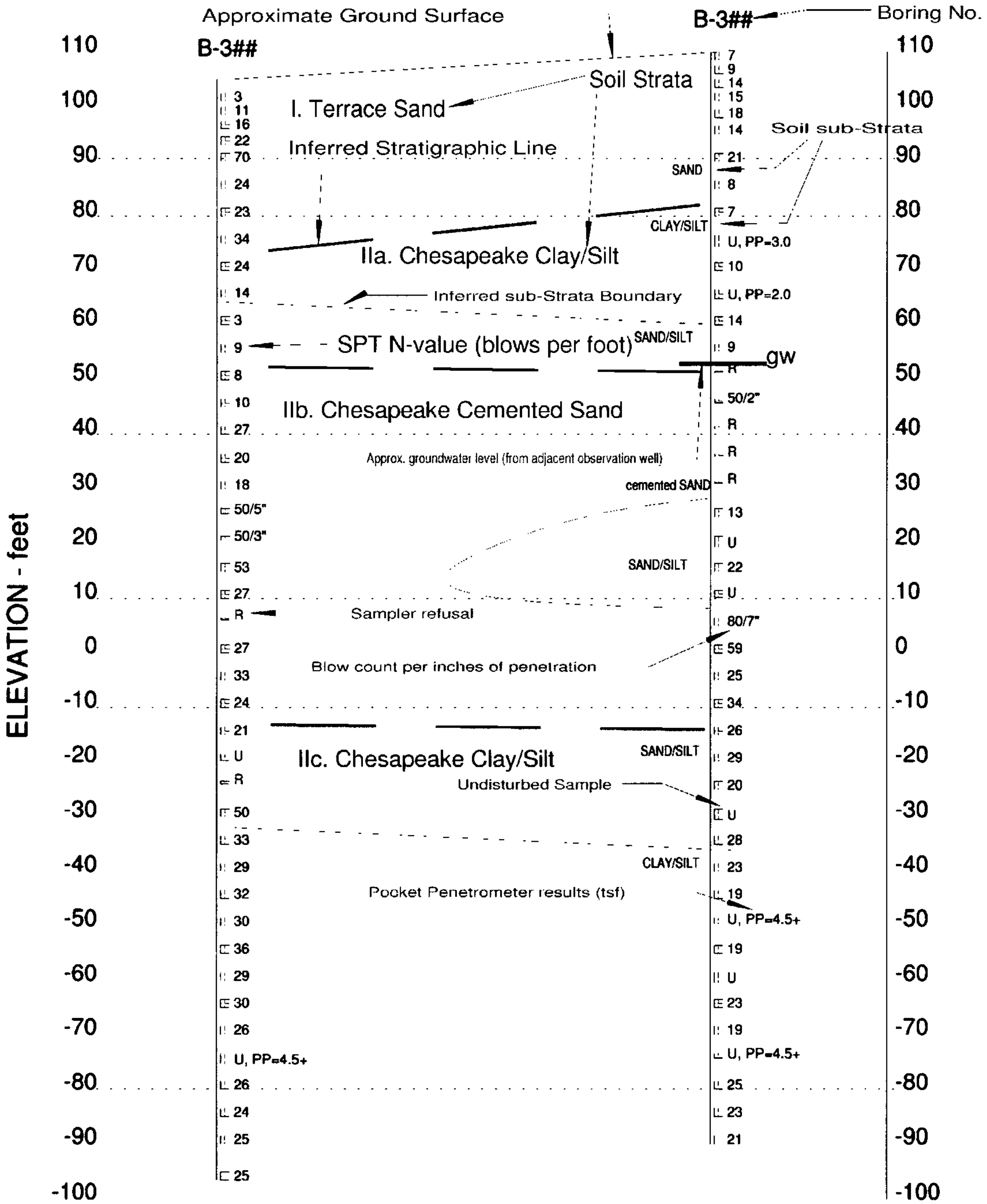


FIGURE 2.5.4-3 Rev. 0

TEST PIT TEST LOCATION PLAN

CCNPP UNIT 3 FSAR



Notes - Boring locations & elevations were obtained from as-built survey.

Soil strata boundaries are inferred from boring locations & may differ from actual soil conditions.

0 50 100
HORIZONTAL SCALE - FEET

FIGURE 2.5.4-4 **Rev. 0**

SUBSURFACE PROFILE LEGEND

CCNPP UNIT 3 FSAR

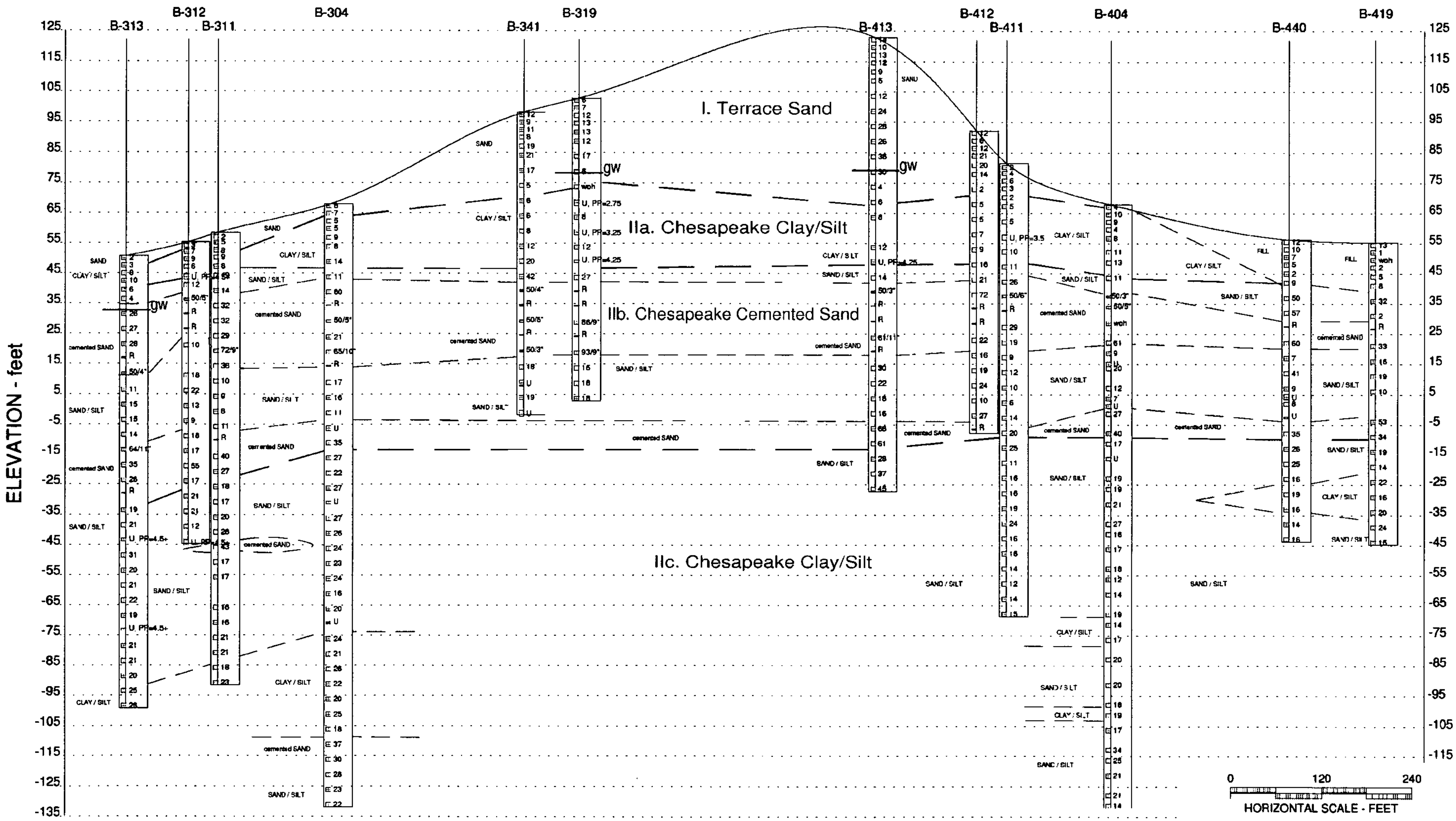


FIGURE 2.5.4-5 **Rev. 0**

INFERRED SUBSURFACE PROFILE NS-1

CCNPP UNIT 3 FSAR

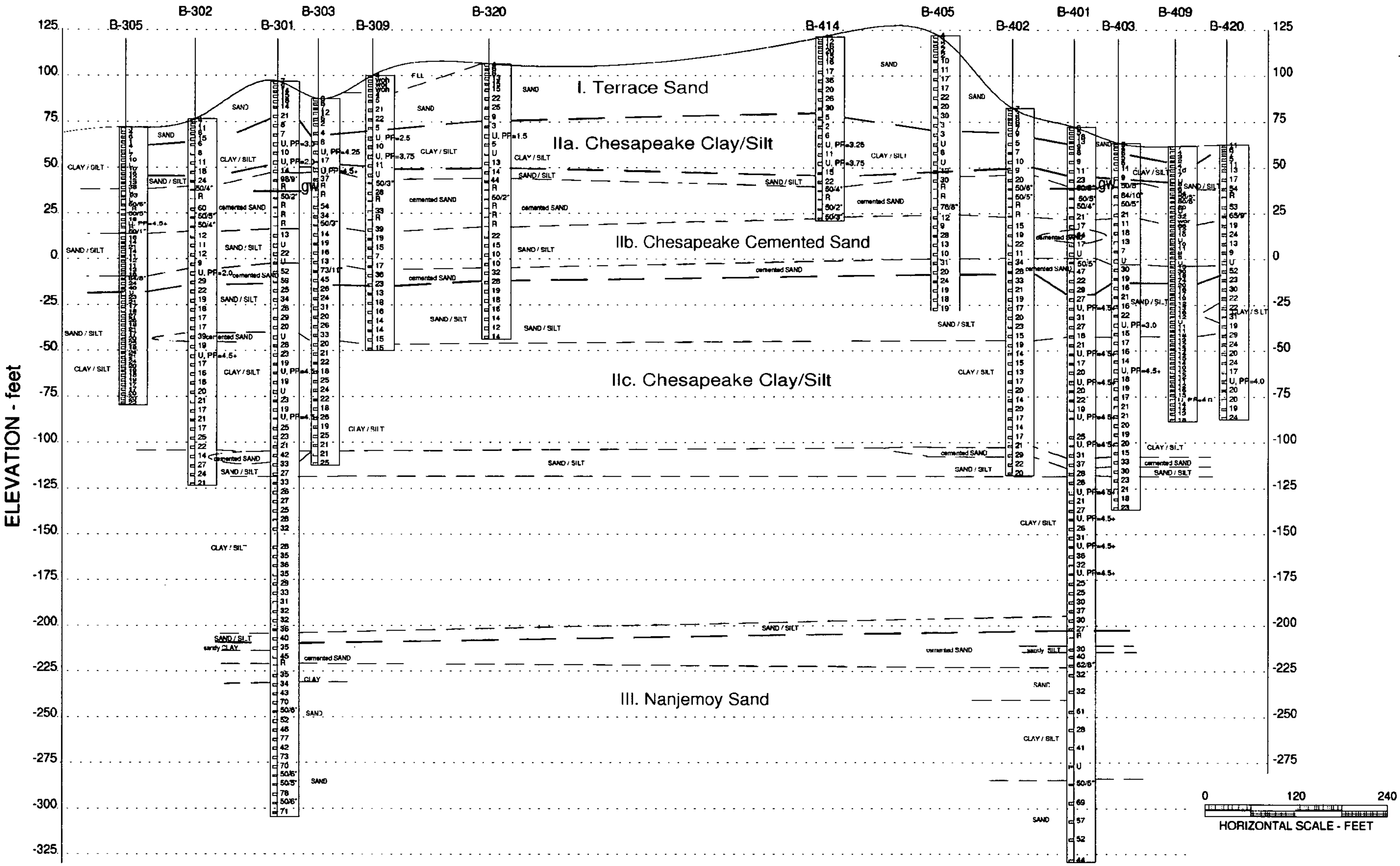


FIGURE 2.5.4-6 **Rev. 0**

INFERRED SUBSURFACE PROFILE NS-2

CCNPP UNIT 3 FSAR

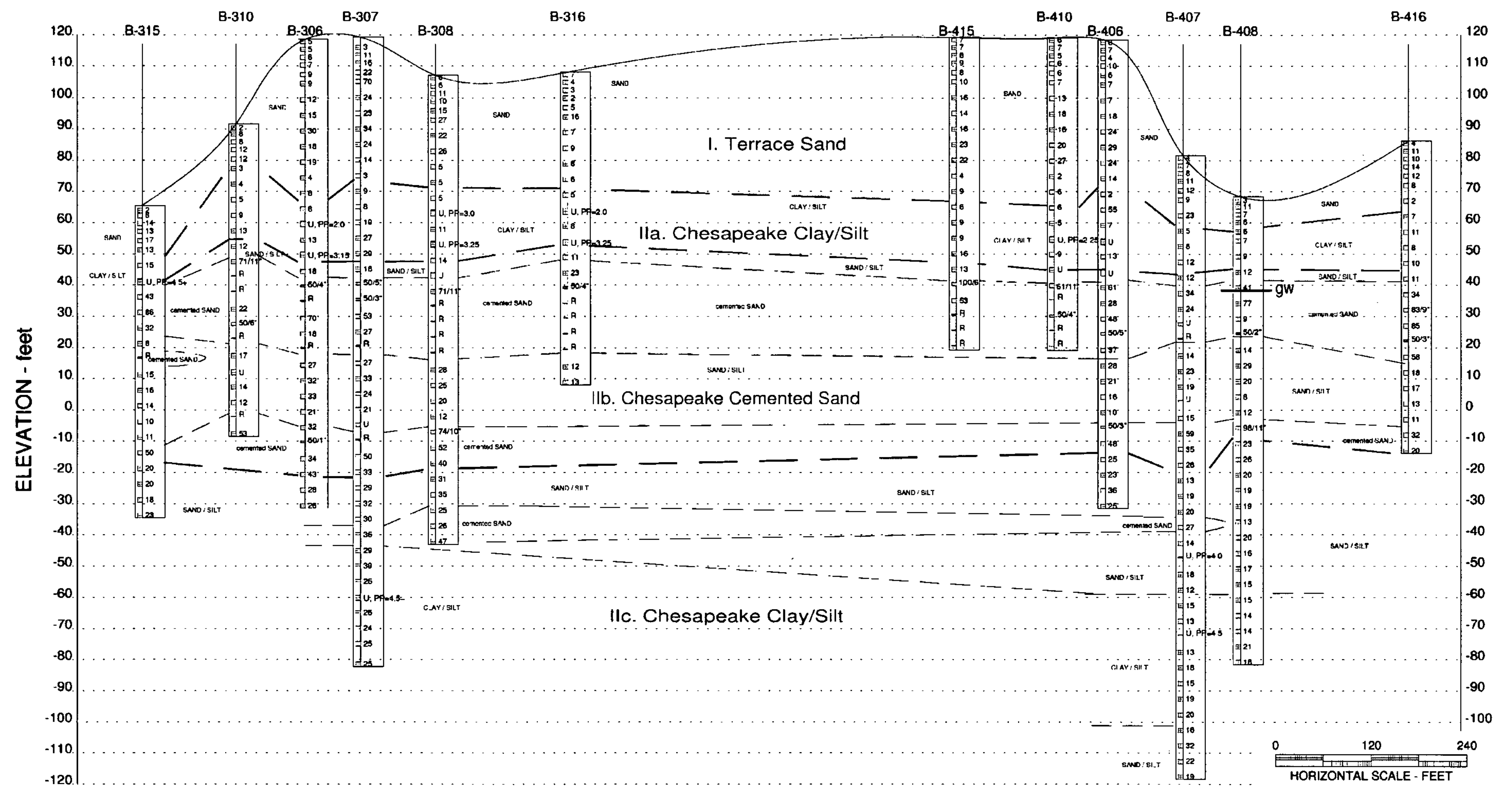


FIGURE 2.5.4-7 **Rev. 0**

INFERRED SUBSURFACE PROFILE NS-3

CCNPP UNIT 3 FSAR

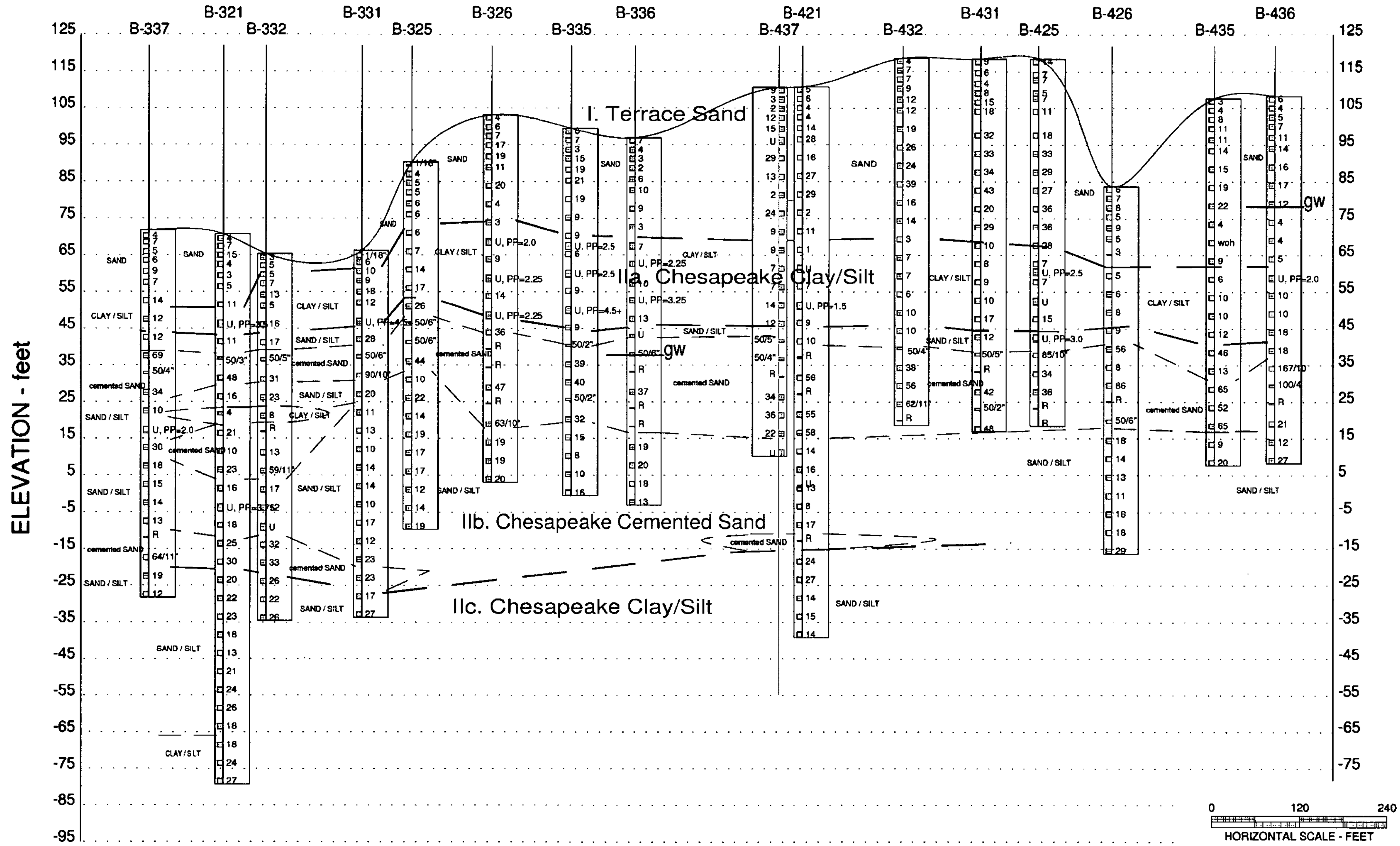


FIGURE 2.5.4-8 Rev. 0

INFERRED SUBSURFACE PROFILE NS-4

CCNPP UNIT 3 FSAR

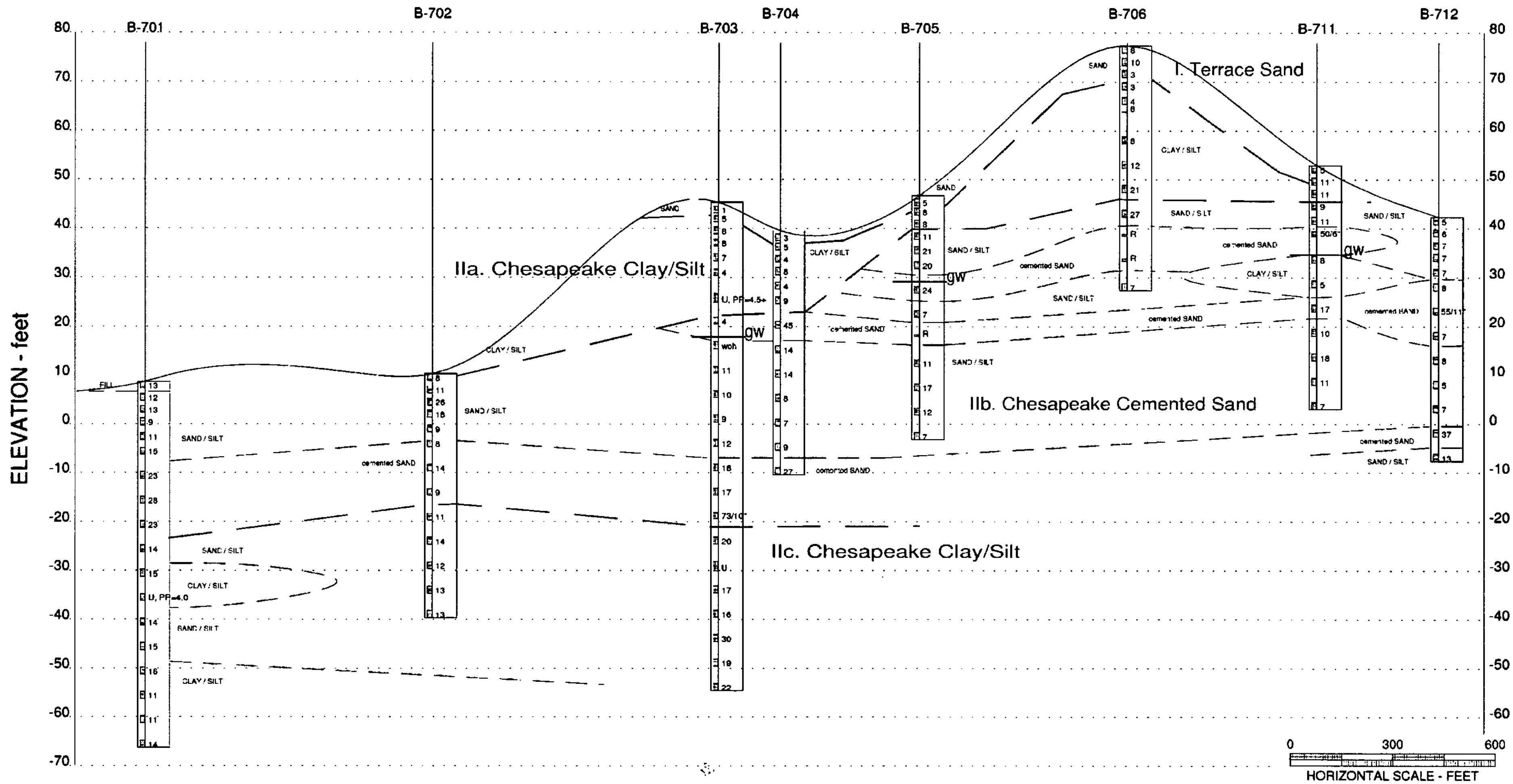


FIGURE 2.5.4-9 **Rev. 0**

INFERRED SUBSURFACE PROFILE IDP-1

CCNPP UNIT 3 FSAR

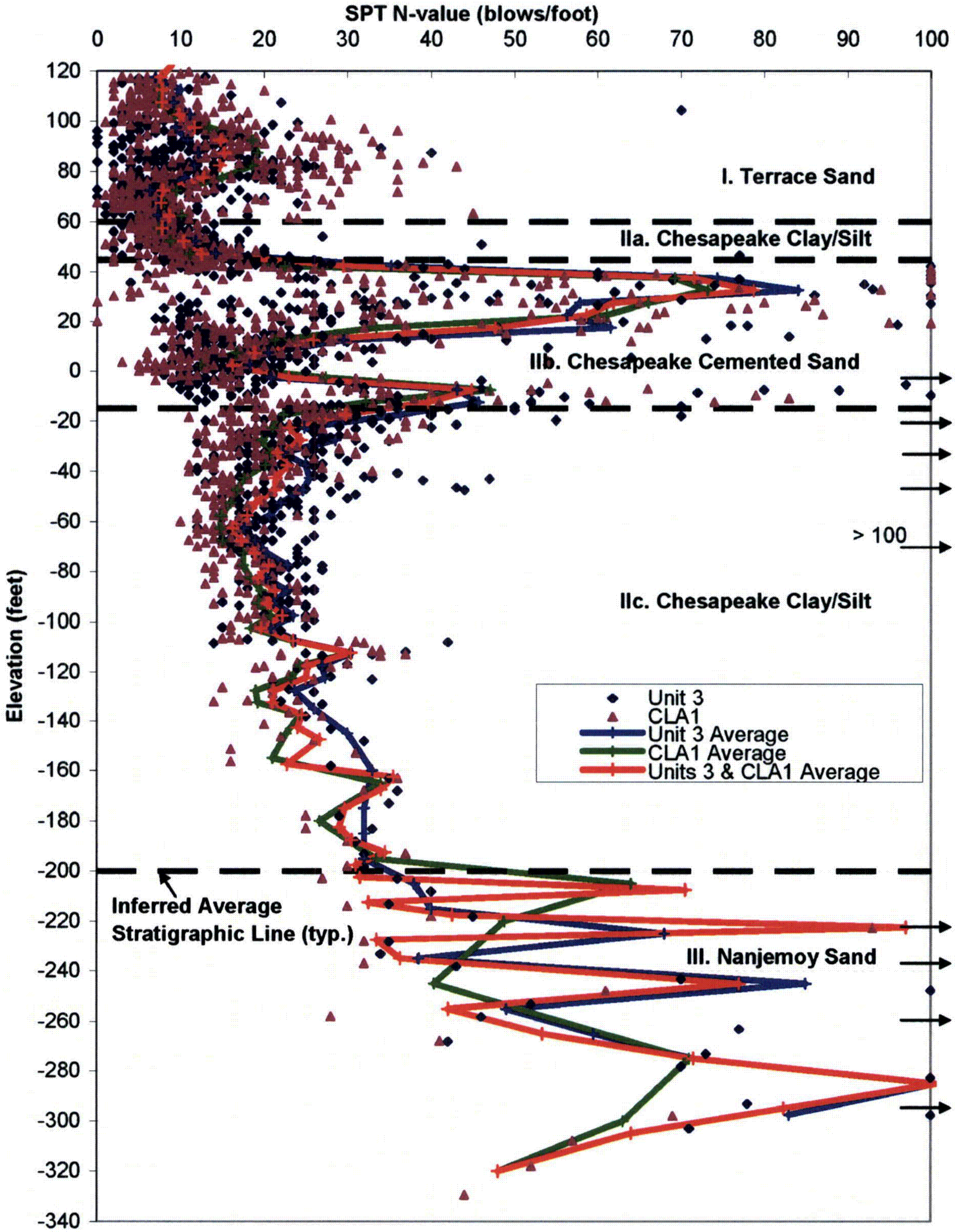


FIGURE 2.5.4-10 **Rev. 0**

MEASURED STANDARD PENETRATION TEST N-VALUES

CCNPP UNIT 3 FSAR

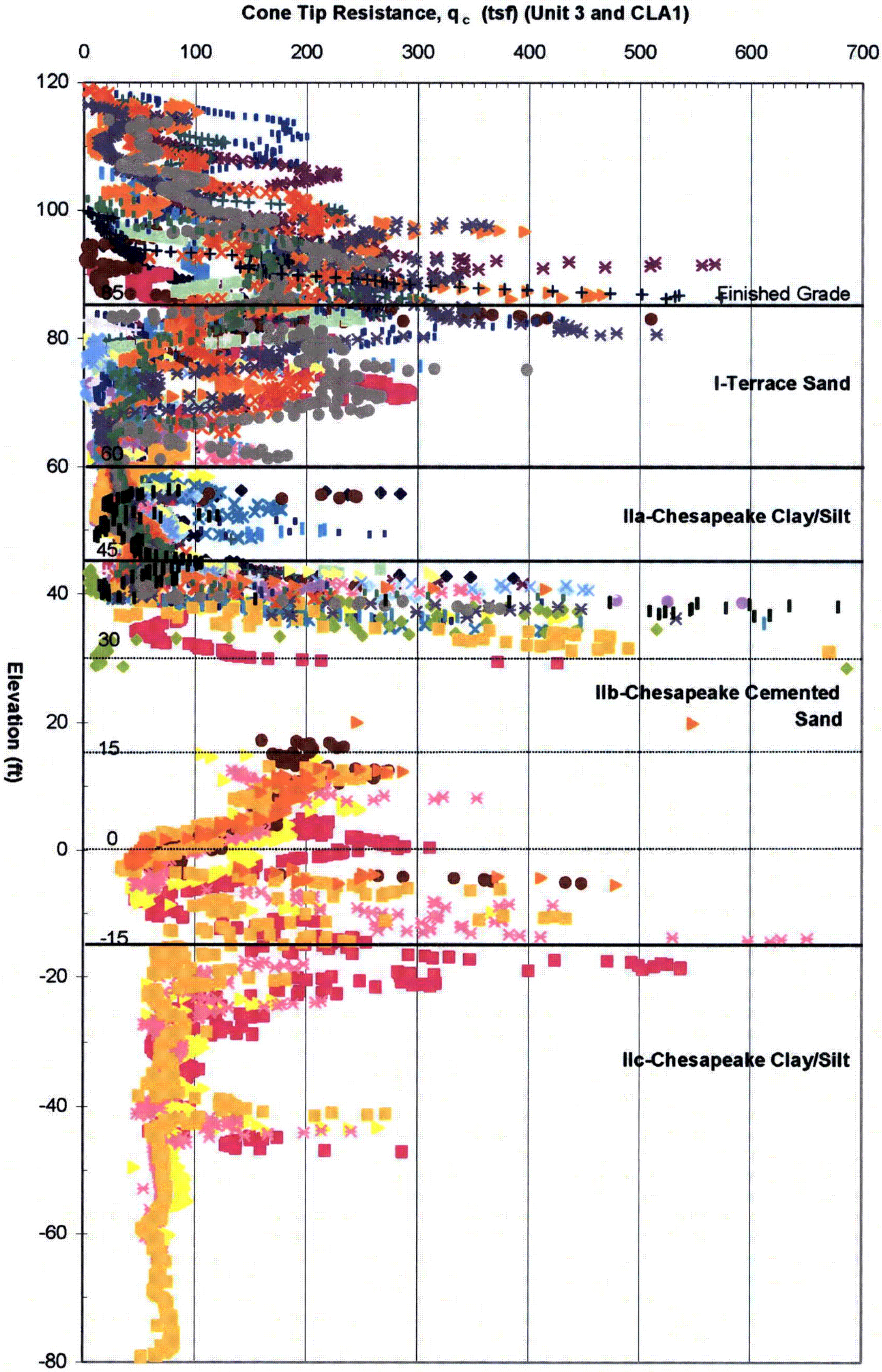


FIGURE 2.5.4-11 Rev. 0

MEASURED CPT TIP RESISTANCE VALUES

CCNPP UNIT 3 FSAR

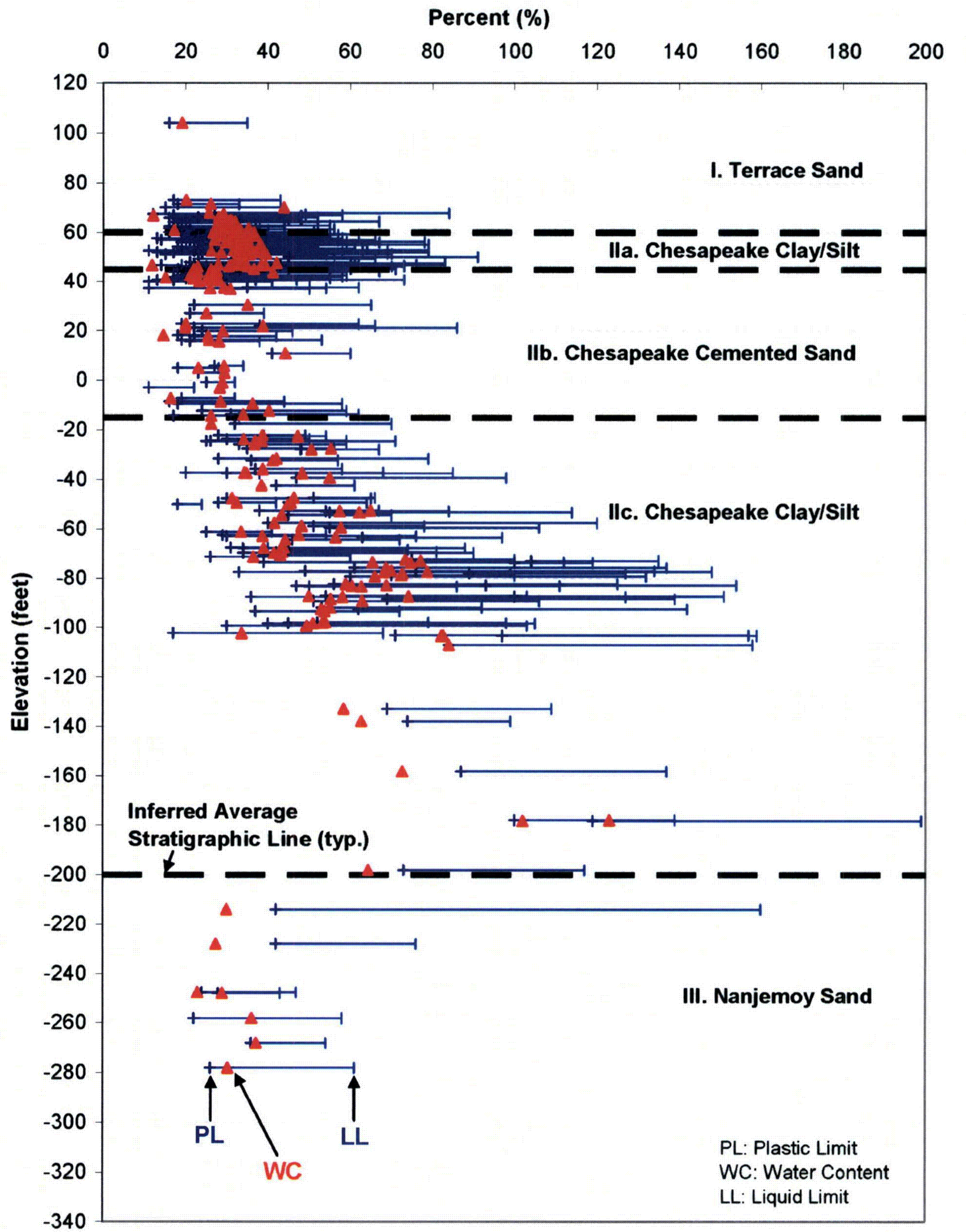


FIGURE 2.5.4-12 **Rev. 0**

WATER CONTENTS AND LIMITS PROFILE

CCNPP UNIT 3 FSAR

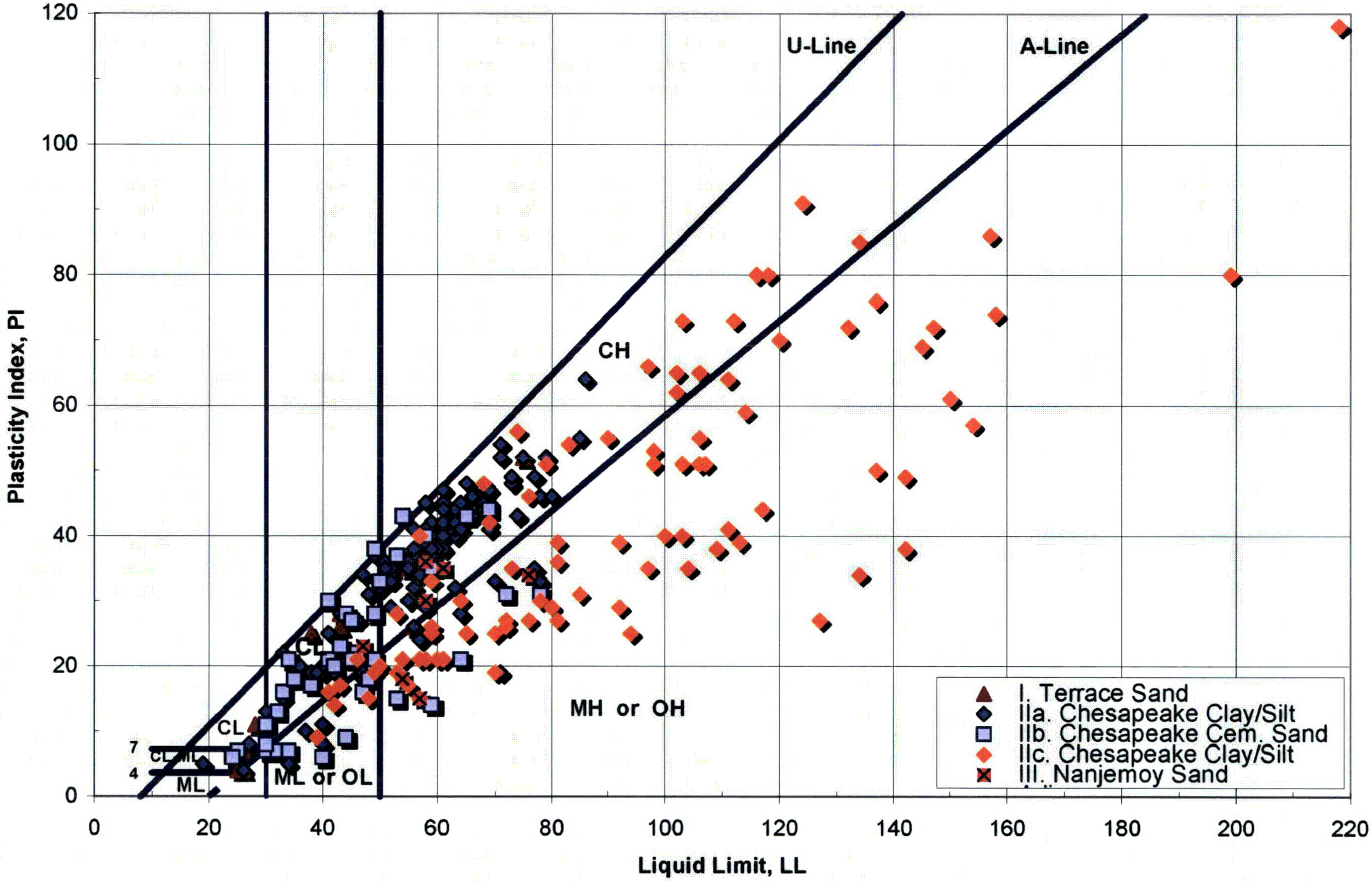


FIGURE 2.5.4-13 Rev. 0

PLASTICITY CHART

CCNPP UNIT 3 FSAR

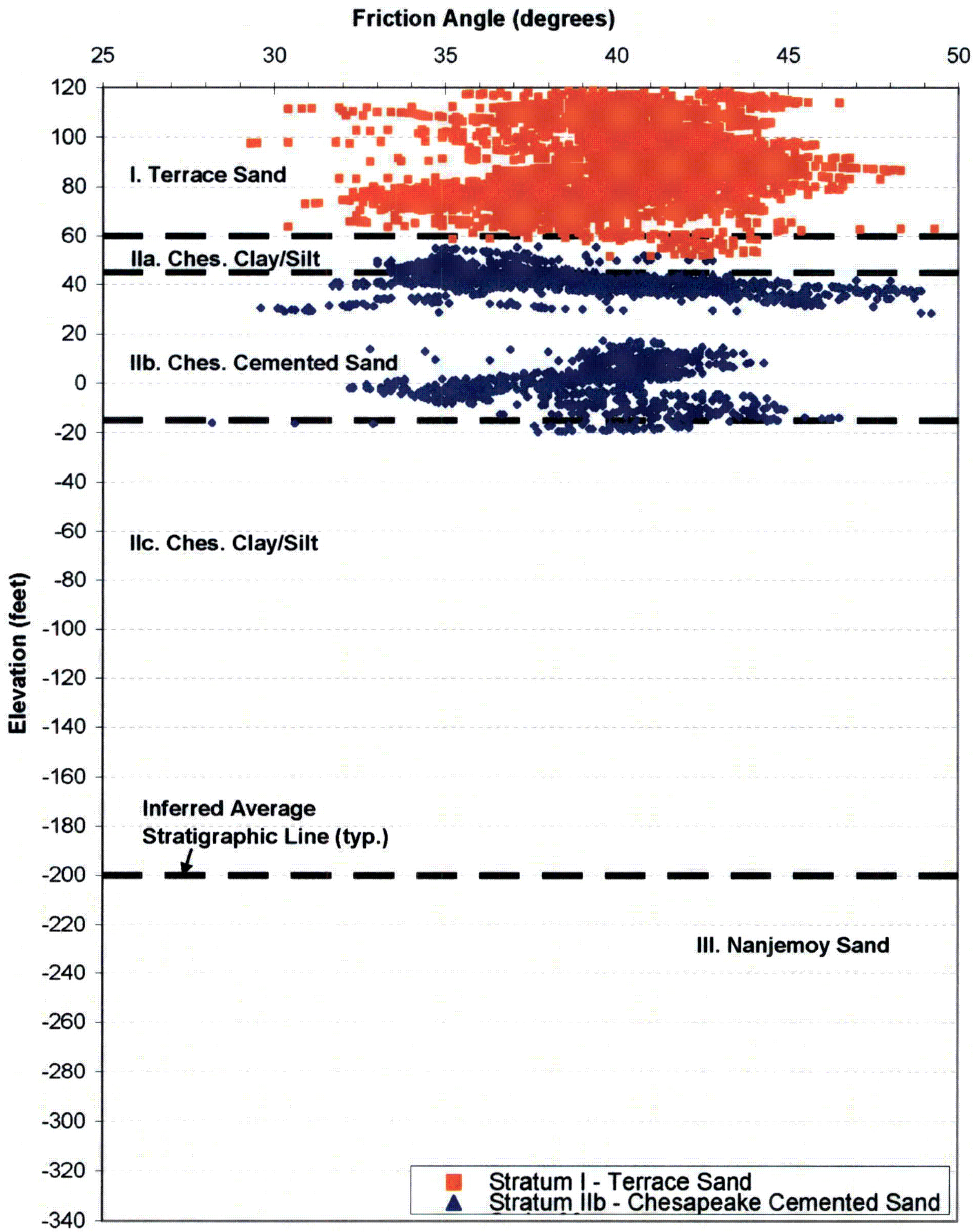


FIGURE 2.5.4-14 **Rev. 0**

FRICTION ANGLE INTERPRETATION FROM CPT RESULTS

CCNPP UNIT 3 FSAR

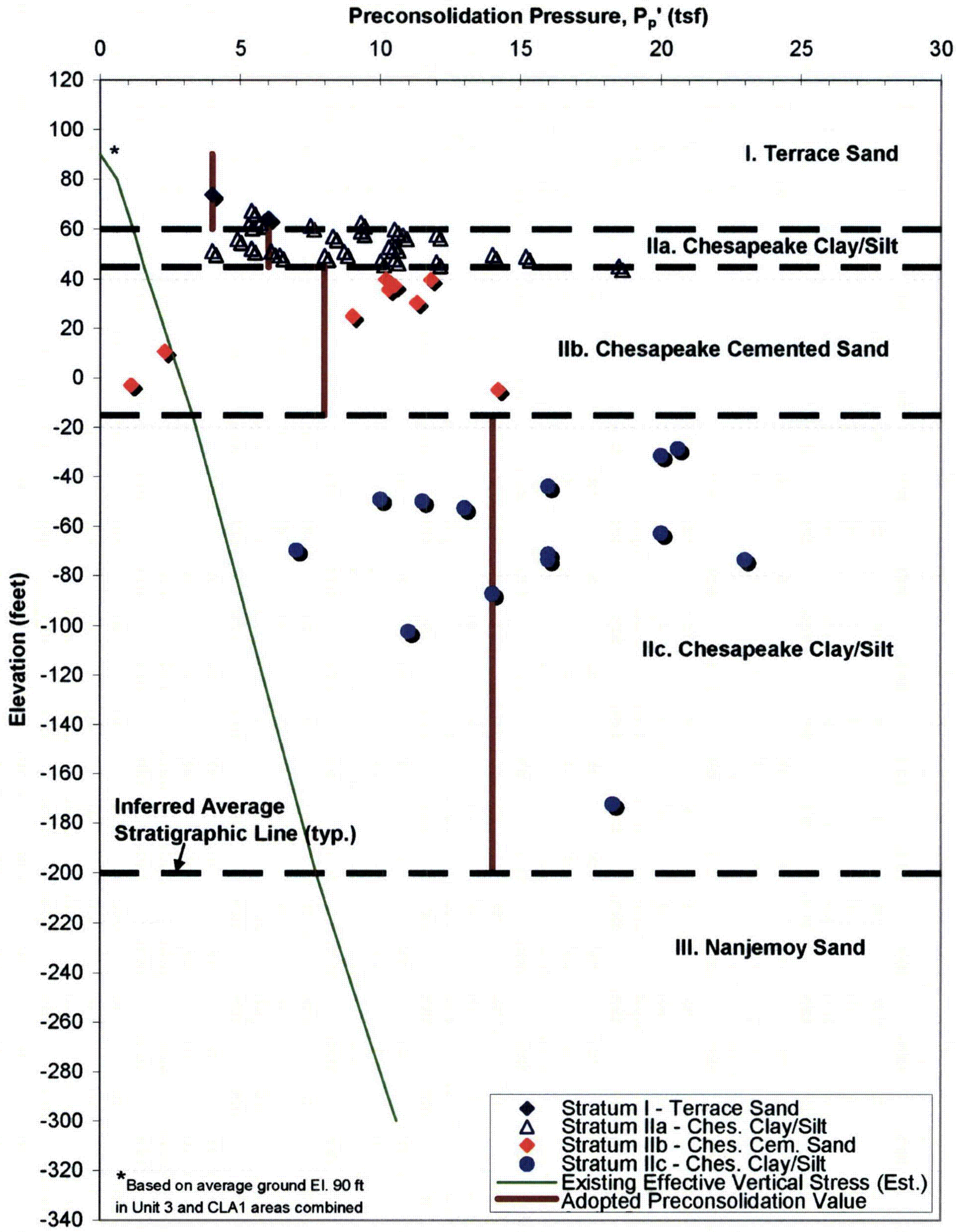


FIGURE 2.5.4-15 Rev. 0

PRECONSOLIDATION PRESSURE FROM LABORATORY TESTING

CCNPP UNIT 3 FSAR

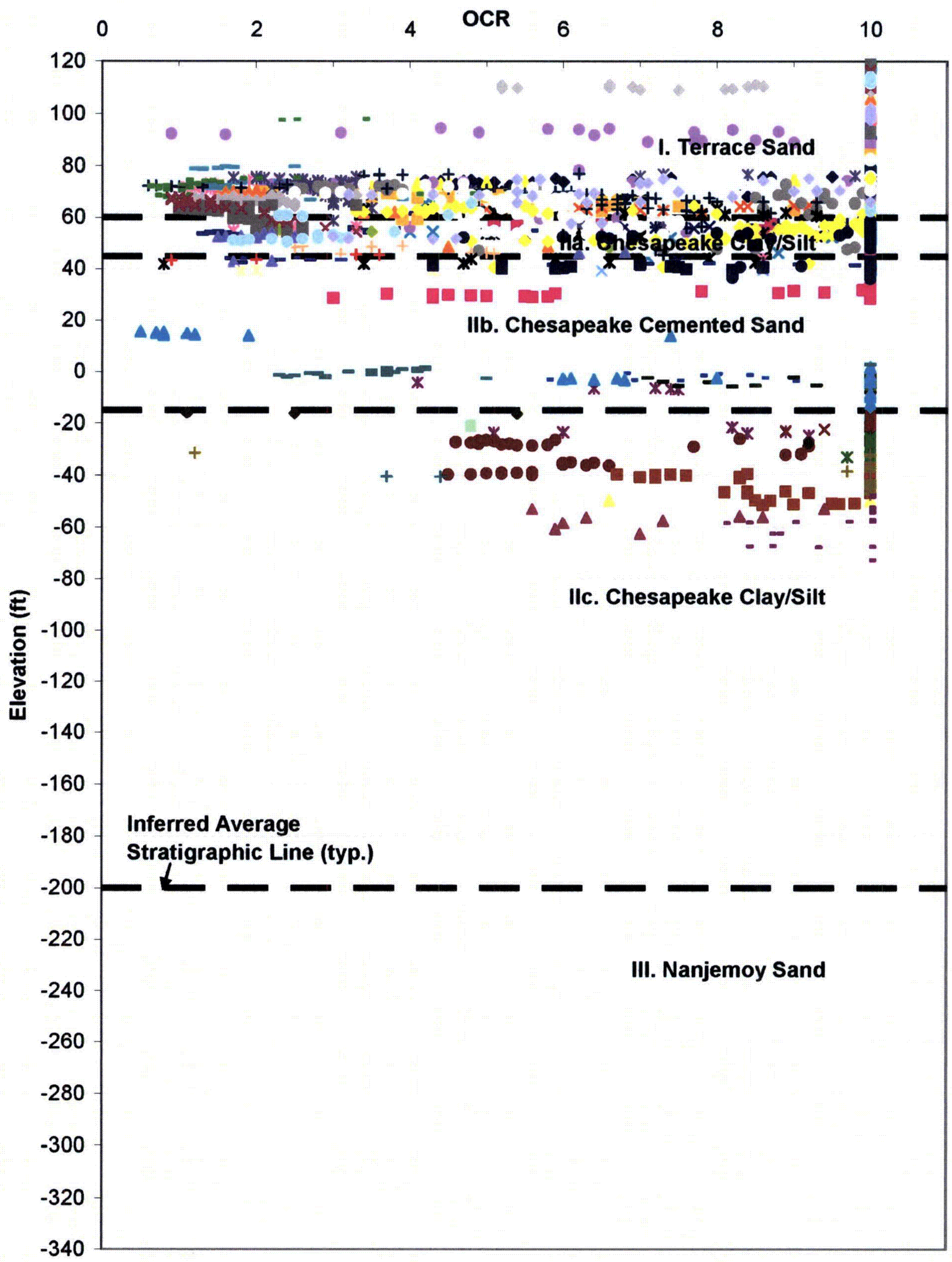


FIGURE 2.5.4-16 **Rev. 0**

OCR INTERPRETATION FROM CPT RESULTS

CCNPP UNIT 3 FSAR

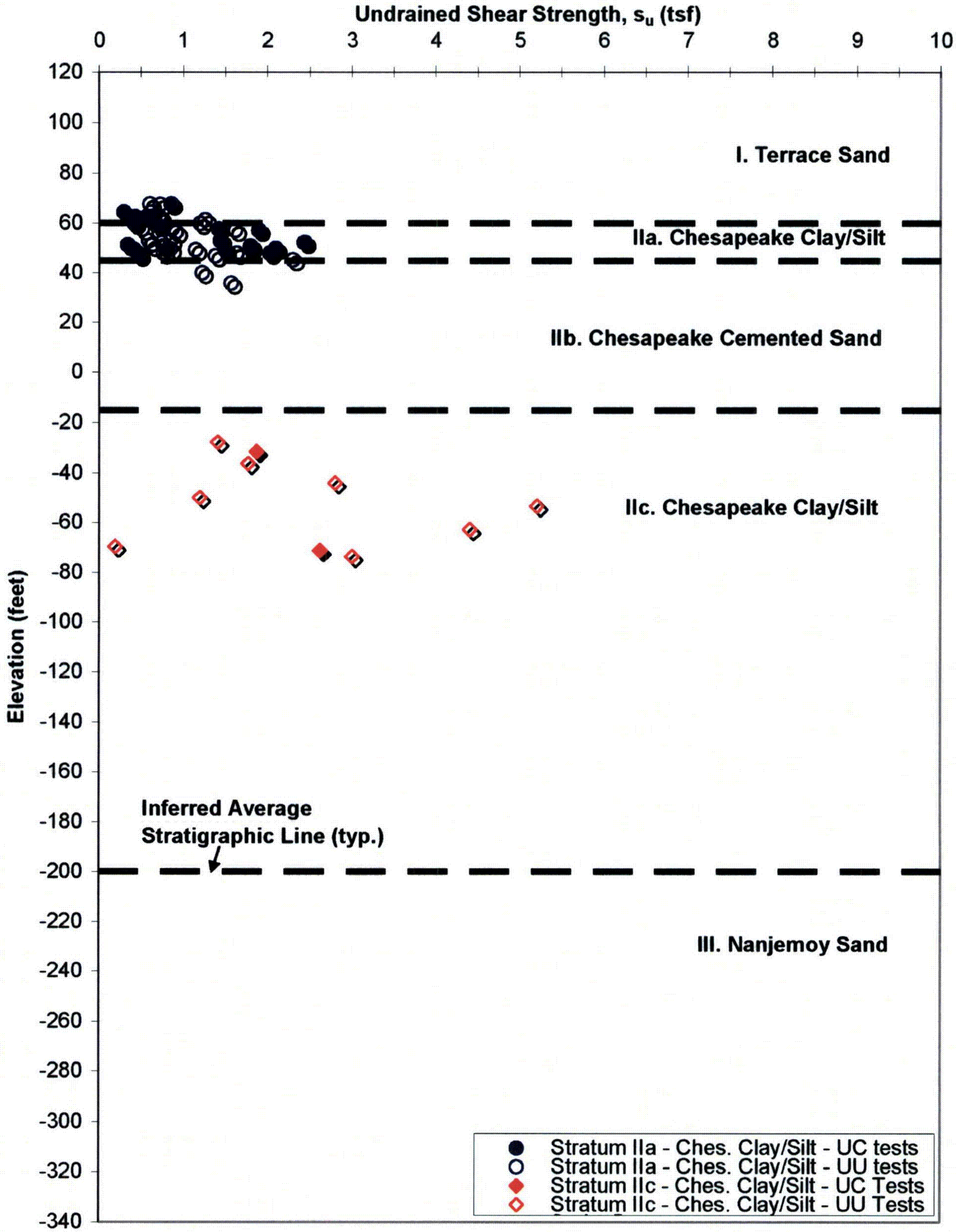


FIGURE 2.5.4-17 **Rev. 0**

UNDRAINED SHEAR STRENGTH FROM LABORATORY TESTS

CCNPP UNIT 3 FSAR

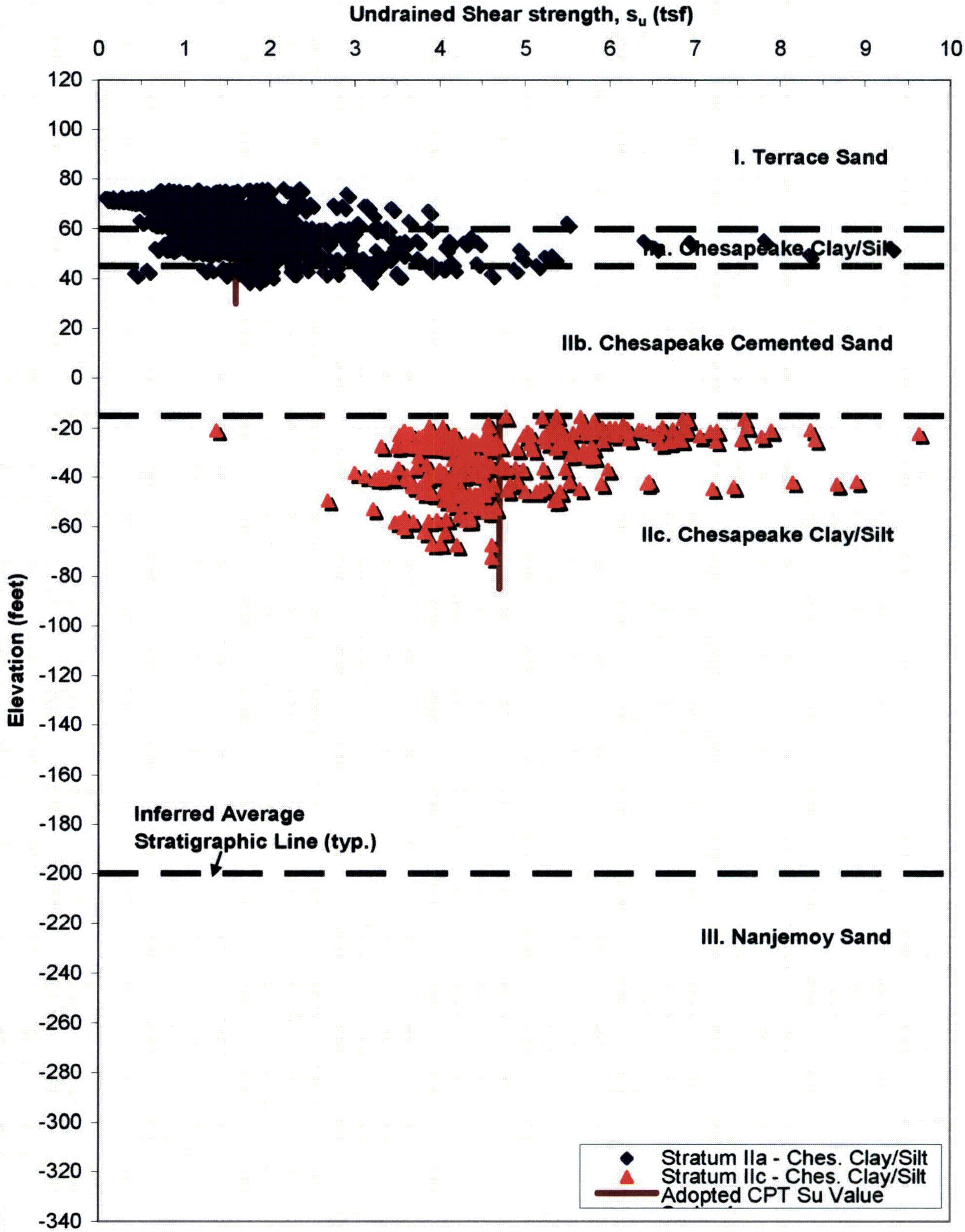


FIGURE 2.5.4-18 **Rev. 0**

UNDRAINED SHEAR STRENGTH INTERPRETED FROM CPT DATA

CCNPP UNIT 3 FSAR

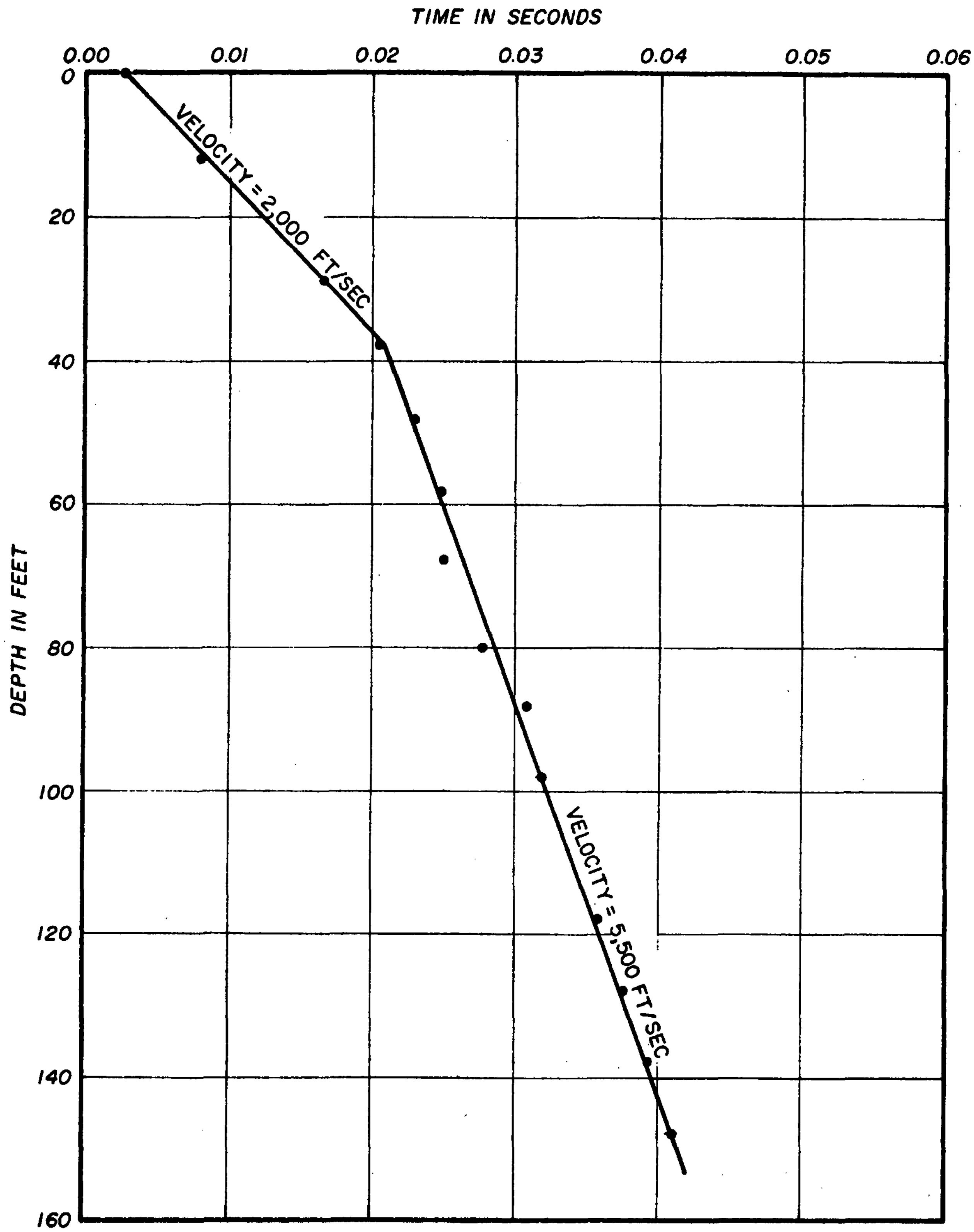
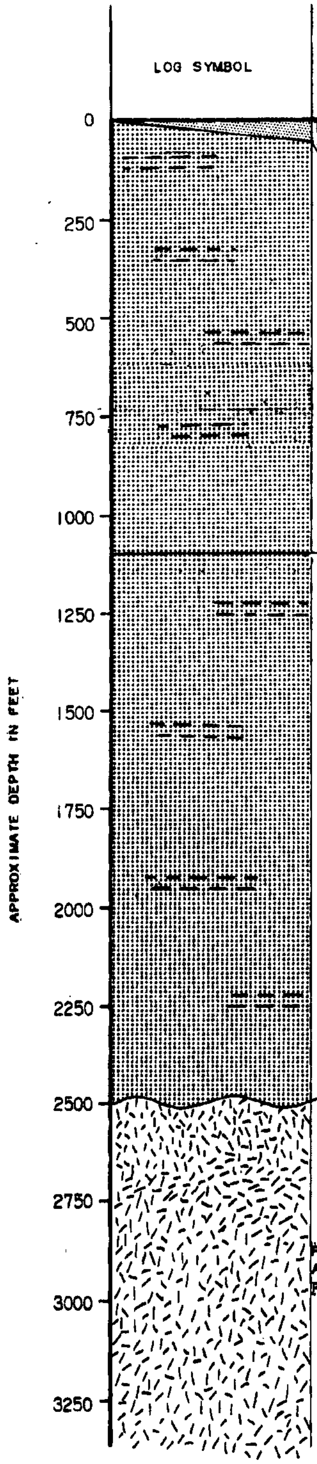


FIGURE 2.5.4-19 **Rev. 0**

UPHOLE SEISMIC SURVEY RESULTS FROM UFSAR(1)

CCNPP UNIT 3 FSAR



DESCRIPTION	COMPRESSIONAL WAVE VELOCITY (FT/SEC) (MEASURED)	POISSON'S RATIO	SHEAR WAVE VELOCITY (FT/SEC)	UNIT WEIGHT (LBS/ CU.FT.)
PLEISTOCENE SILT, SAND, CLAY AND GRAVEL	2200 ± 200	0.32 (ESTIMATED)	500 (COMPUTED)	120 (MEASURED)
MIOCENE, EOCENE AND PALEOCENE SAND, SILT AND CLAY.	5900 ± 300	0.46 (COMPUTED)	1600 (MEASURED)	120 TO 135 (MEASURED)
CRETACEOUS SAND, SILT AND CLAY	*7000 ± 400	0.35 (ESTIMATED)	3400 (COMPUTED)	130 TO 135 (ESTIMATED)
PRECAMBRIAN IGNEOUS AND METAMORPHIC ROCK	*16,600 ± 1000	0.15 (ESTIMATED)	10,000 (COMPUTED)	170 (ESTIMATED)

* TAKEN FROM EWING M. AND WORZEL L. (1948) EXPLOSION SOUNDS IN SHALLOW WATER, GEOLOGICAL SOCIETY OF AMERICA MEMOIR 27

FIGURE 2.5.4-20 Rev. 0

GEOPHYSICAL MODEL OF THE SITE FOR UNITS 1 AND 2 FROM UFSAR(1)

CCNPP UNIT 3 FSAR

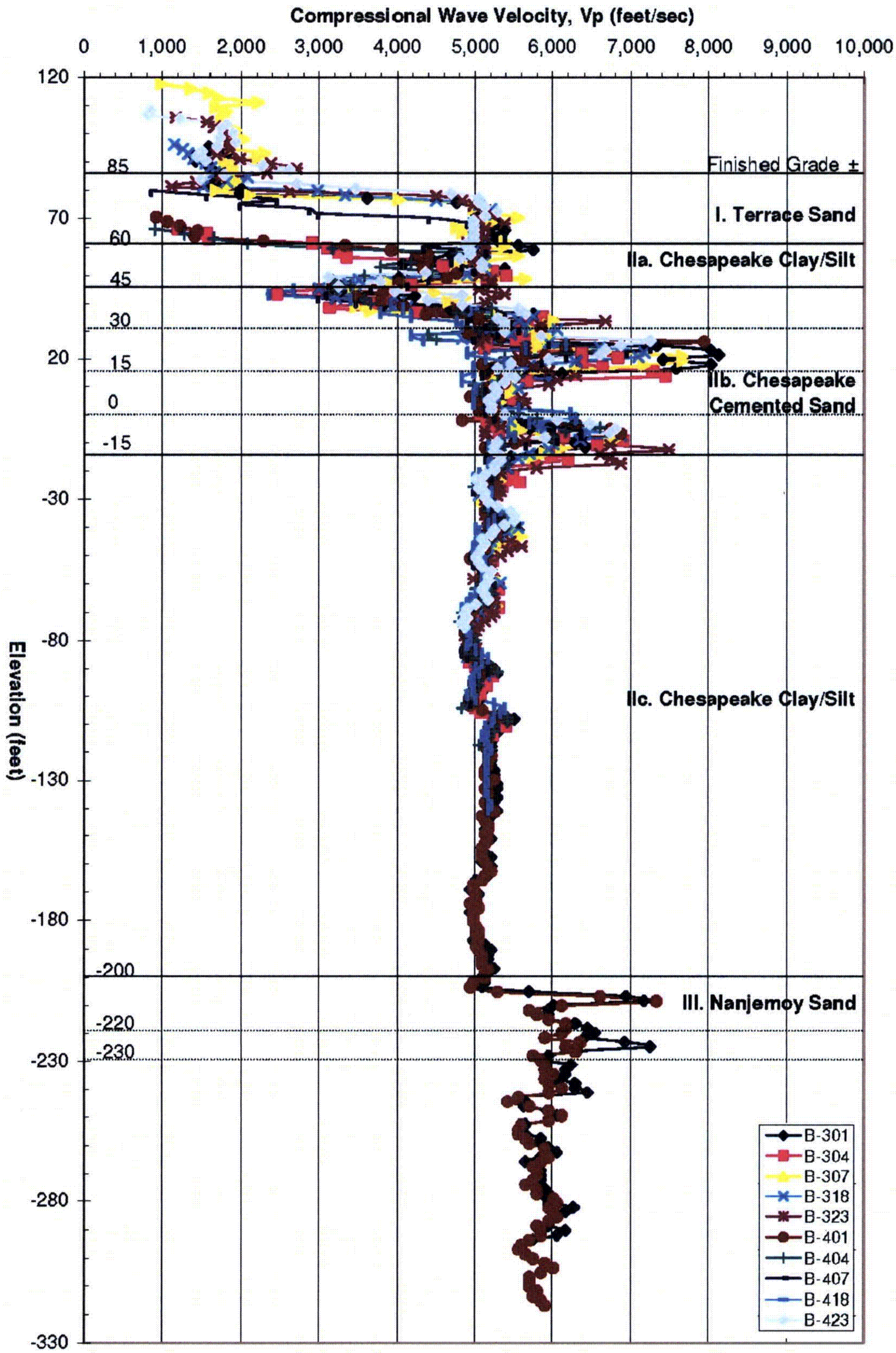


FIGURE 2.5.4-21 **Rev. 0**

V_p MEASUREMENTS FROM SUSPENSION P–S VELOCITY LOGGING

CCNPP UNIT 3 FSAR

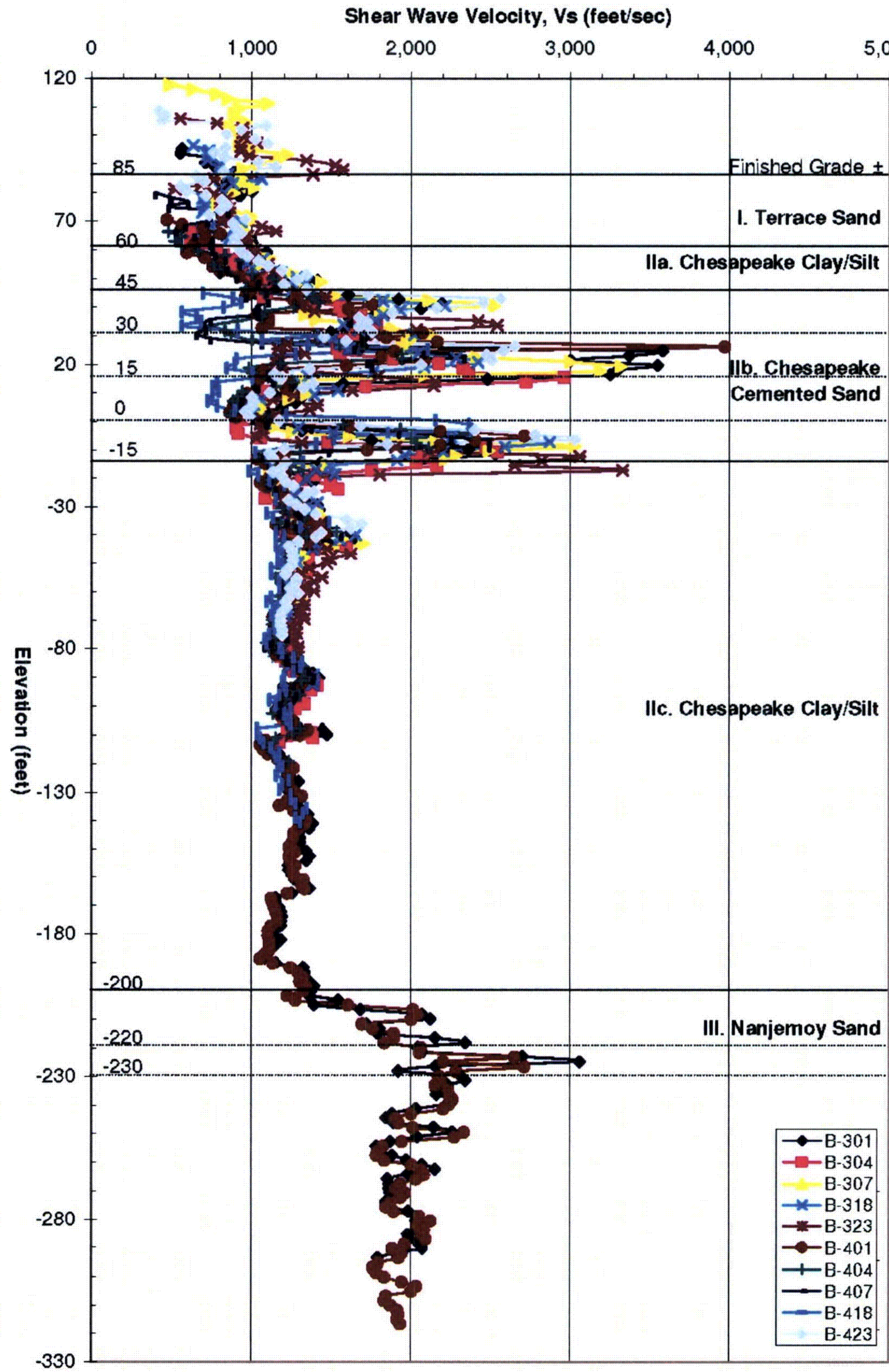


FIGURE 2.5.4-22 Rev. 0

V_S MEASUREMENTS FROM SUSPENSION P–S VELOCITY LOGGING

CCNPP UNIT 3 FSAR

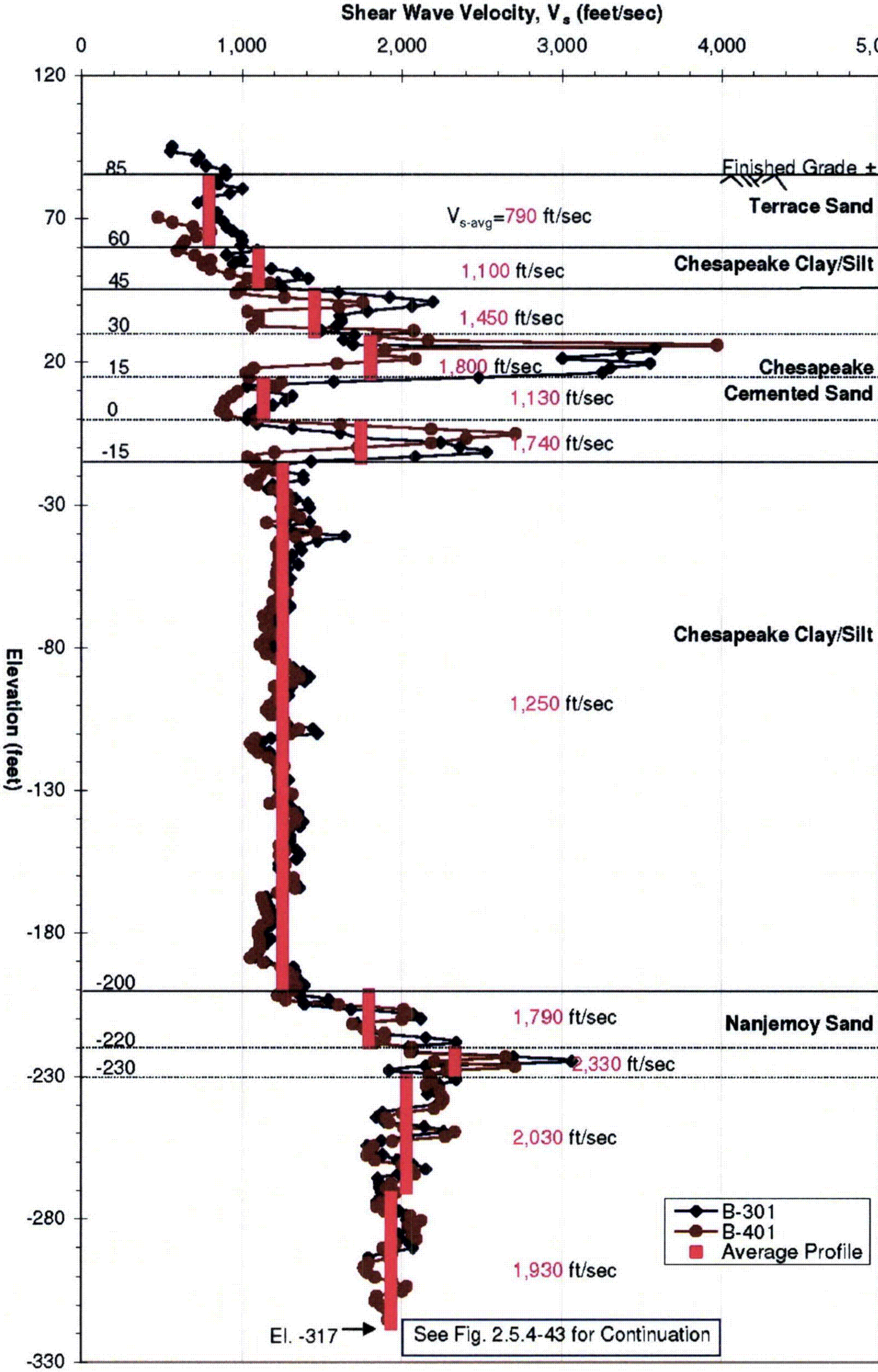


FIGURE 2.5.4-23 **Rev. 0**

AVERAGE V_S MEASUREMENTS FROM SUSPENSION P–S VELOCITY LOGGING

CCNPP UNIT 3 FSAR

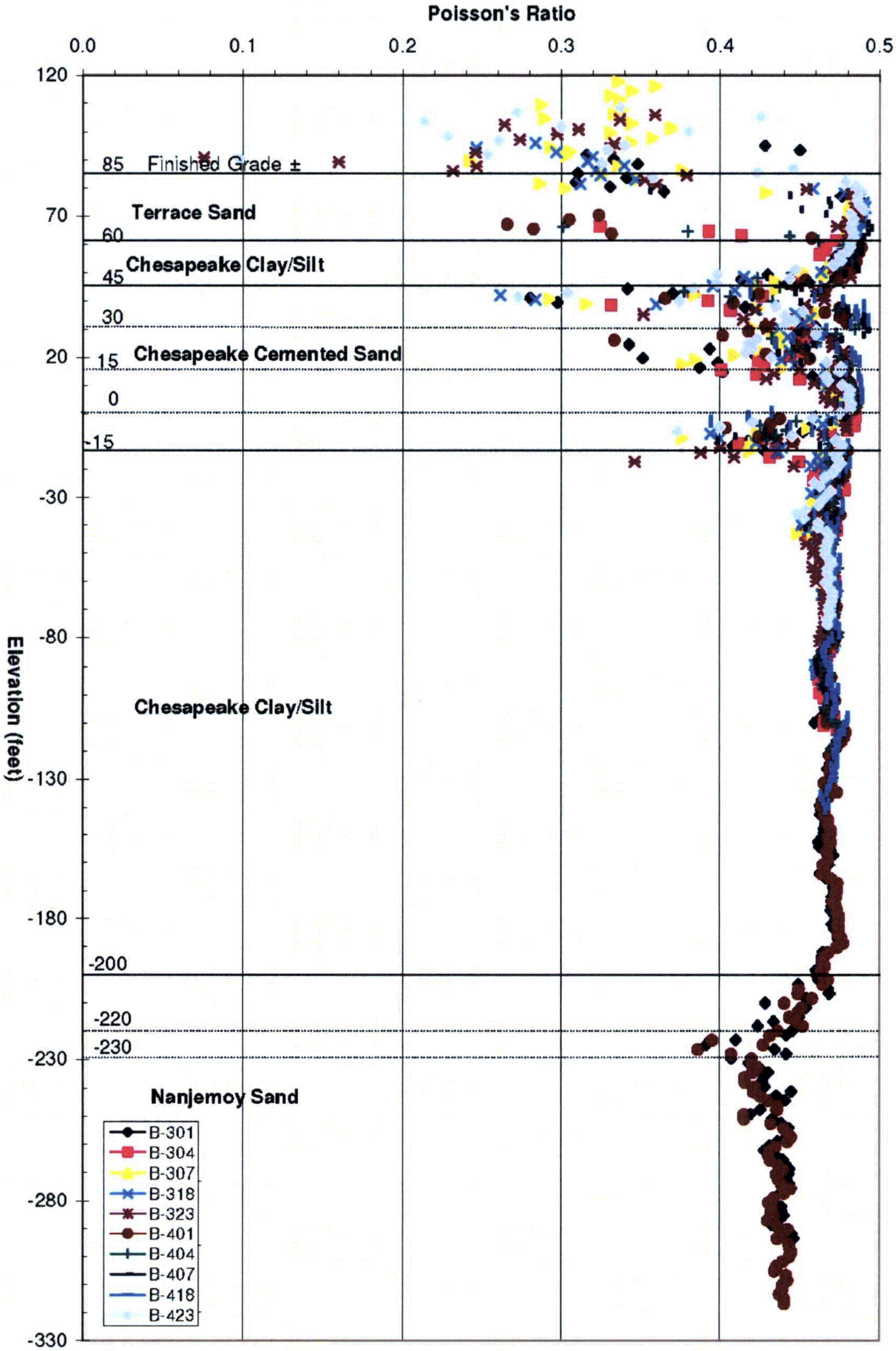


FIGURE 2.5.4-24 **Rev. 0**

POISSON'S RATIO FROM SUSPENSION P-S VELOCITY LOGGING

CCNPP UNIT 3 FSAR

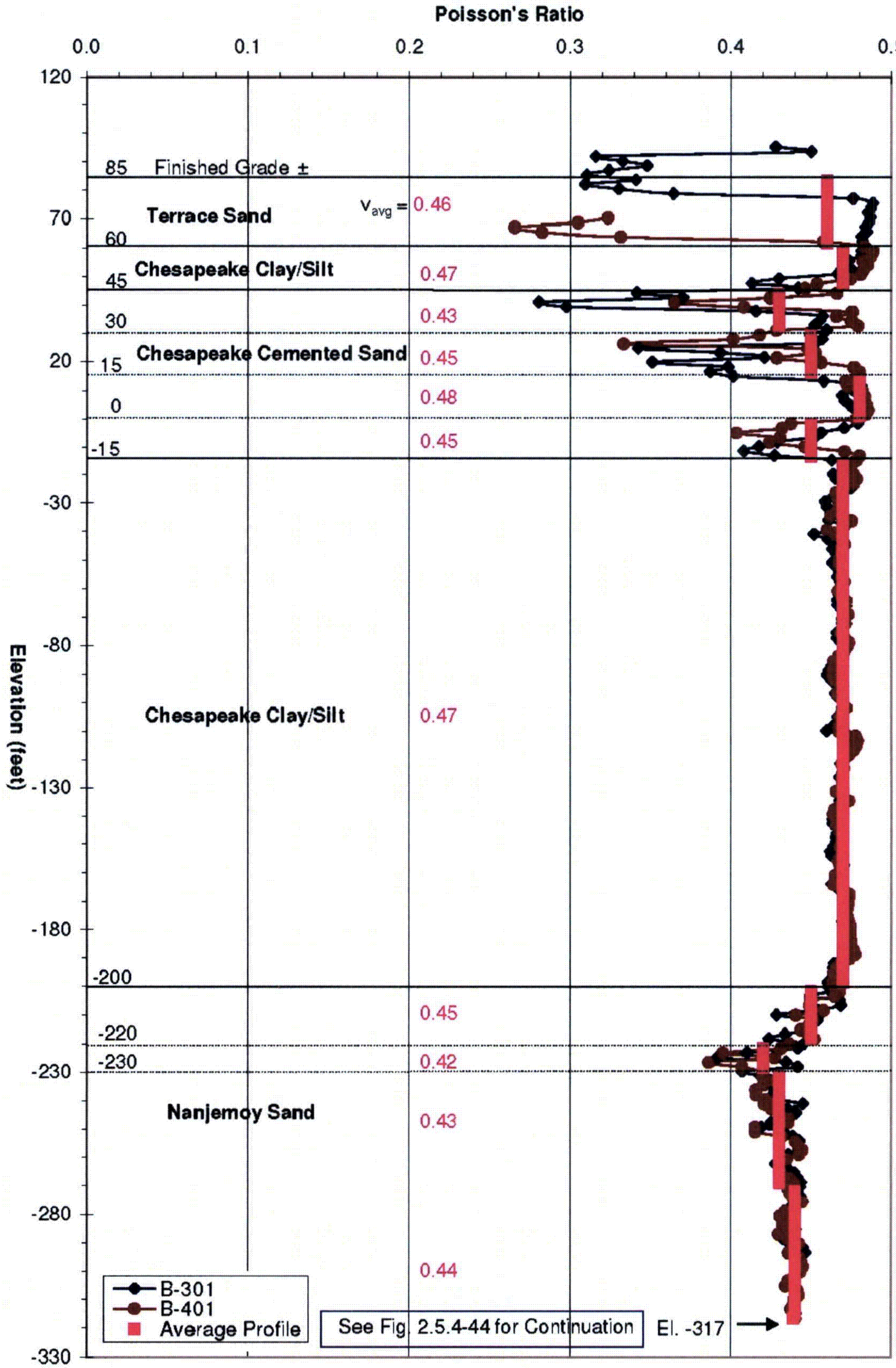


FIGURE 2.5.4-25 Rev. 0

AVERAGE POISSON'S RATIO FROM SUSPENSION P-S VELOCITY LOGGING

CCNPP UNIT 3 FSAR

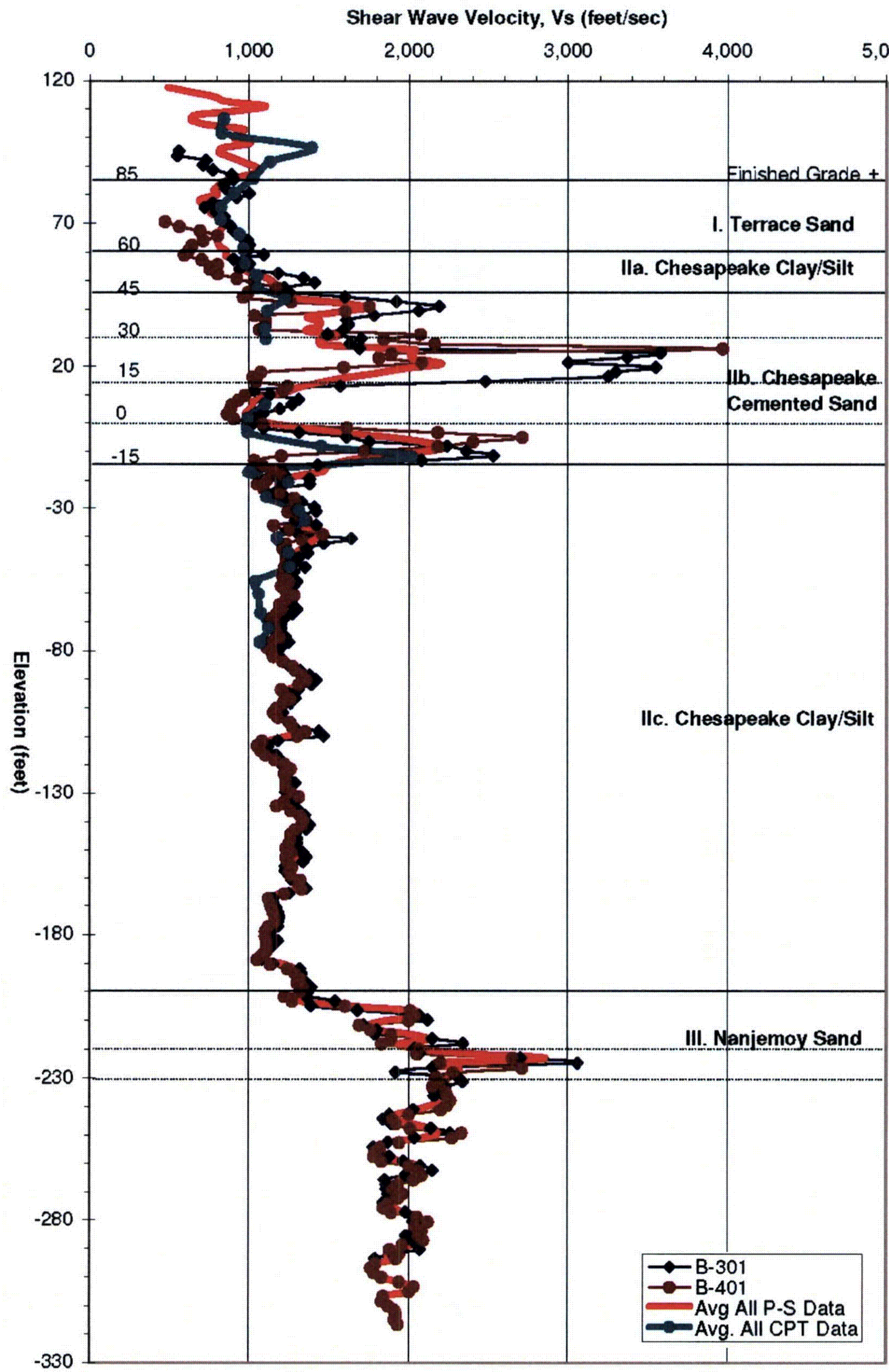


FIGURE 2.5.4-26 Rev. 0

COMPARISON OF Vs MEASUREMENTS FROM SUSPENSION P–S VELOCITY LOGGING AND CPT SEISMIC TESTS

CCNPP UNIT 3 FSAR

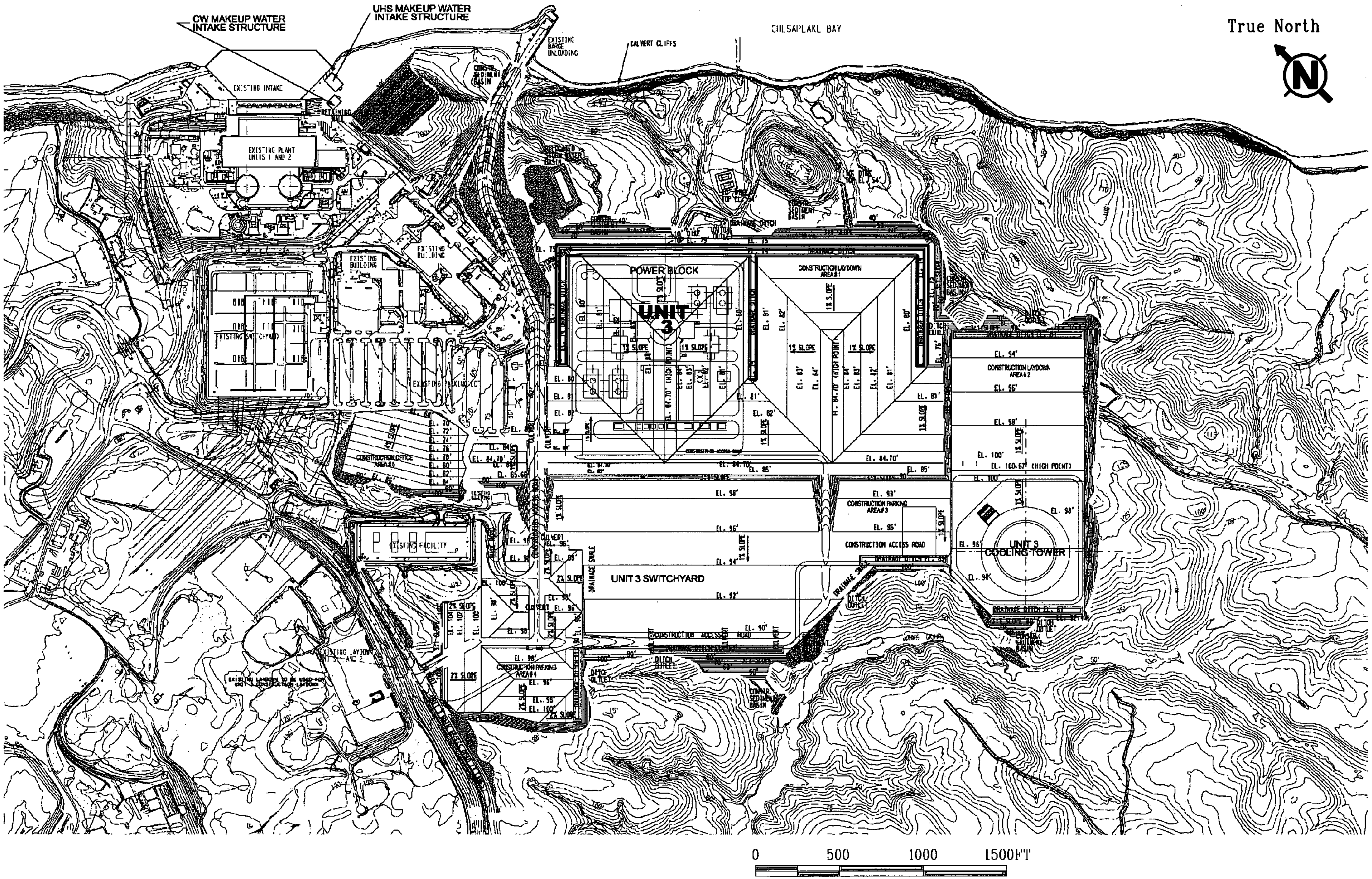
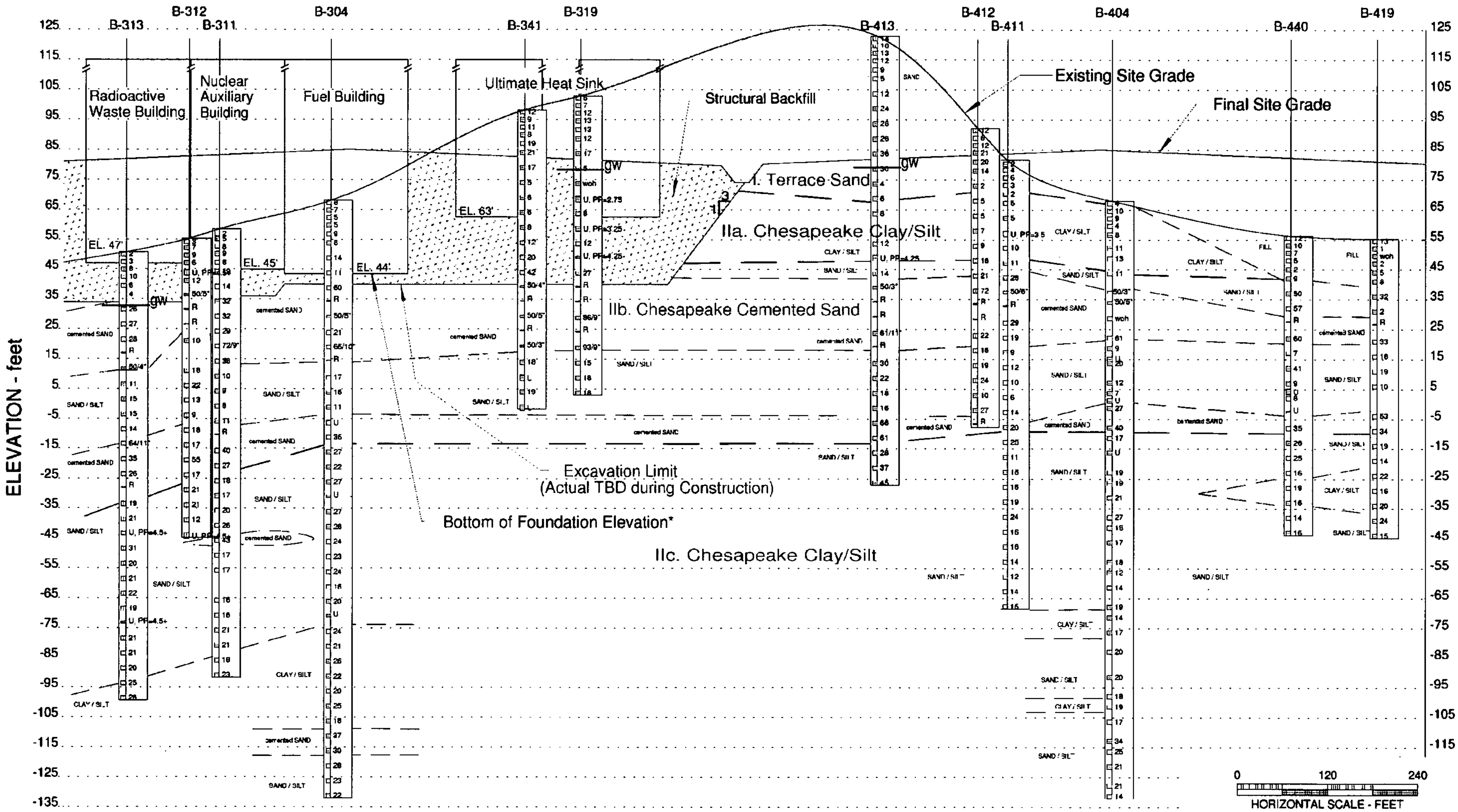


FIGURE 2.5.4-27 **Rev. 0**

SITE GRADING PLAN

CCNPP UNIT 3 FSAR

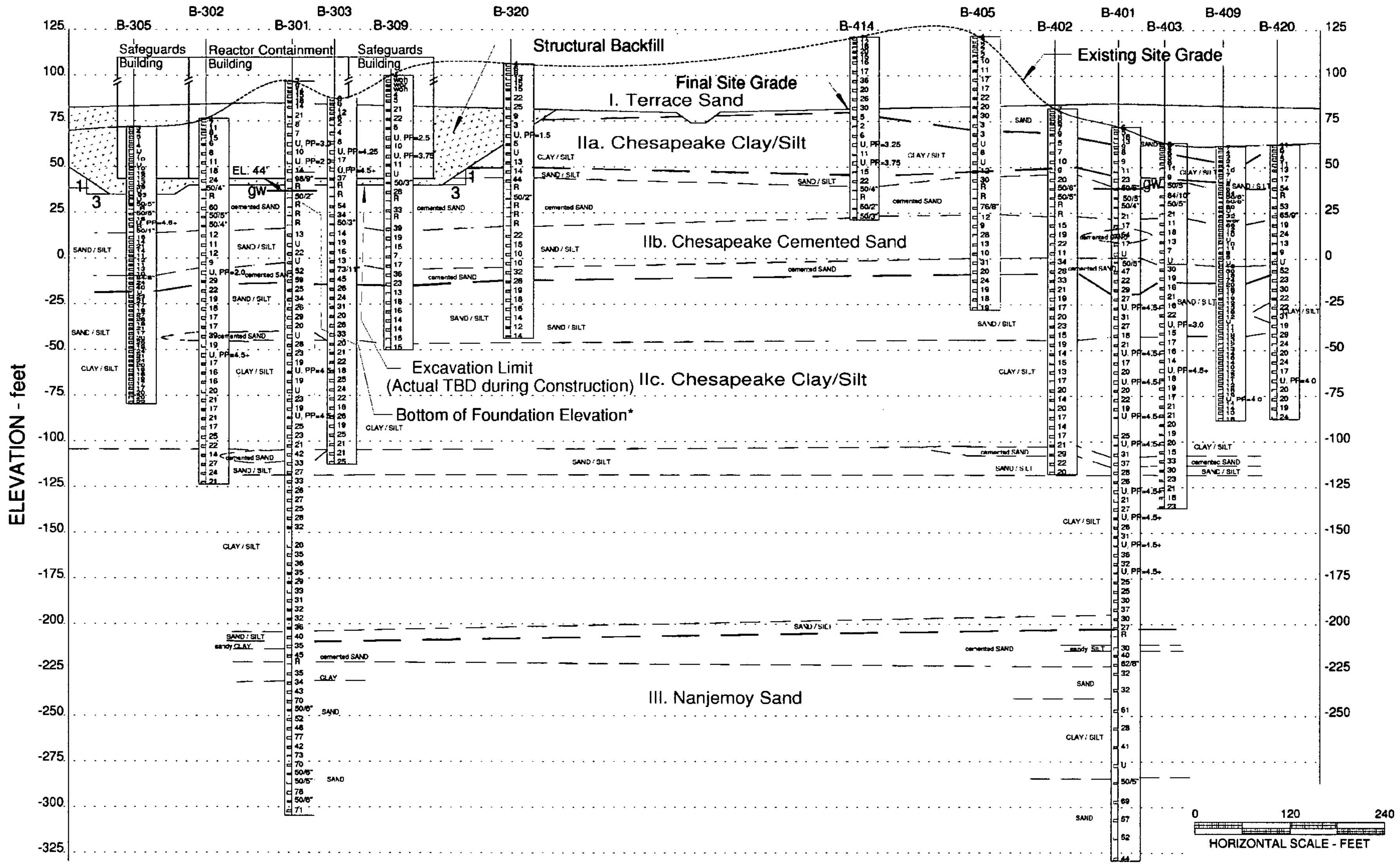


* preliminary, actual TBD during detailed design

FIGURE 2.5.4-28 Rev. 0

EXCAVATION PROFILE NS-1

CCNPP UNIT 3 FSAR

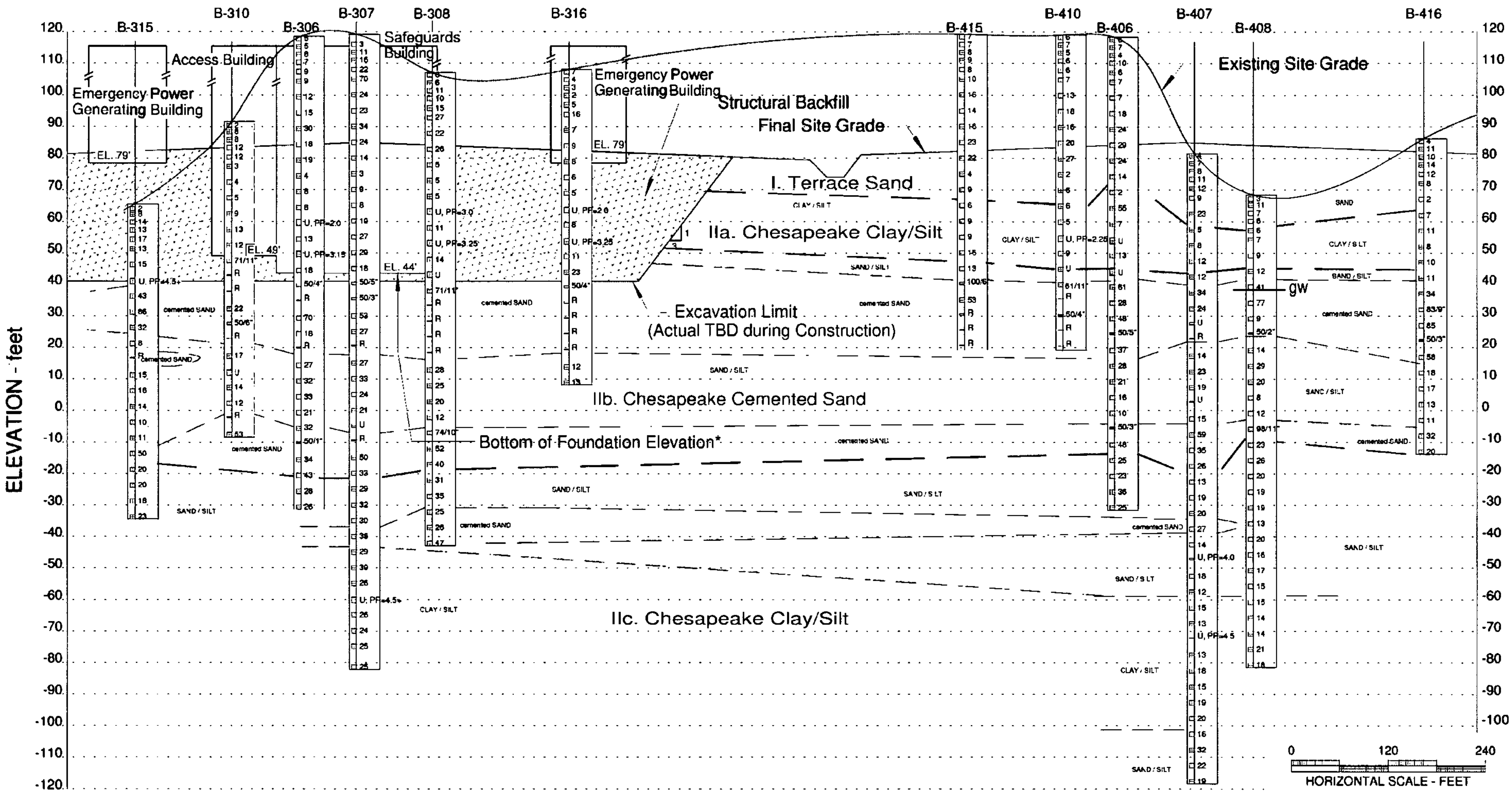


* preliminary, actual TBD during detailed design

FIGURE 2.5.4-29 Rev. 0

EXCAVATION PROFILE NS-2

CCNPP UNIT 3 FSAR

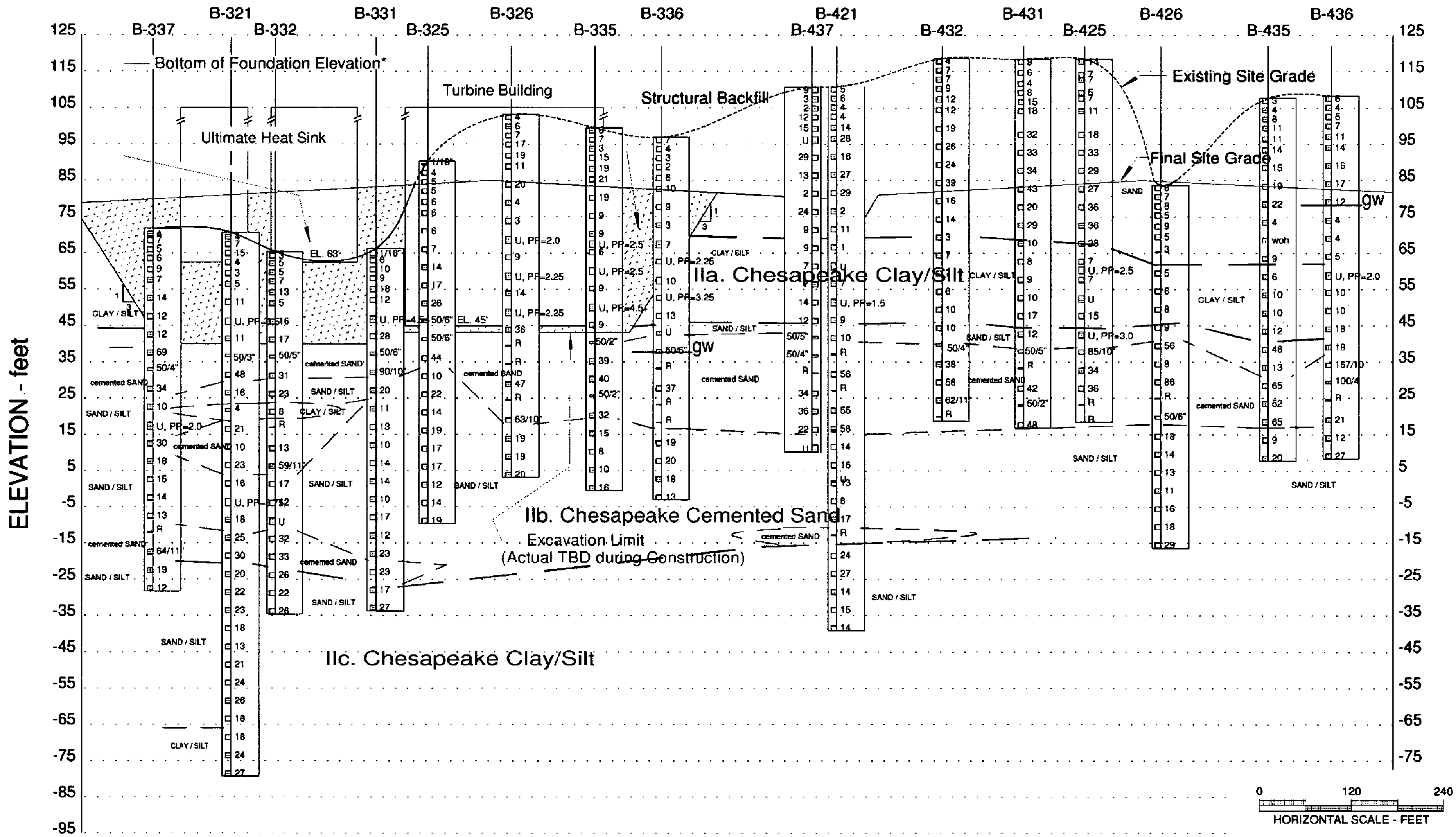


* preliminary, actual TBD during detailed design

FIGURE 2.5.4-30 Rev. 0

EXCAVATION PROFILE NS-3

CCNPP UNIT 3 FSAR

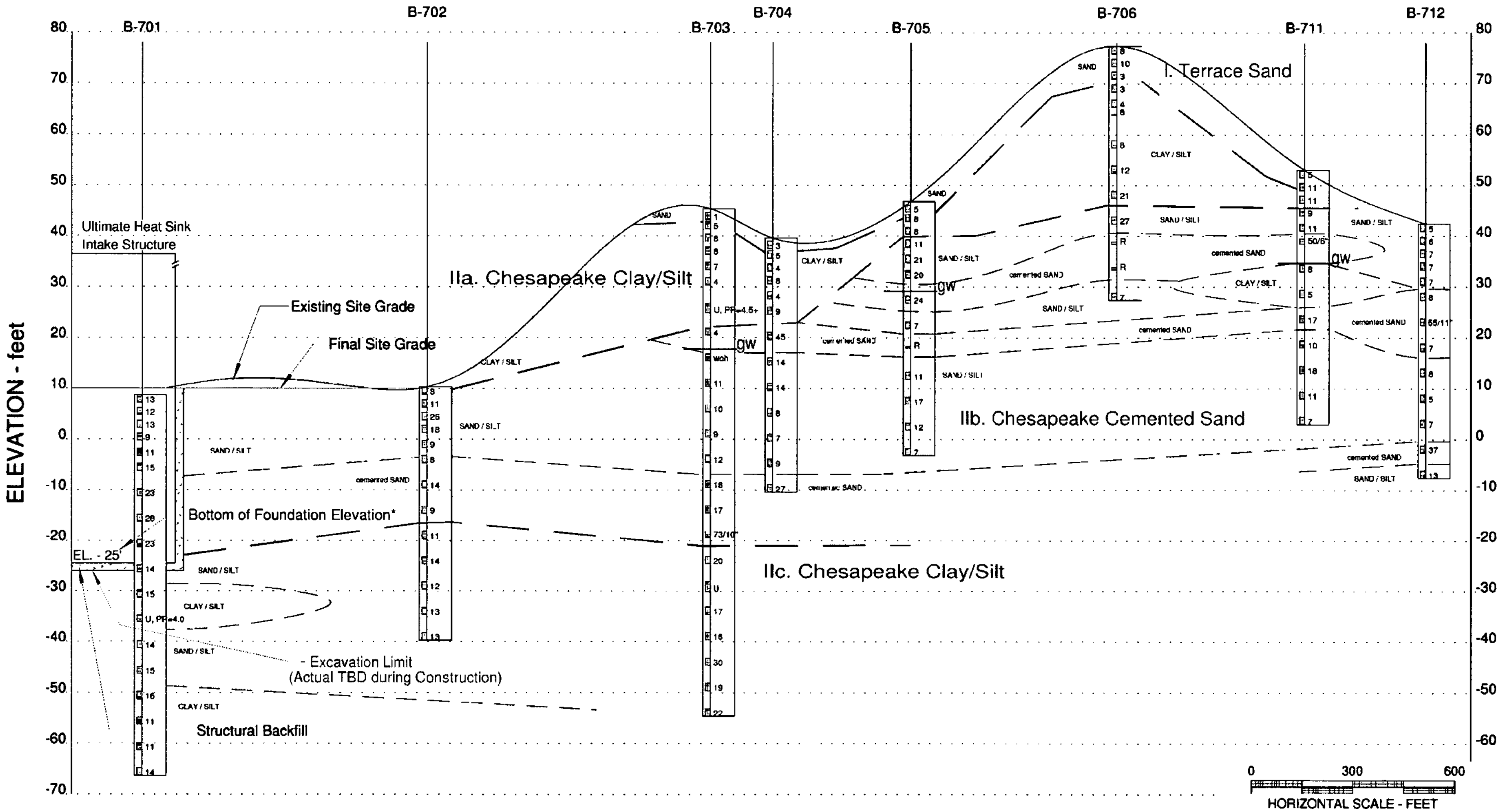


* preliminary, actual TBD during detailed design

FIGURE 2.5.4-31 **Rev. 0**

EXCAVATION PROFILE NS-4

CCNPP UNIT 3 FSAR

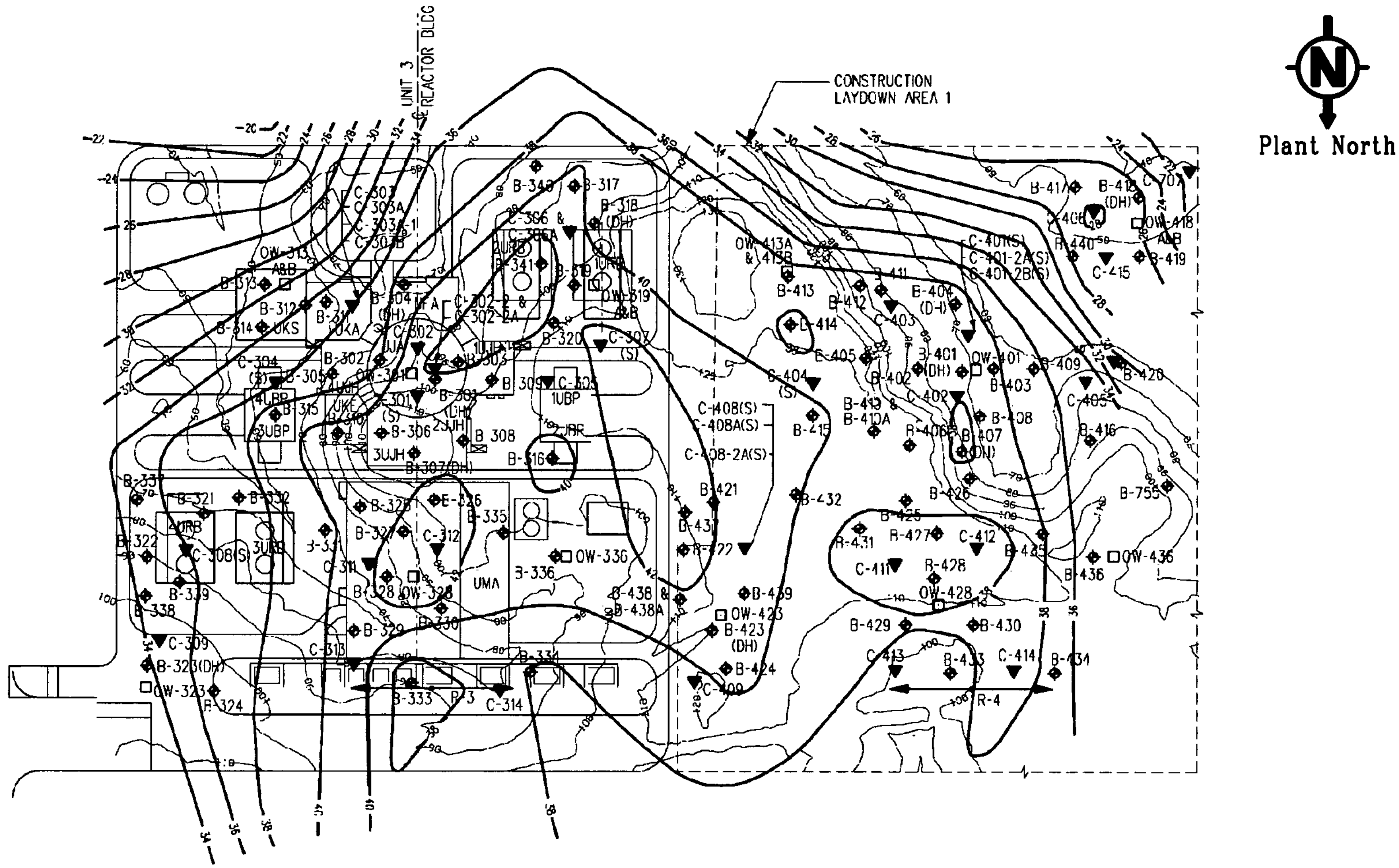


* preliminary, actual TBD during detailed design

FIGURE 2.5.4-32 **Rev. 0**

EXCAVATION PROFILE IDP1

CCNPP UNIT 3 FSAR



NOTES:

1. CONTOURS OF TOP OF STRATUM IIb CEMENTED SAND ARE APPROXIMATE.
2. FINAL ELEVATIONS OF TOP OF CEMENTED SAND TO BE CONFIRMED DURING CONSTRUCTION.

LEGEND

1-4	UBP	EMERGENCY POWER GENERATING BUILDING
	UFA	FUEL BUILDING
	UJA	REACTOR BUILDING
1-4	UJH	SAFEGUARD BUILDING
	UKA	NUCLEAR AUXILIARY BUILDING
	UKE	ACCESS BUILDING
	UKS	RADIOACTIVE WASTE BUILDING
	UMA	TURBINE BUILDING
1-4	URB	ULTIMATE HEAT SINK

- ◆ B-302 BORING NUMBER
- ◆ B-307(DH) BORING WITH DOWNHOLE GEOPHYSICAL MEASUREMENTS
- ▼ C-303 CPT NUMBER
- ▼ C-307(S) CPT WITH SEISMIC TESTING
- □ OW-428 OBSERVATION WELL (A&B INDICATE WELL PAIR)
- ←→ R-1 FIELD ELECTRICAL RESISTIVITY TEST
- —40— STRATUM IIb ELEVATION CONTOUR

0 200 400 600FT

FIGURE 2.5.4-33 **Rev. 0**

ELEVATION CONTOURS OF TOP OF STRATUM IIb CEMENTED SAND

CCNPP UNIT 3 FSAR

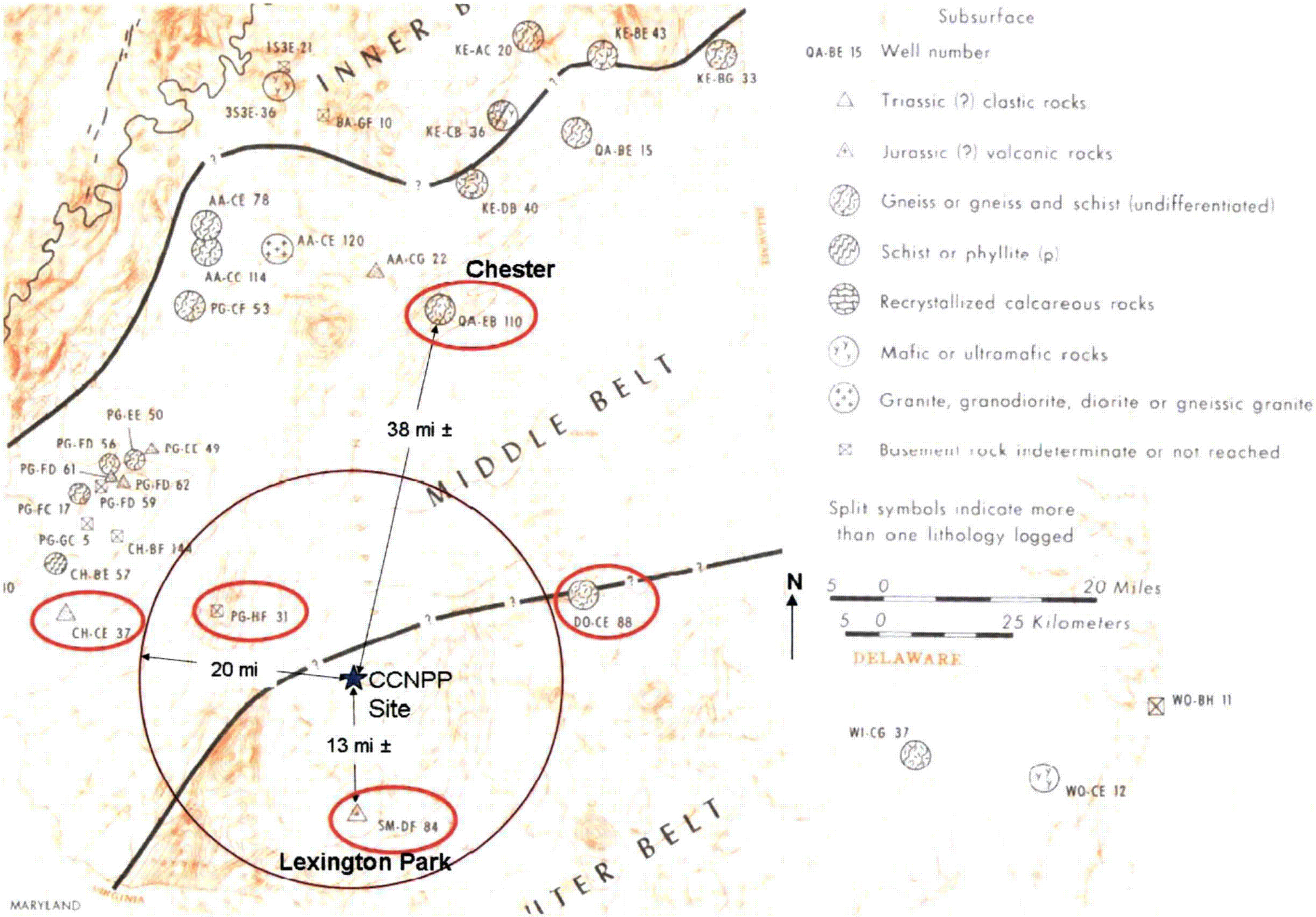


FIGURE 2.5.4-34 **Rev. 0**

PROXIMITY OF CHESTER AND LEXINGTON PARK SITES TO CCNPP

CCNPP UNIT 3 FSAR

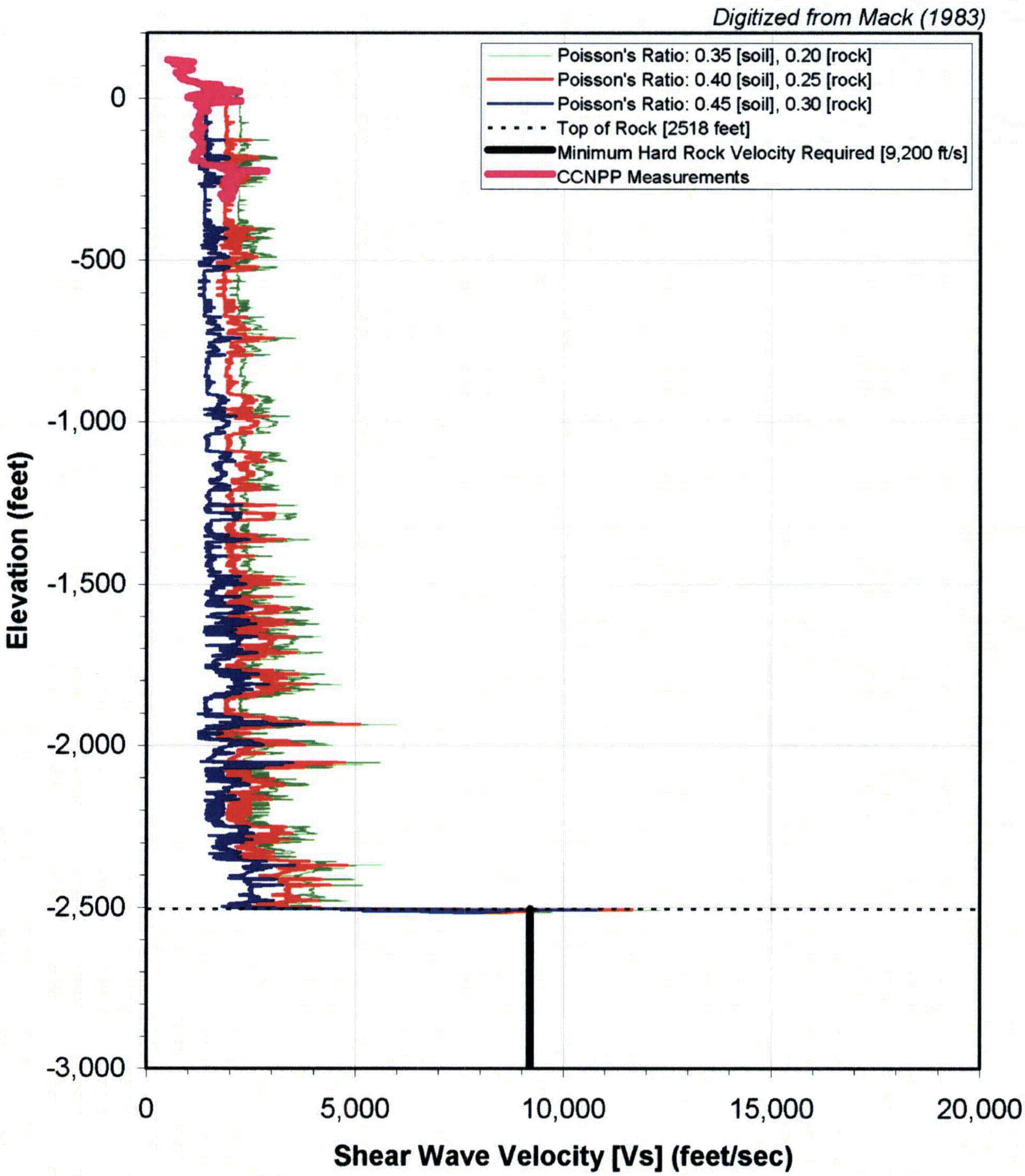


FIGURE 2.5.4-35 **Rev. 0**

Vs LOG BASED ON CHESTER (KENT ISLAND) MEASUREMENTS

CCNPP UNIT 3 FSAR

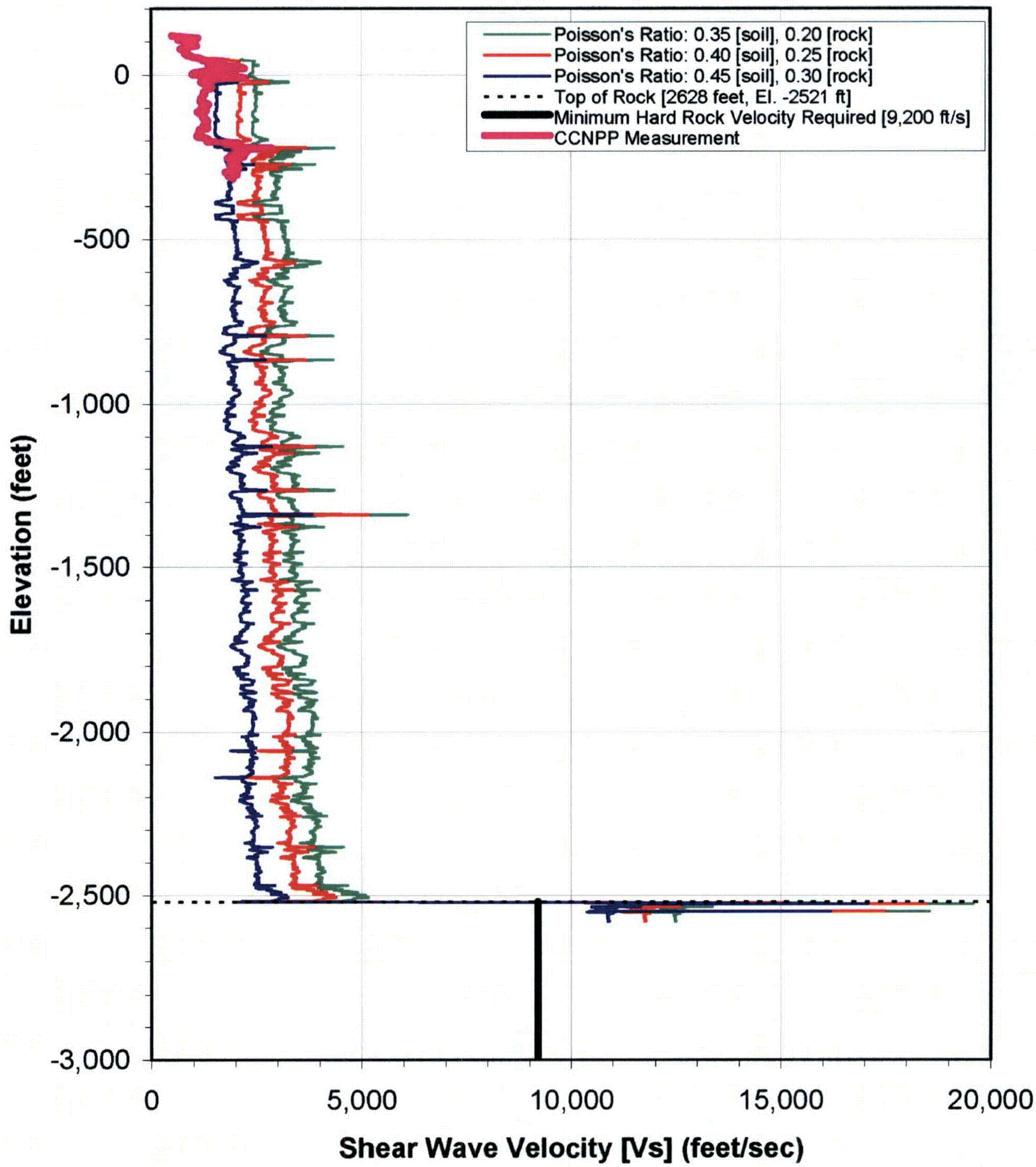


FIGURE 2.5.4-36 **Rev. 0**

Vs LOG BASED ON LEXINGTON PARK MEASUREMENTS

CCNPP UNIT 3 FSAR

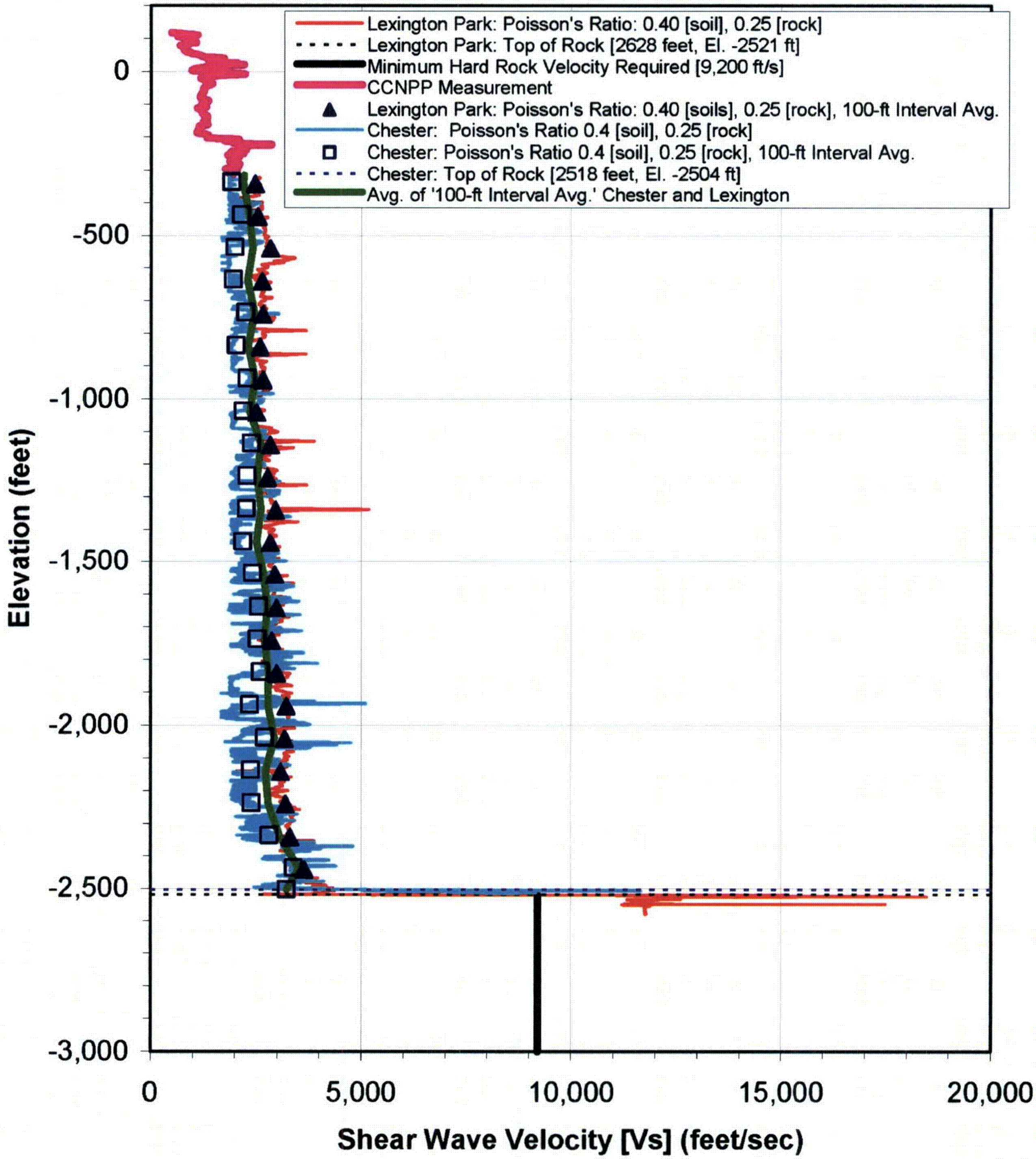


FIGURE 2.5.4-37 Rev. 0

SMOOTHED AND AVERAGED V_S LOG FOR CHESTER AND LEXINGTON PARK MEASUREMENTS

CCNPP UNIT 3 FSAR

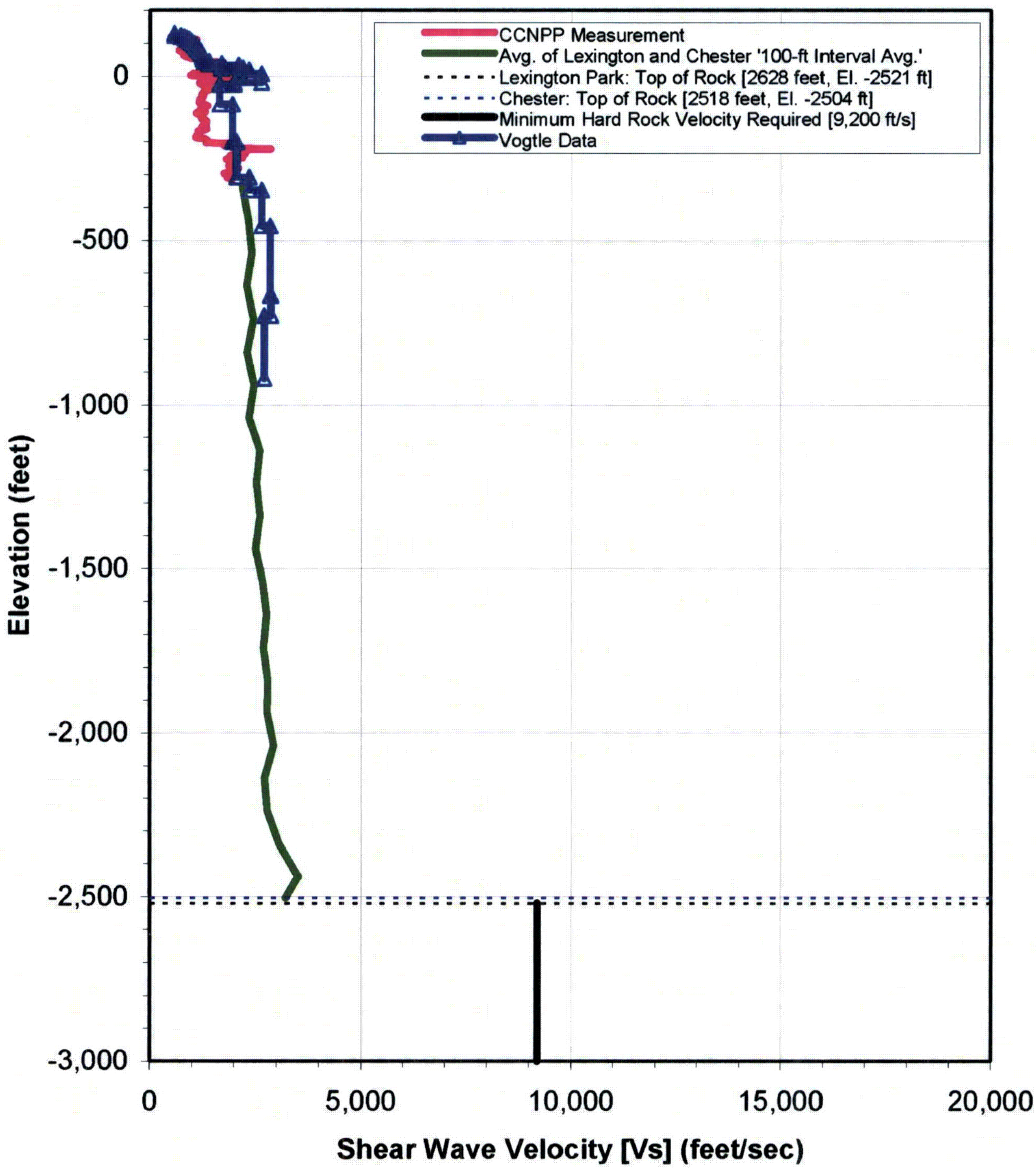


FIGURE 2.5.4-38 Rev. 0

COMPARISON OF AVERAGE V_S FOR CHESTER, LEXINGTON PARK, MARYLAND AND DEEP MEASUREMENTS IN COASTAL PLAIN SOILS FROM REF. 2.5.4.59 (VOGTLE)

CCNPP UNIT 3 FSAR

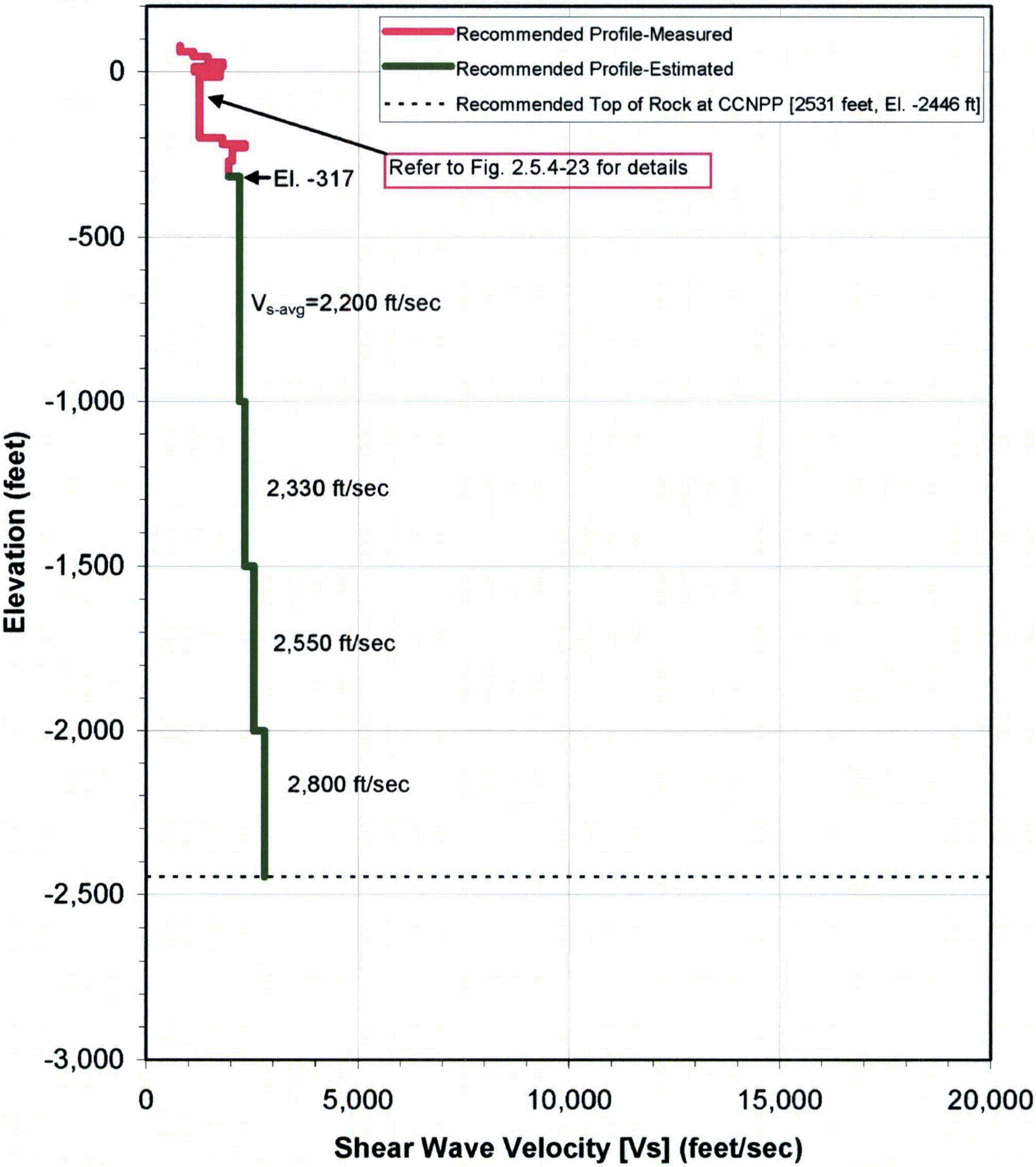


FIGURE 2.5.4-39 **Rev. 0**

RECOMMENDED Vs SOIL PROFILE AT CCNPP SITE

CCNPP UNIT 3 FSAR

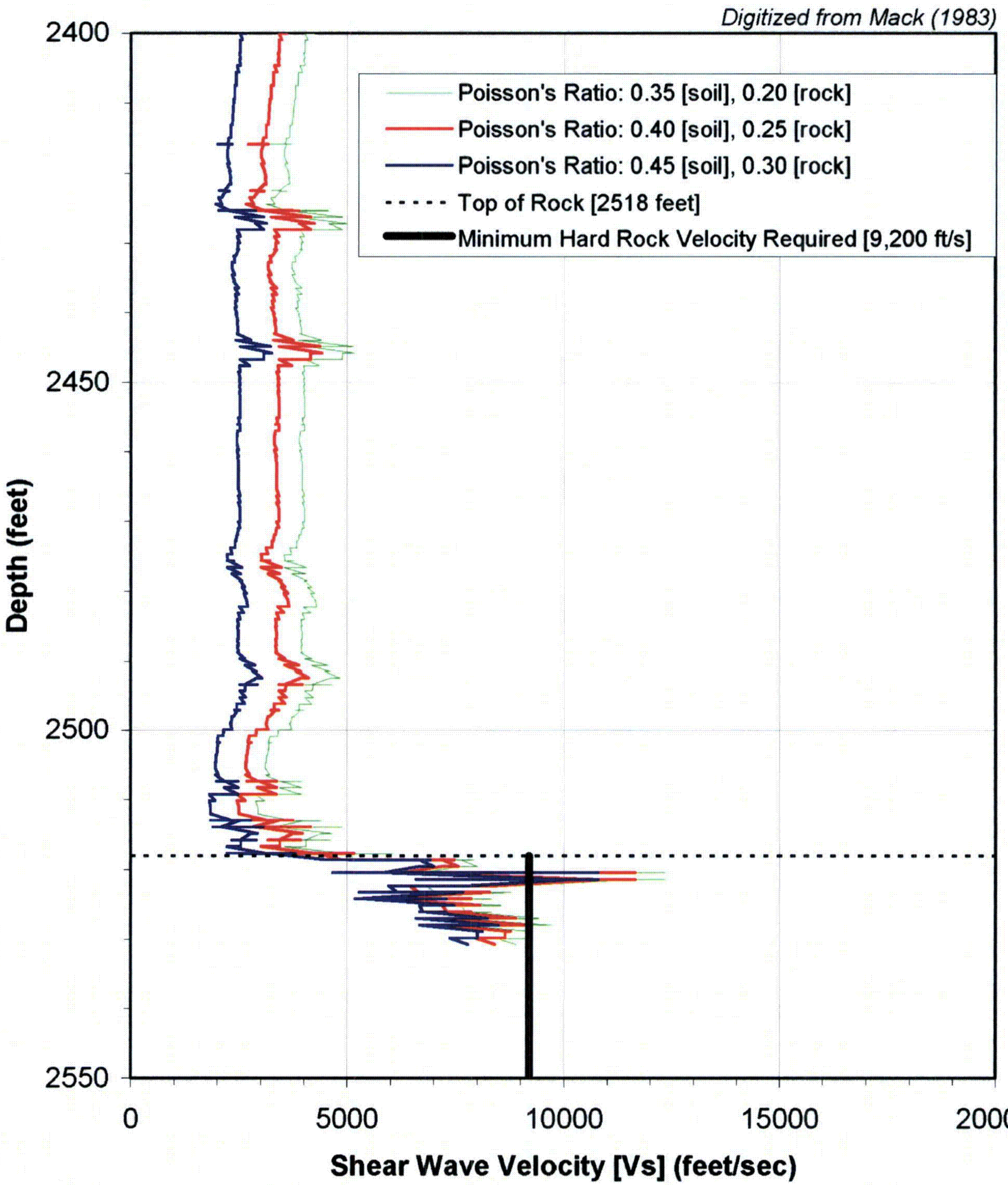


FIGURE 2.5.4-40 **Rev. 0**

BEDROCK Vs LOG FOR CHESTER (KENT ISLAND), MARYLAND

CCNPP UNIT 3 FSAR

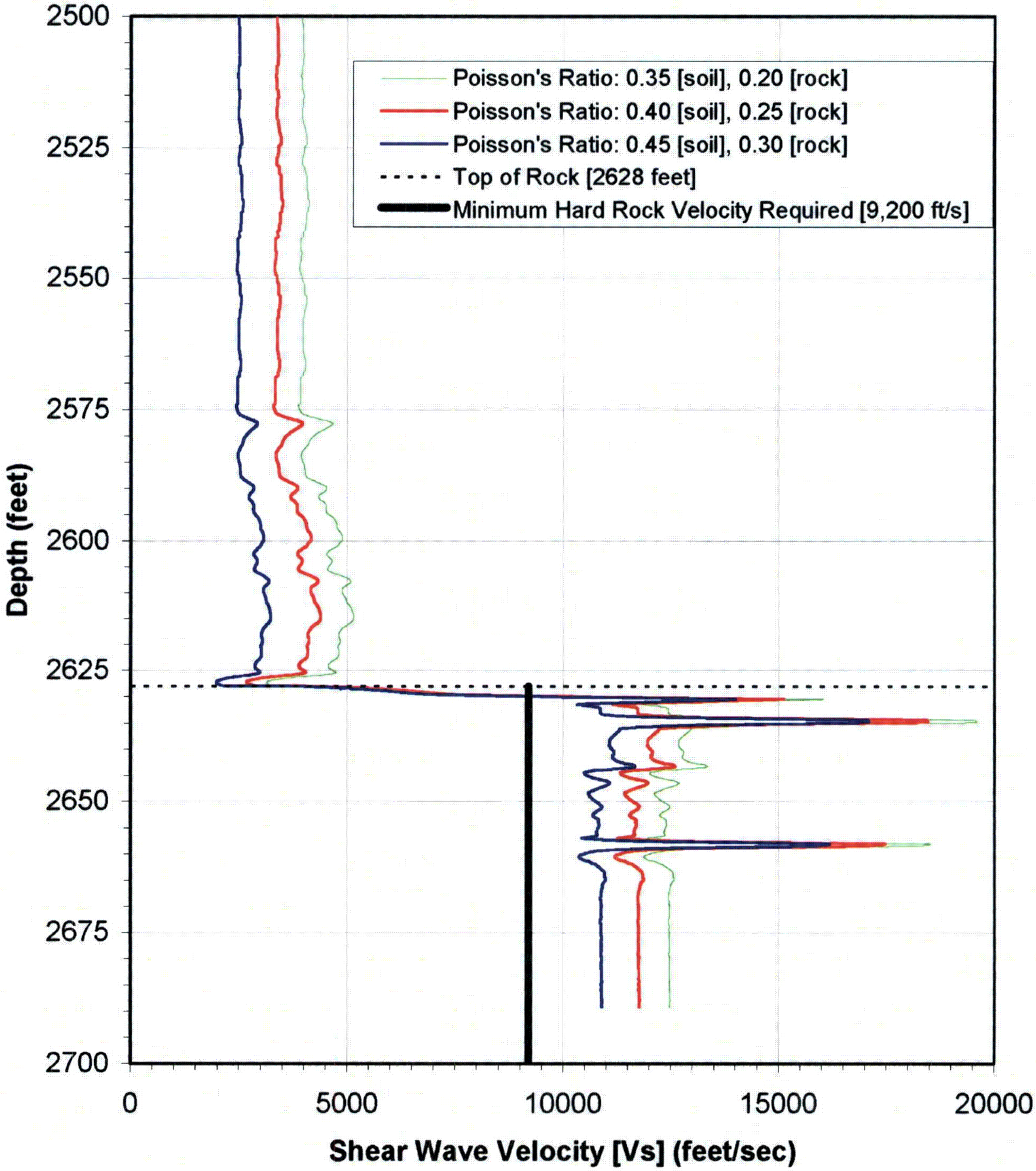


FIGURE 2.5.4-41 **Rev. 0**

BEDROCK Vs LOG FOR LEXINGTON PARK, MARYLAND

CCNPP UNIT 3 FSAR

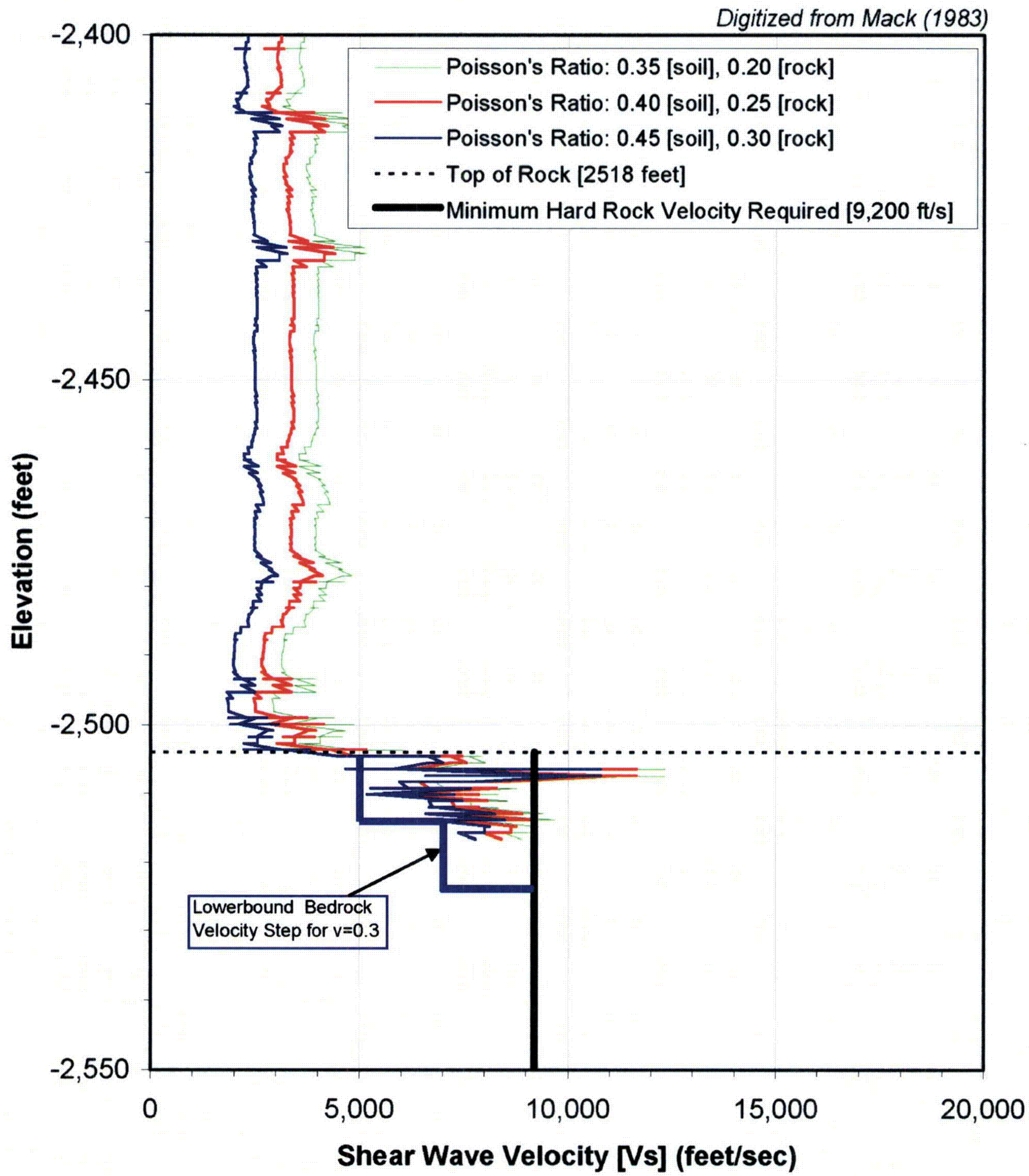


FIGURE 2.5.4-42 **Rev. 0**

INTERPRETATION OF BEDROCK VELOCITY GRADIENT FOR CHESTER MEASUREMENT

CCNPP UNIT 3 FSAR

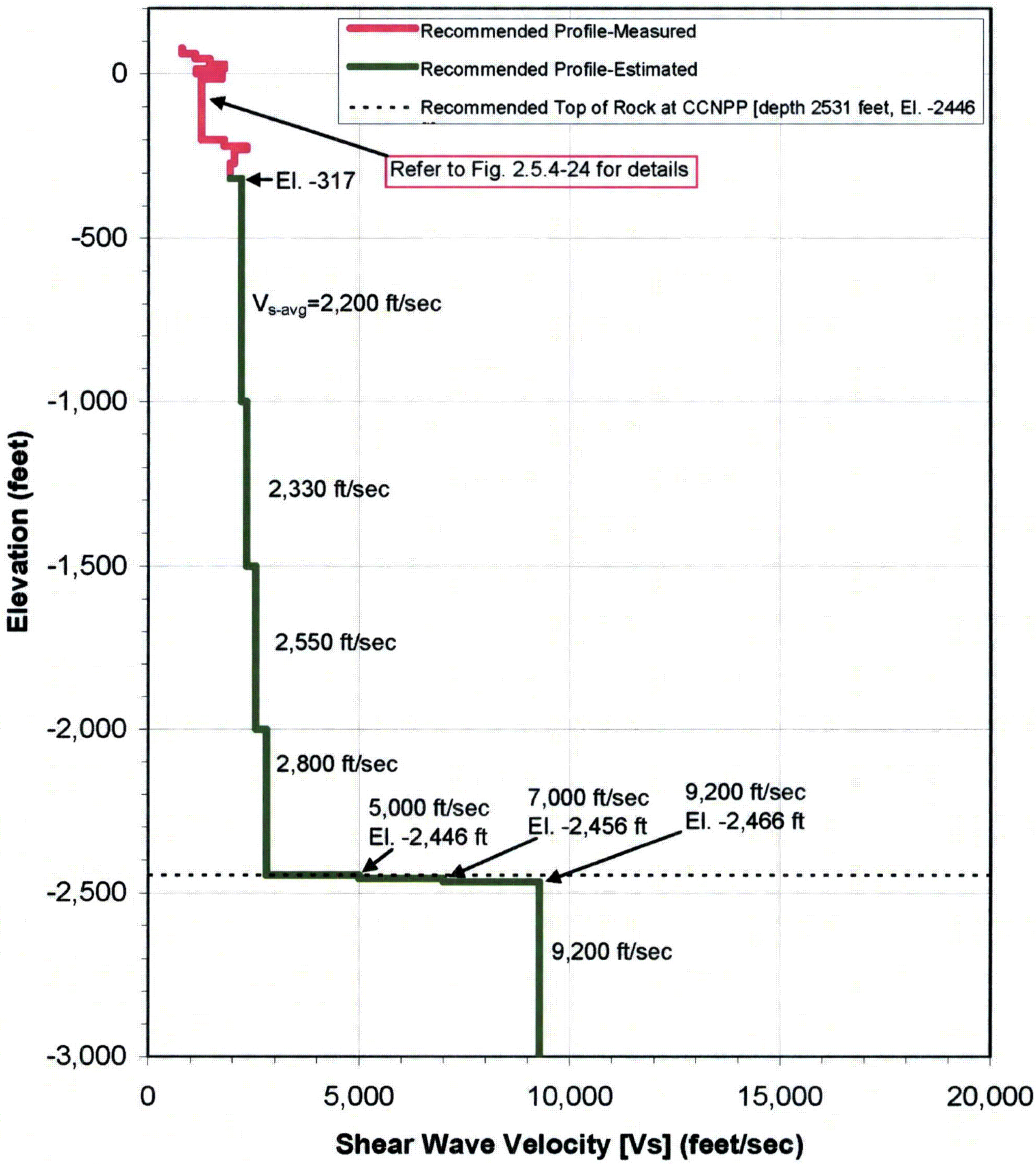


FIGURE 2.5.4-43 **Rev. 0**

RECOMMENDED V_s PROFILE FOR THE CCNPP SITE

CCNPP UNIT 3 FSAR

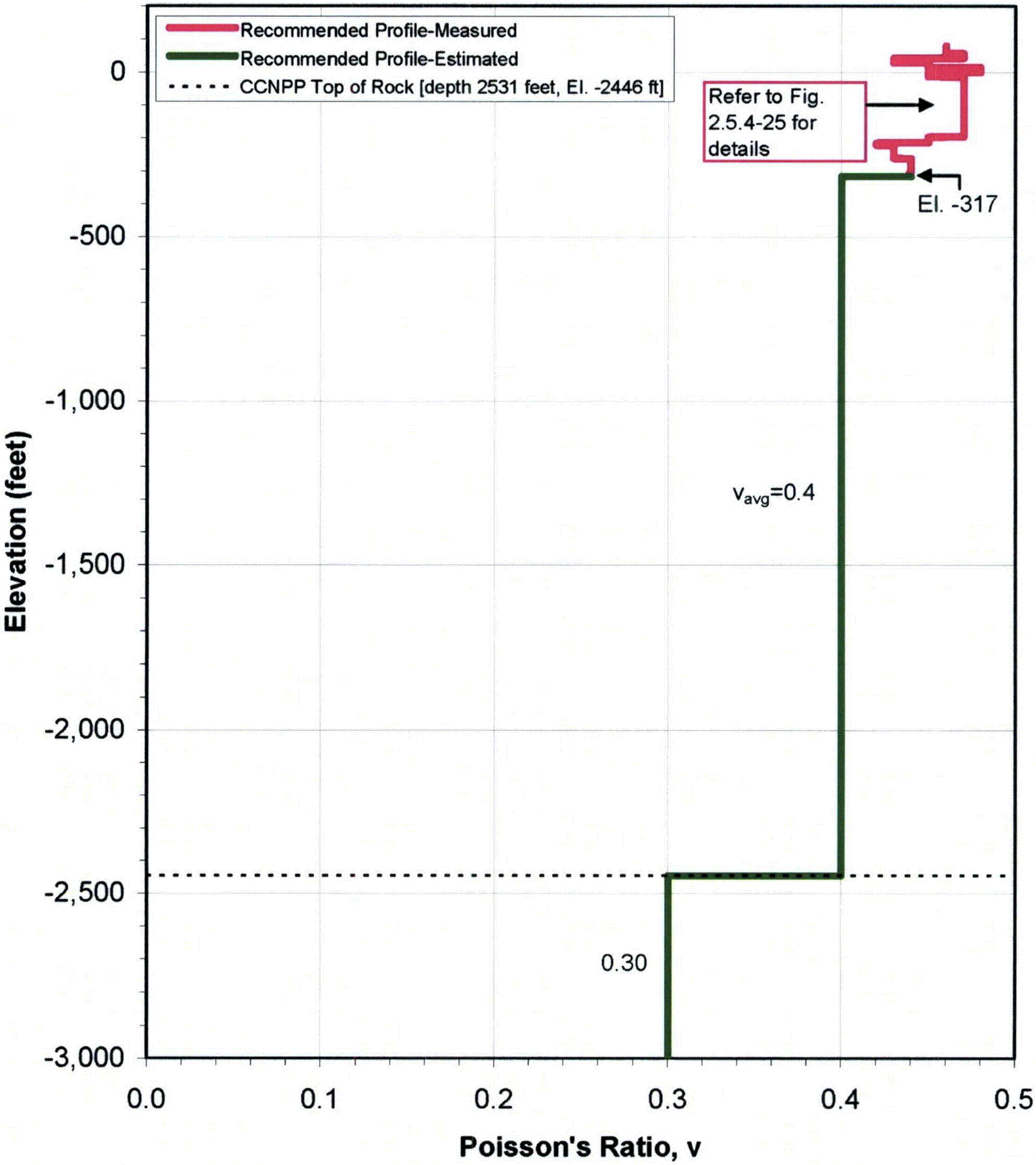
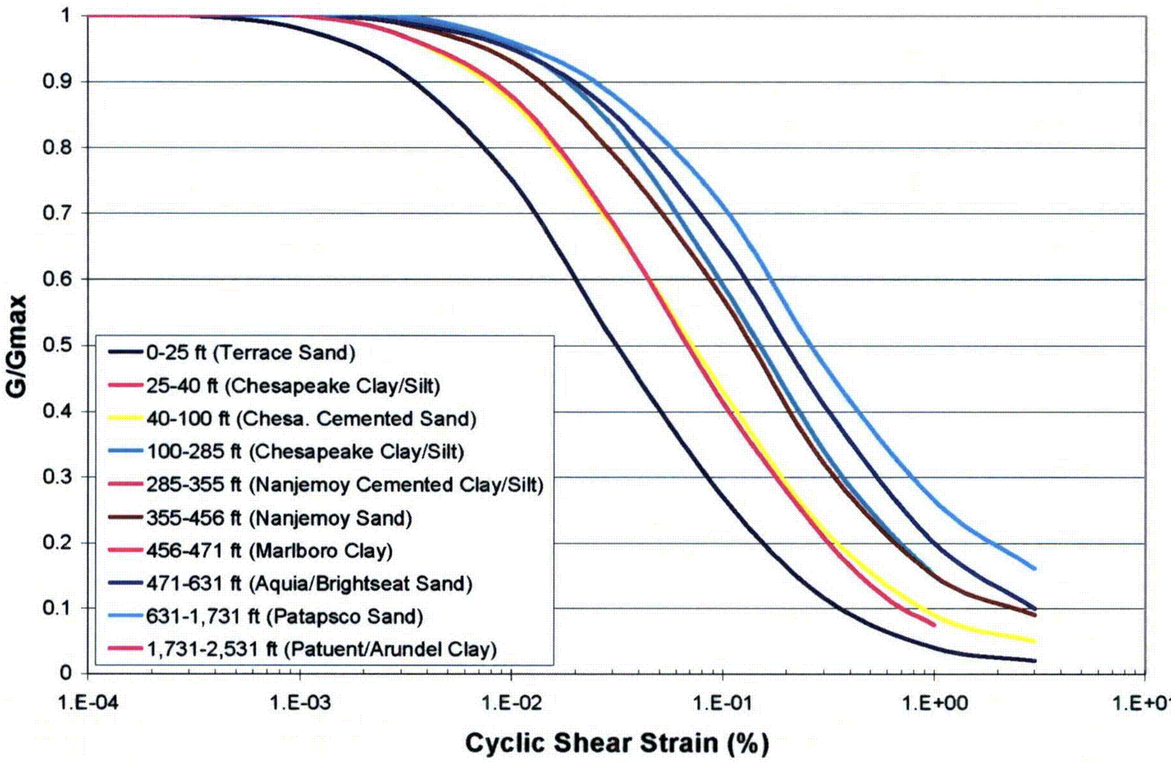


FIGURE 2.5.4-44 **Rev. 0**

RECOMMENDED POISSON'S RATIO PROFILE FOR THE CCNPP SITE

CCNPP UNIT 3 FSAR



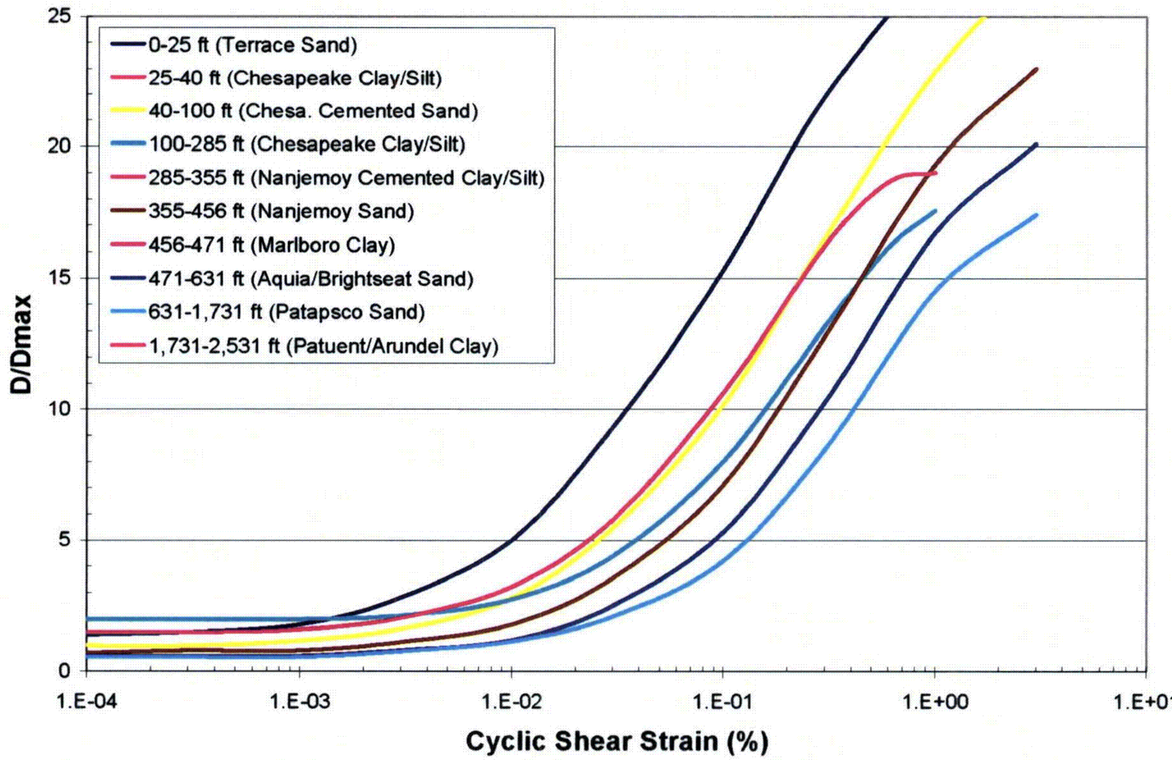


FIGURE 2.5.4-45 Rev. 0

RECOMMENDED MODULUS AND DAMPING RATIO CURVES FOR THE CCNPP SITE

CCNPP UNIT 3 FSAR

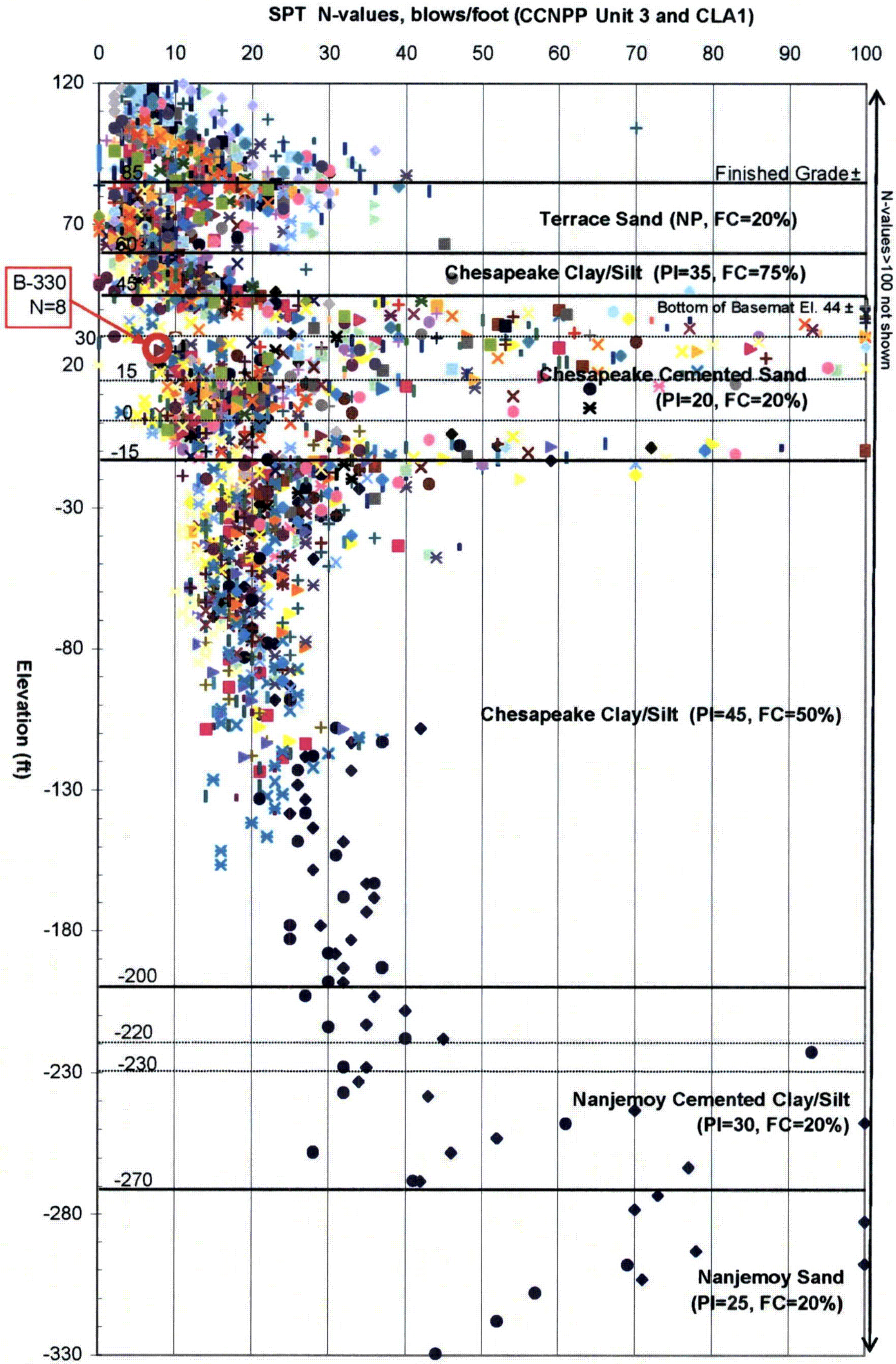


FIGURE 2.5.4-46 **Rev. 0**

MEASURED SPT N-VALUES (UNCORRECTED)

CCNPP UNIT 3 FSAR

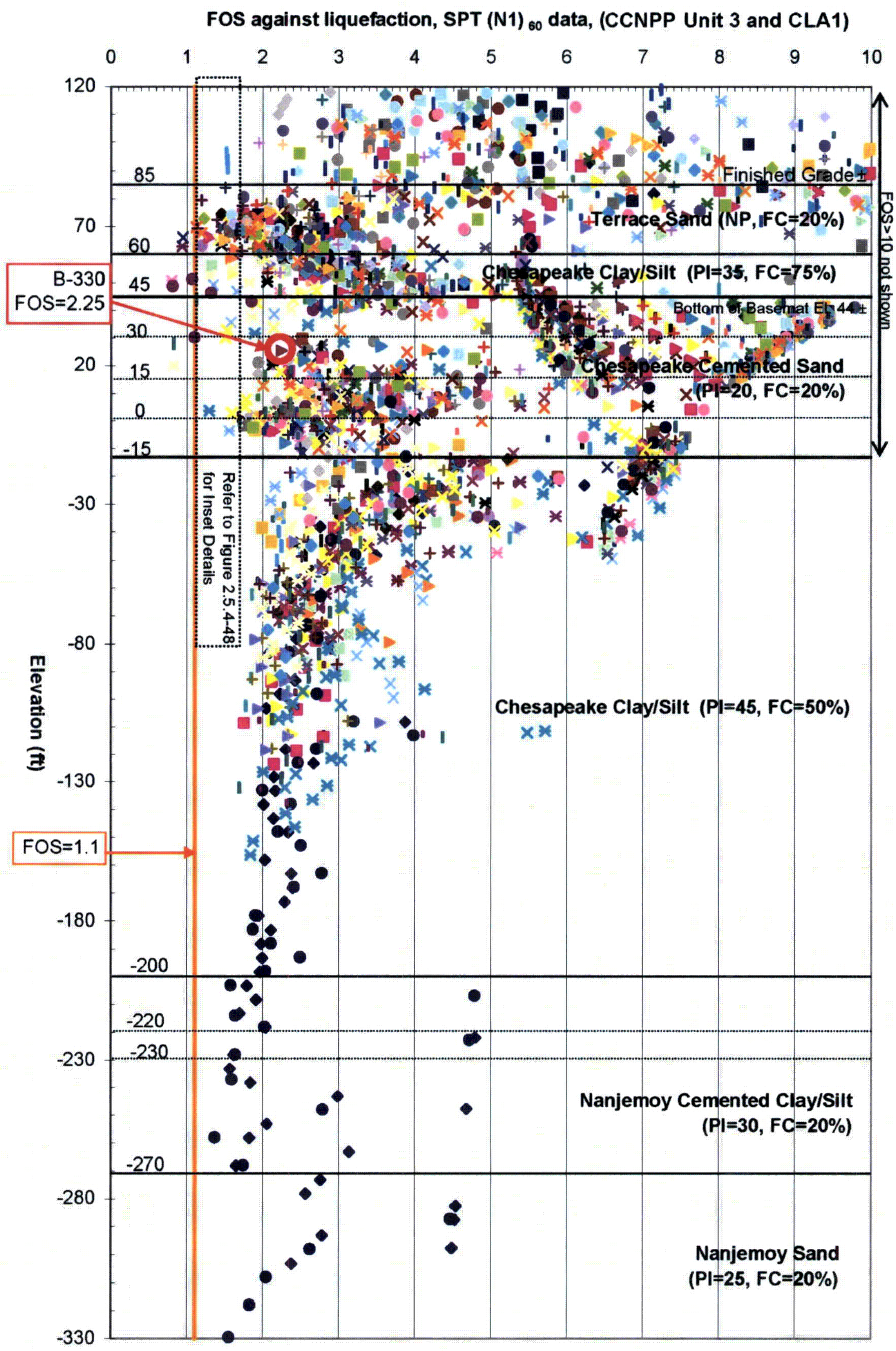


FIGURE 2.5.4-47 **Rev. 0**

CALCULATED FOS BASED ON SPT N-VALUES

CCNPP UNIT 3 FSAR

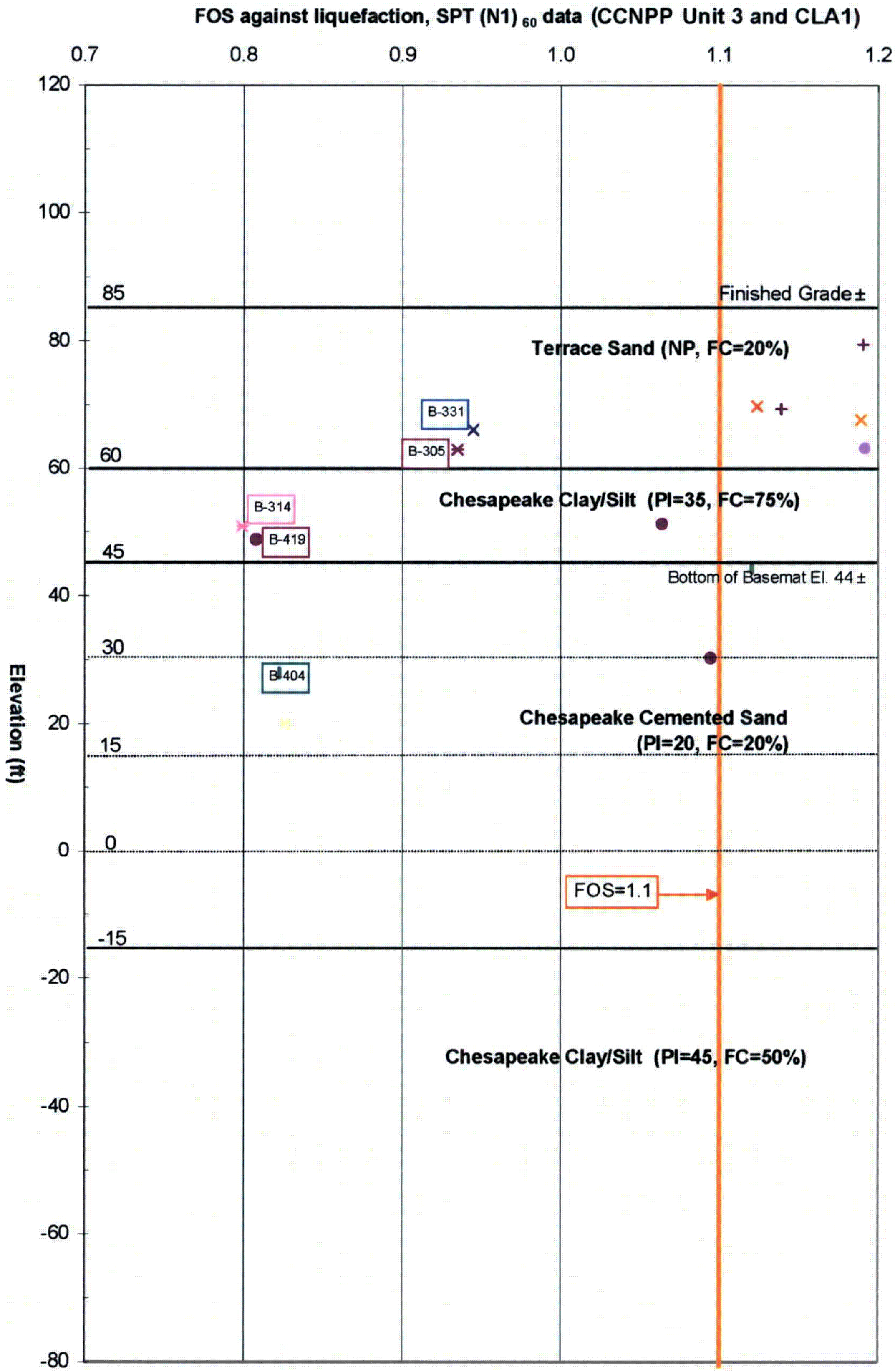


FIGURE 2.5.4-48 **Rev. 0**

CALCULATED FOS <1.1 BASED ON SPT N-VALUES

CCNPP UNIT 3 FSAR

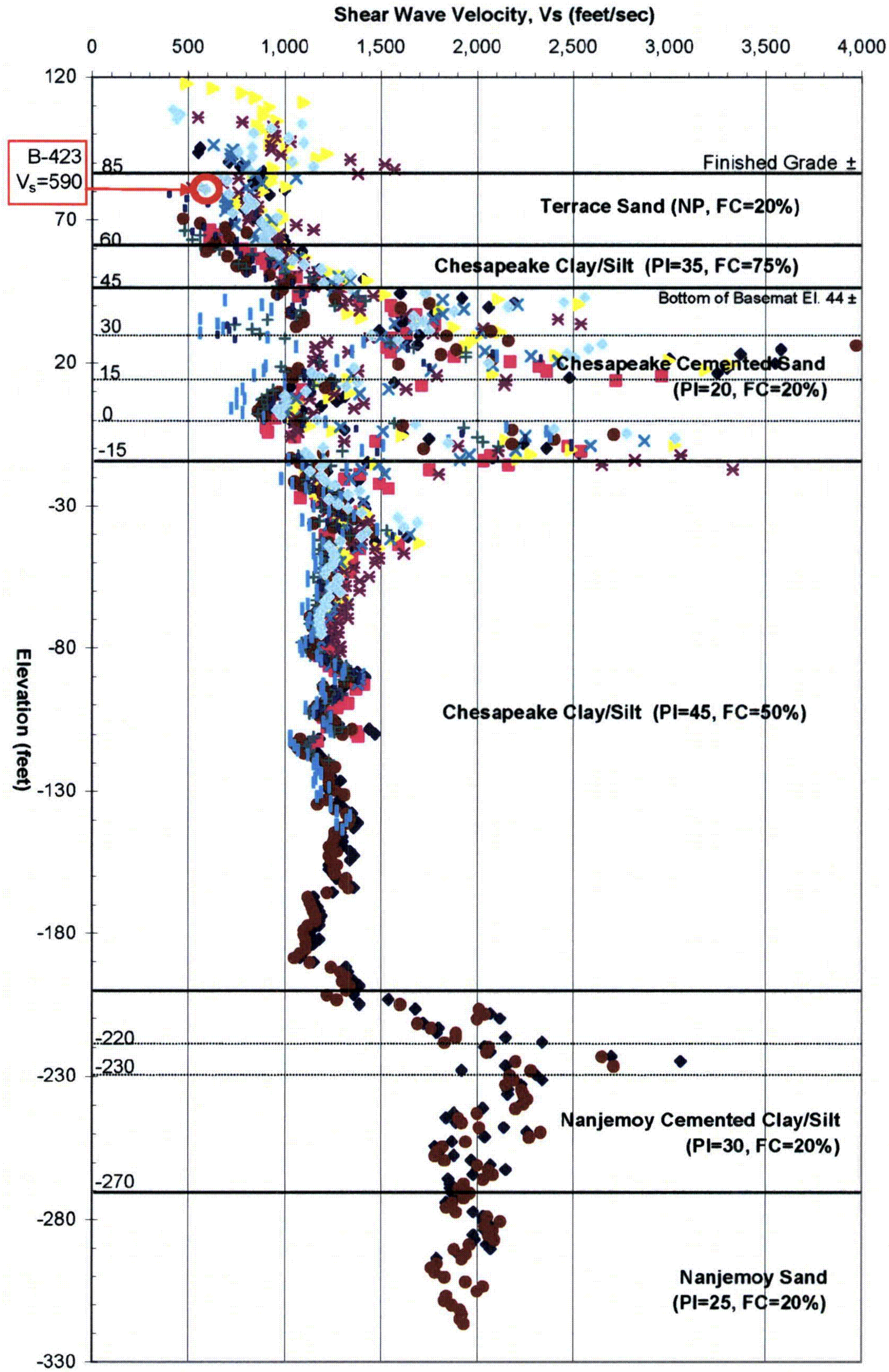


FIGURE 2.5.4-49 **Rev. 0**

MEASURED SUSPENSION P-S VELOCITY LOGGING Vs DATA (UNCORRECTED)

CCNPP UNIT 3 FSAR

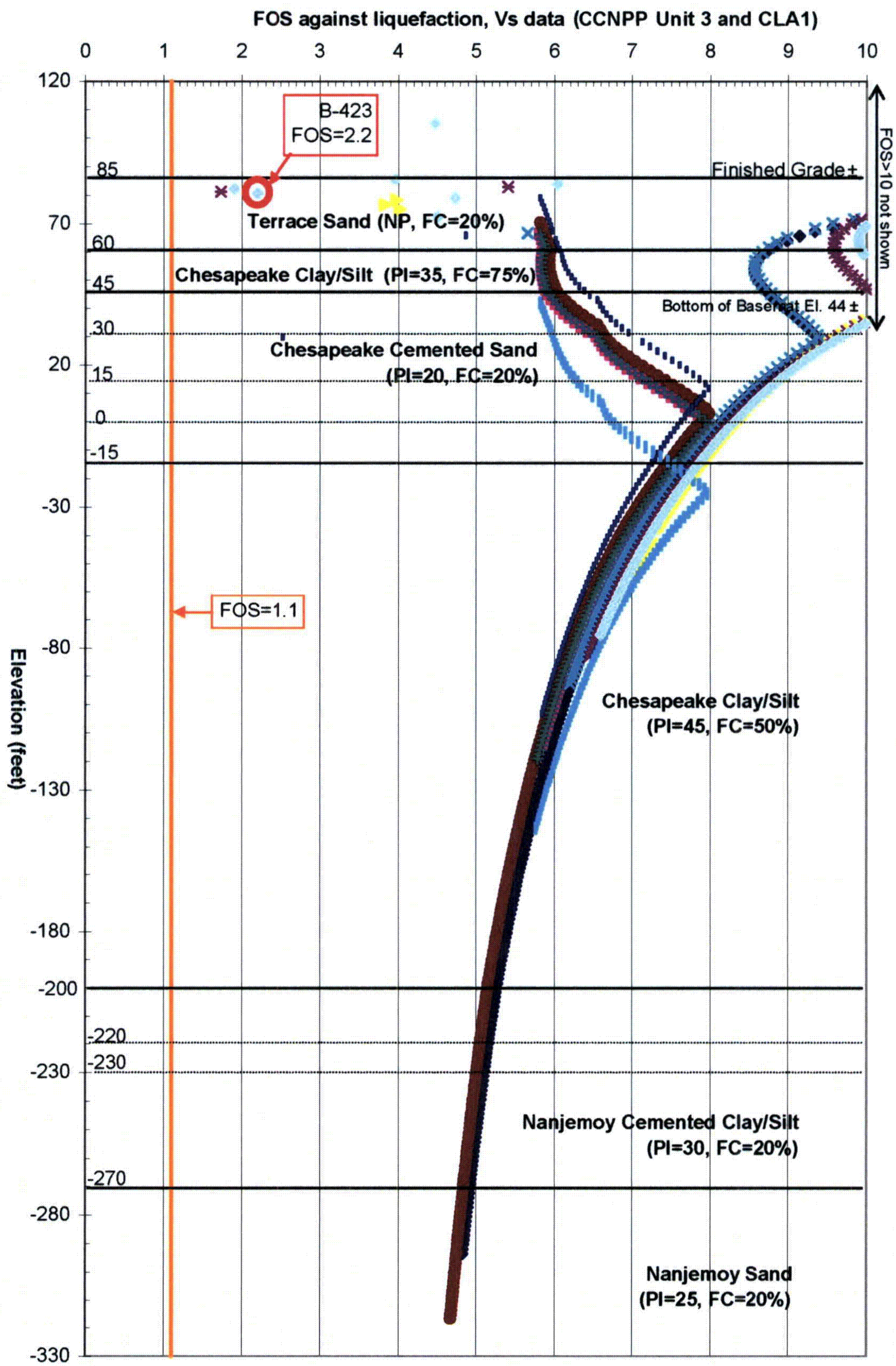


FIGURE 2.5.4-50 **Rev. 0**

CALCULATED FOS BASED ON P–S LOGGING Vs DATA

CCNPP UNIT 3 FSAR

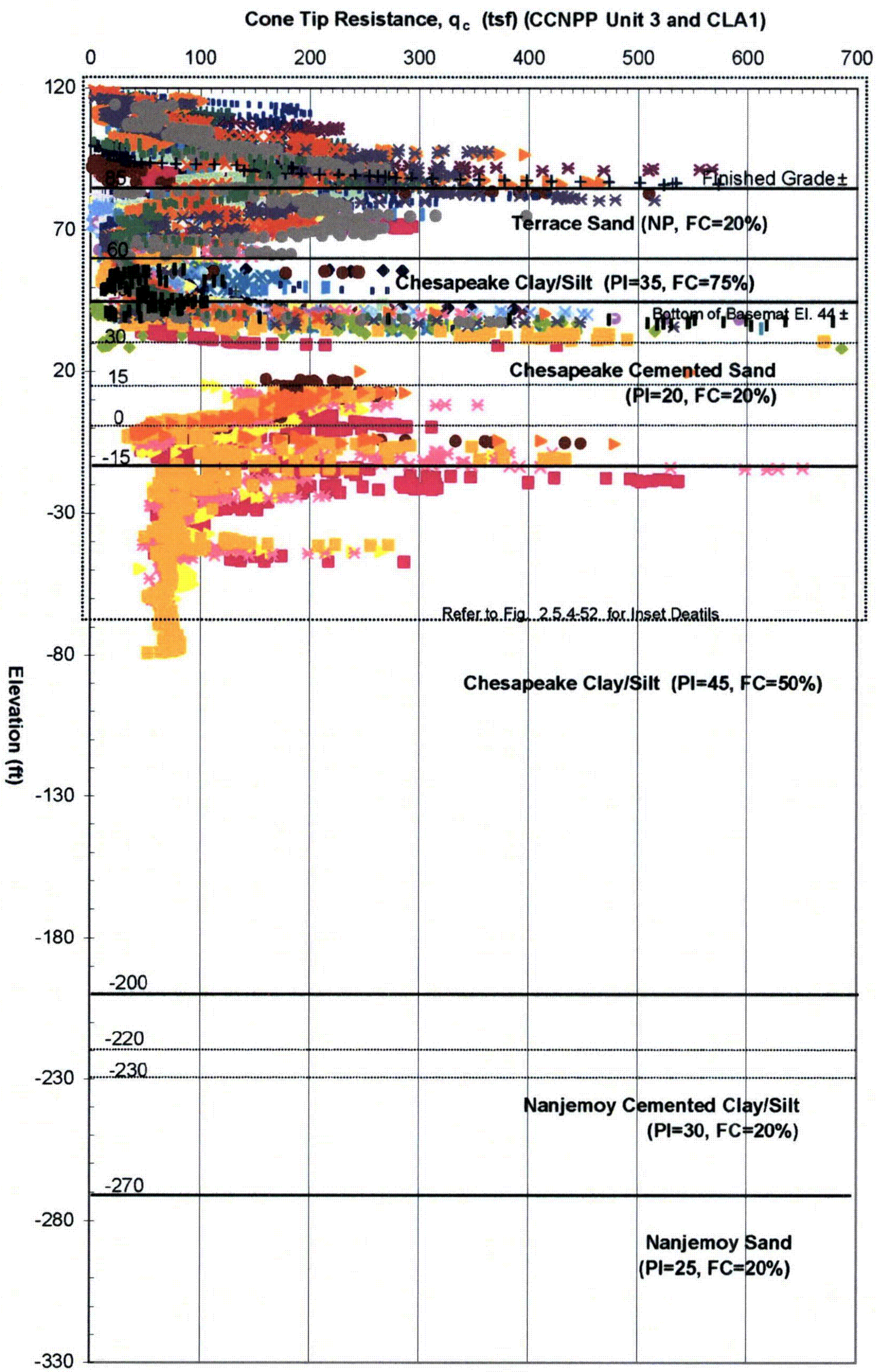


FIGURE 2.5.4-51 **Rev. 0**

MEASURED CPT TIP RESISTANCE VALUES

CCNPP UNIT 3 FSAR

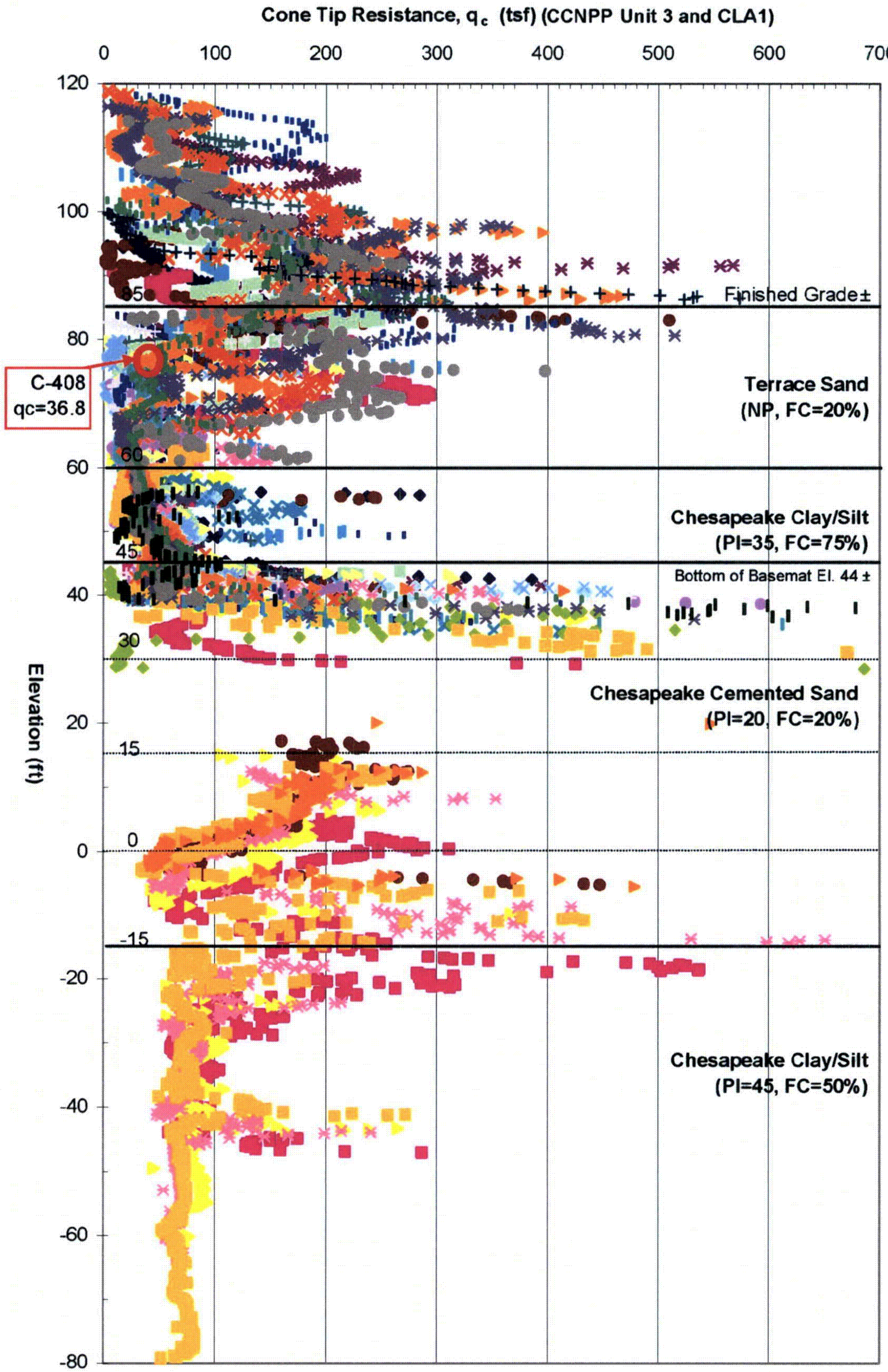


FIGURE 2.5.4-52 **Rev. 0**

MEASURED CPT TIP RESISTANCE VALUES

CCNPP UNIT 3 FSAR

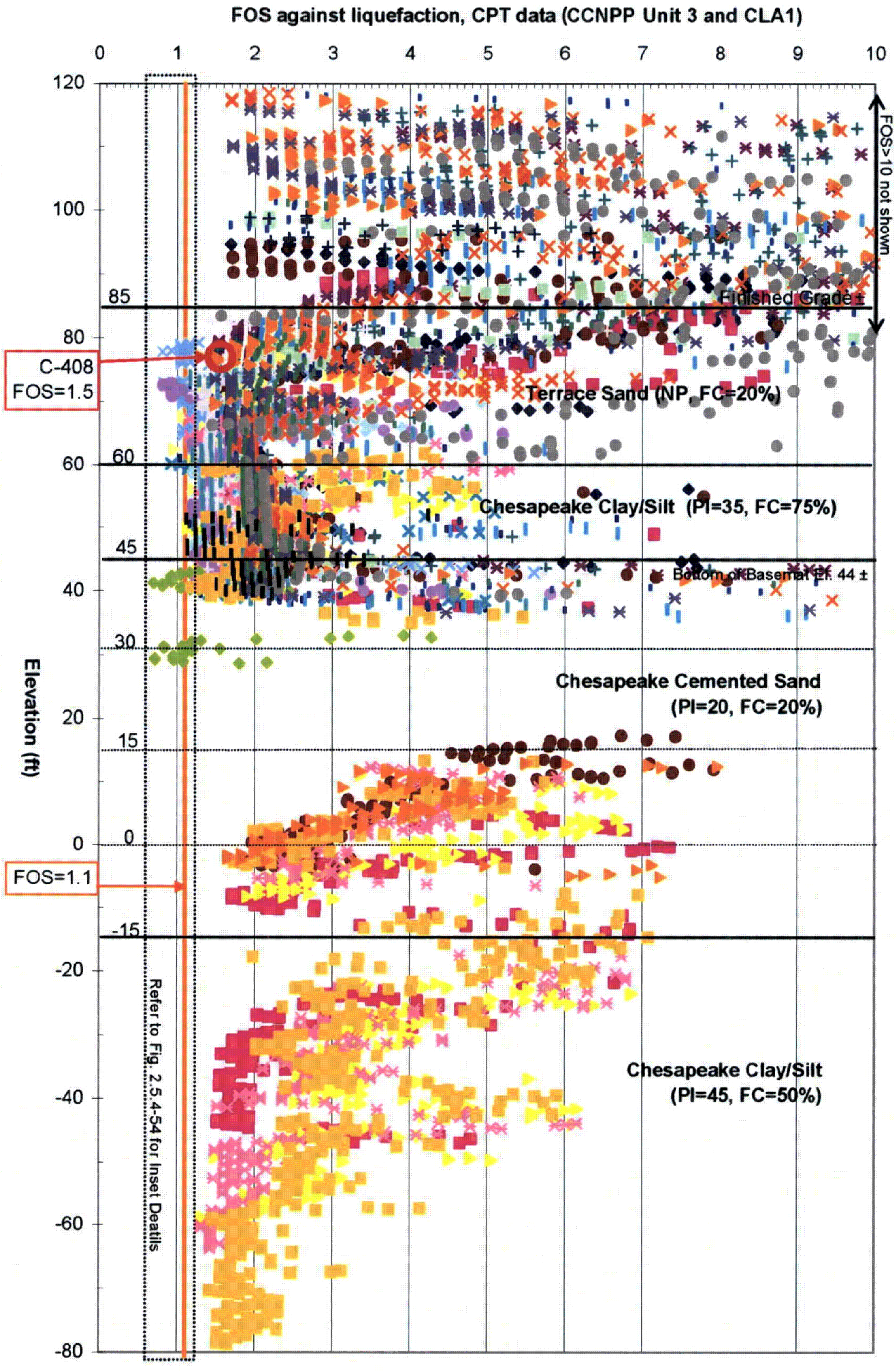


FIGURE 2.5.4-53 **Rev. 0**

CALCULATED FOS BASED ON CPT TIP RESISTANCE DATA

CCNPP UNIT 3 FSAR

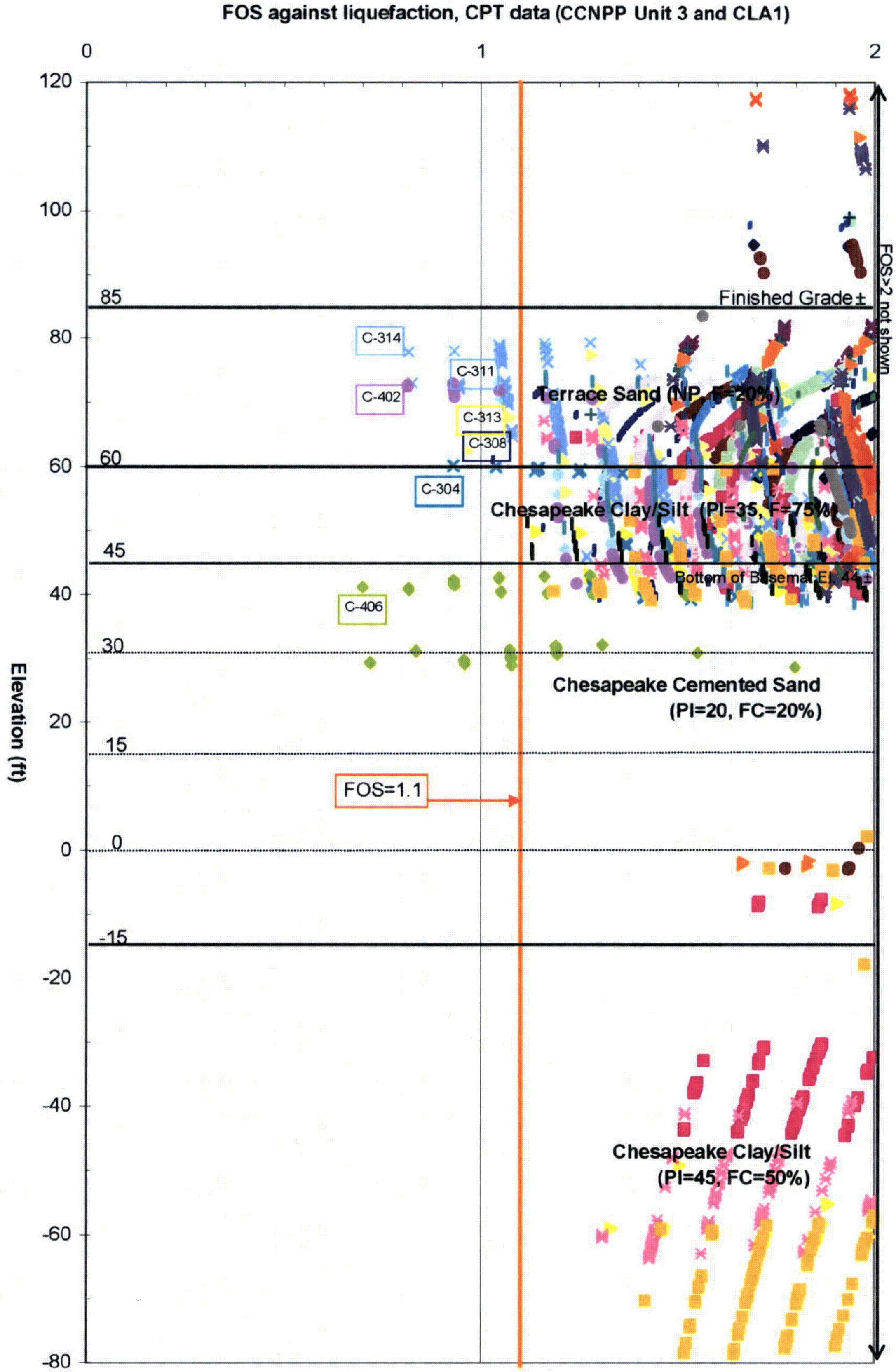


FIGURE 2.5.4-54 **Rev. 0**

CALCULATED FOS <1.1 BASED ON CPT TIP RESISTANCE DATA

CCNPP UNIT 3 FSAR

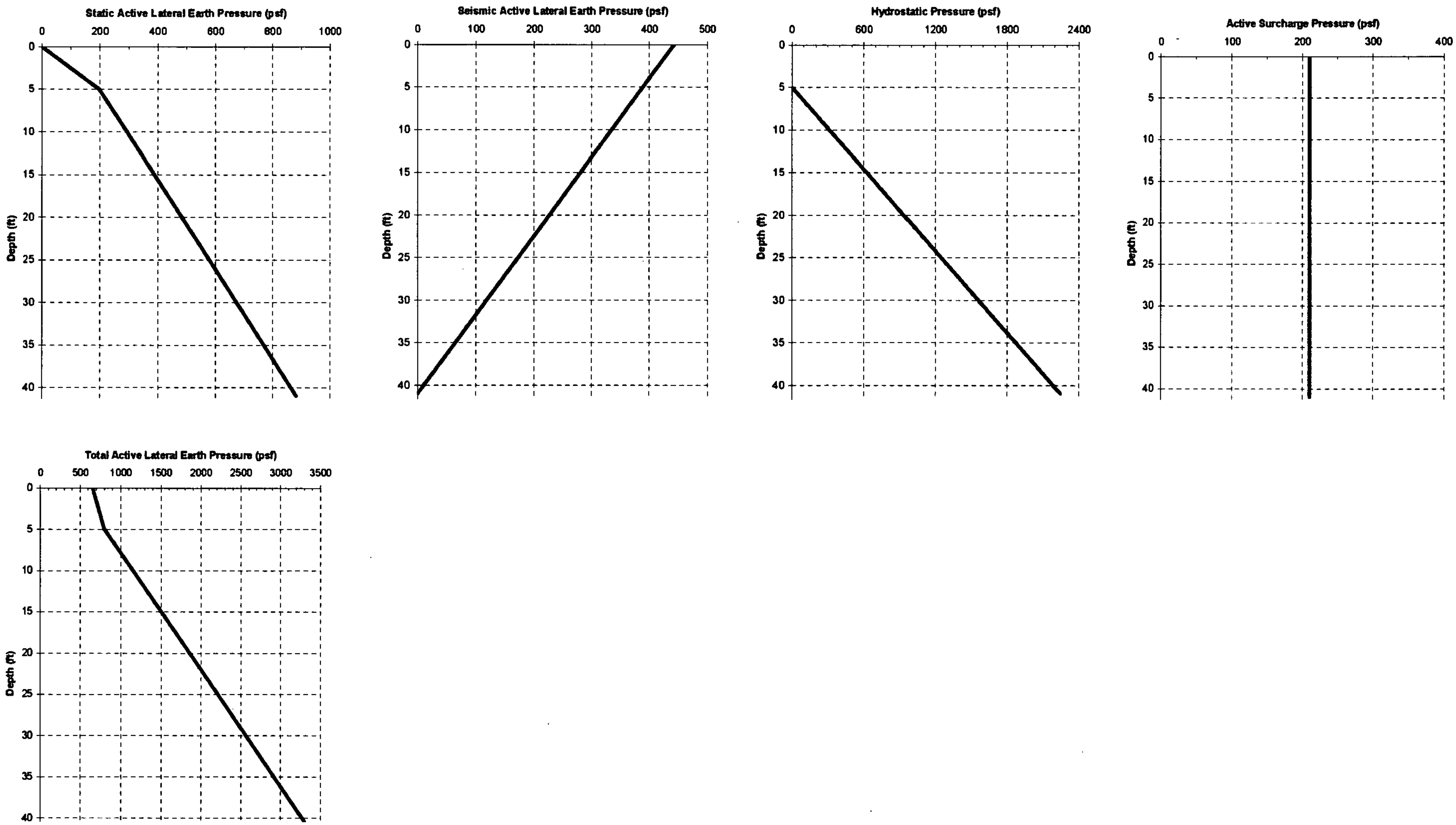


FIGURE 2.5.4-55 Rev. 0

SAMPLE ACTIVE LATERAL EARTH PRESSURE DIAGRAMS

CCNPP UNIT 3 FSAR

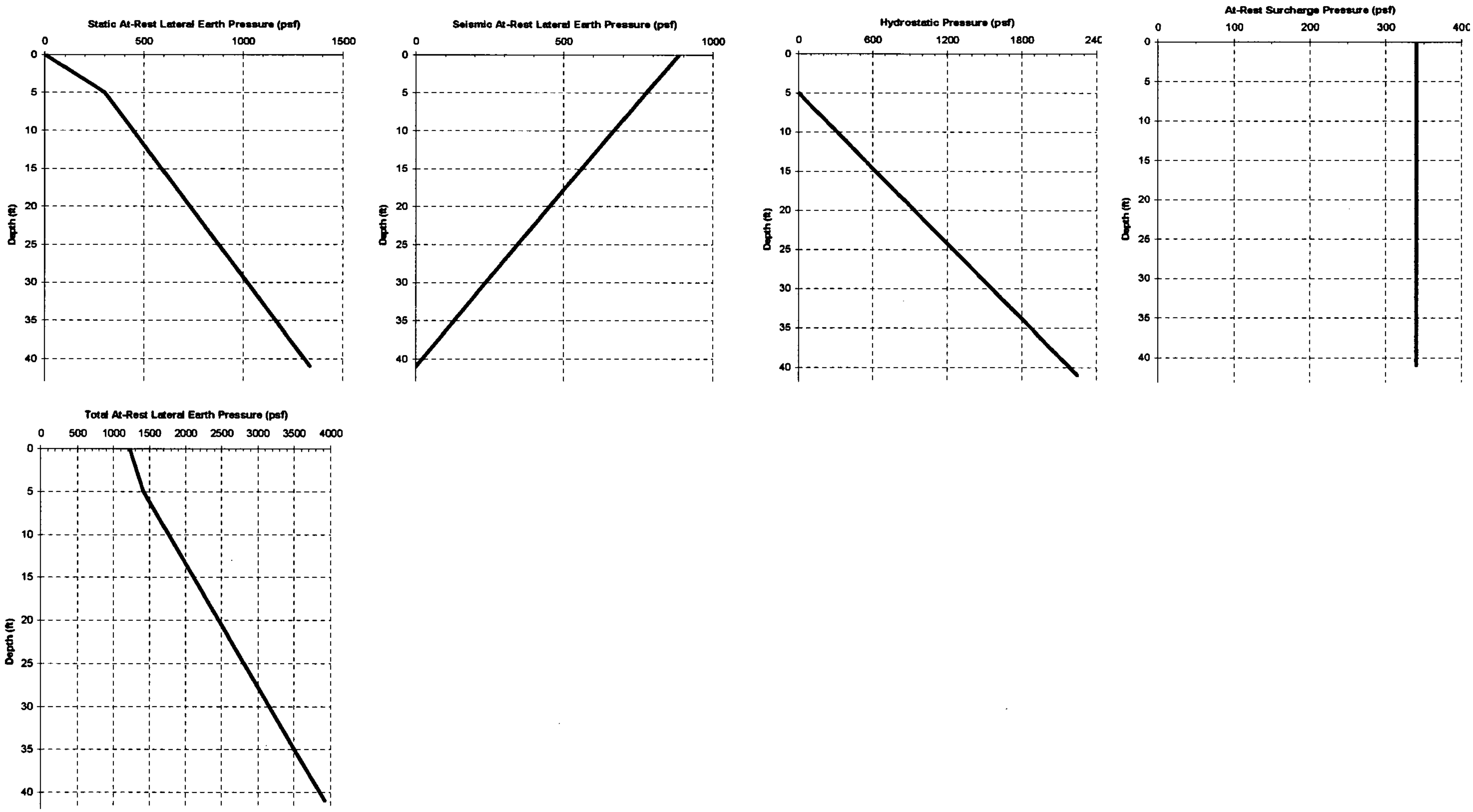


FIGURE 2.5.4-56 Rev. 0

SAMPLE AT-REST LATERAL EARTH PRESSURE DIAGRAMS

CCNPP UNIT 3 FSAR

2.5.5 STABILITY OF SLOPES

The U.S. EPR DCD includes the following COL Item for Section 2.5.5.

> A COL applicant that references the U.S. EPR design certification will evaluate site-specific information concerning the stability of earth and rock slopes, both natural and manmade (cuts, fill, embankments, dams, etc.), of which failure could adversely affect the safety of the plant.

This COL item is addressed in the following sections.

{This Section addresses the stability of constructed and natural slopes. It was prepared based on the guidance in relevant Section of NRC Regulatory Guide 1.206, "Combined License Applications for Nuclear Power Plants (LWR Edition)," (NRC, 2007). Constructed slopes evolve as part of the overall site development. The site of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 is comprised of rolling topography. The site is planned to be graded in order to establish the final grade for the project, resulting in cuts and fills, as well as slopes. The stability of these slopes and their potential impact on safety-related structures are evaluated herein. Natural slopes at the site consist of the Calvert Cliffs. They are steep slopes, undergoing continuous erosion. The impact of naturally-occurring erosion on these cliffs and their potential impact on safety-related structures are also evaluated.}

2.5.5.1 Slope Characteristics

{The characteristics of constructed and natural slopes are described below.

2.5.5.1.1 Characteristics of Constructed Slopes

Natural ground surface elevations at the CCNPP Unit 3 site area range approximately from Elevation 50 ft (15.2 m) to Elevation 120 ft (36.5 m), as shown in Figure 2.5.4-1. It is noted that all elevations referenced in this Section are based on NGVD 29. Site grading for CCNPP Unit 3 structures will include such areas as the power block, switchyard, cooling tower, and Ultimate Heat Sink (UHS) / Circulating Water Supply System (CWS) makeup intake structures. The power block includes the Reactor Building, Fuel Building, Safeguards Building, Emergency Power Generating Building, Nuclear Auxiliary Building, Access Building, Radioactive Waste Building, Turbine Building, and Ultimate Heat Sink. The centerline of the CCNPP Unit 3 power block is planned to be graded to approximately Elevation 85 ft (25.9 m). The finished grade in the area of each major structure will be approximately:

- Power block: Elevation 75 ft (22.9 m) to 85 ft (25.9 m)
- Switchyard: Elevation 90 ft (27.4 m) to 98 ft (29.9 m)
- Cooling Tower: Elevation 94 to 101 ft (28.6 m to 30.8 m)
- UHS/CWS Makeup Intake Structures: Elevation 10 ft (3 m)

Locations of these structures, and a schematic of the overall grading configuration, are shown in Figure 2.5.5-1. The site grading will require both cut and fill, currently estimated at approximately 40 ft (12.1 m) and 45 ft (13.7 m) maximum depth, respectively, except in the area near the UHS/CWS makeup intake structures where a maximum cut of about 70 ft (21.3 m) is estimated. The cut/fill operations will result in permanent slopes in and around the power block and around Category I structures outside the immediate power block area. The maximum height of new slopes in the area of CCNPP Unit 3 power block is approximately 50 ft (15.2 m), located on the eastern side of the power block area. The maximum height of new slopes in the area of UHS/CWS makeup intake structures is approximately 92 ft (28 m), located on the western side of UHS/CWS makeup intake structures. All permanent slopes, whether cut or fill, will have an inclination of 3:1 (Horizontal:Vertical). Earthworks for slope construction, including fill control, compaction, testing, etc. are addressed in Section 2.5.4.5.

Seven cross-sections (Cross-Sections A through G) that represent the typical site grading configuration were selected for evaluation based on location (e.g., proximity to major structures), slope geometry (e.g., height), and soil conditions. These cross-sections, and their locations, are shown in Figure 2.5.5-1 through Figure 2.5.5-3. Slope stability calculations were made for these cross-sections; the results are discussed in Section 2.5.5.2.

2.5.5.1.2 Characteristics of Natural Calvert Cliffs

The CCNPP Unit 3 site area is located about 1,000 ft (305 m) west of the steep cliffs known as the Calvert Cliffs, as shown in Figure 2.5.5-1. These cliffs make up the Chesapeake Bay shoreline and reach elevations as high as 100 ft (30.5 m) at their closest point to the CCNPP Unit 3 power block area. A profile of the Calvert Cliffs is shown in Figure 2.5.5-4 (BGE, 1992) for illustration purposes. Stability of the Calvert Cliffs is discussed in Section 2.5.5.2.

2.5.5.1.3 Exploration Program and Geotechnical Conditions

The soil exploration program, groundwater conditions, sampling, materials and properties, liquefaction potential, and other details are addressed in Section 2.5.4. A summary relevant to the slope stability evaluation is presented below.

Two (shallow and deep) groundwater regimes, with two different elevations (average about Elevation 80 ft (24.4 m) and Elevation 39 ft (11.9 m)), are presently identified at the CCNPP site based on on-going groundwater level measurements, as discussed in Section 2.5.4.6. The average groundwater level of Elevation 80 ft (24.4 m) was chosen for slope stability evaluation, for conservatism. In naturally low-lying areas, i.e., in areas with ground surface elevations lower than the Elevation 80 ft (24.4 m) (taken as the groundwater level), the ground may be saturated. These areas will be inspected during construction for groundwater condition. Should these areas appear saturated and if they are to receive fill during construction, a layer of highly permeable drainage material, such as crushed stone with associated filter protection, will be placed between the natural soils and the fill to preclude saturation of the fill and to maintain the groundwater level near the bottom of the fill.

As presented in detail in Section 2.5.4, the subsurface stratigraphy at this site is relatively uniform. Based on this uniformity, a typical soil profile at the site may be adopted for the purpose of slope stability evaluation. This profile is shown in Figure 2.5.5-5. The profile parameters are based on material properties derived from the data collected during the preconstruction exploration program, as presented in Table 2.5.4-12. The two soil layers referred to as Clay/Silt IIa and Clay/Silt IIc are the fine-grained portions of Chesapeake soils and are below the adopted groundwater level; therefore, their total stress properties, i.e., undrained shear strength, were used for stability analysis. The Terrace Sand and Cemented Sand are predominately granular soils; therefore, their effective stress properties were used for stability analysis. Also, since fill material for site grading purposes will be obtained from excavated portions of the Terrace Sand stratum, soil properties for the fill were adopted based on properties assigned to Terrace Sand. This is a conservative assumption given that these materials will be placed and compacted to a higher density than their current compactness level. The properties of natural Terrace Sand are given in Table 2.5.4-12. A criterion of 95% modified Proctor will be assigned to compacting these soils during construction, which is equivalent to a very dense condition, compared to their current (natural) condition which is medium dense on average.}

2.5.5.2 Design Criteria and Analysis

{The stability of constructed slopes was assessed using limit equilibrium methods, which generally consider moment or force equilibrium of a potential sliding mass by discretizing the mass into vertical slices, as shown in Figure 2.5.5-6. This approach results in a Factor Of Safety (FOS) that can be defined as (Duncan, 1996):

(Eq. 2.5.5-1)

Various limit equilibrium methods are available for slope stability evaluation, including the Ordinary method (Fellenius, 1936), Bishop's simplified method (Bishop, 1955), Janbu's simplified method (Janbu, 1968), and the Morgenstern-Price method (Morgenstern, 1965), among others. These methods were selected for evaluation of slopes for they are routinely used, and their limitations, and advantages, are well documented. The main differences are:

1. Equations of statics that are included and satisfied
2. Interslice forces that are included in the analysis
3. Assumed relationship between the interslice shear and normal forces

The Ordinary (Fellenius, 1936) method is one of the earliest methods developed. It ignores all interslice forces and satisfies only moment equilibrium. Both Bishop's (Bishop, 1955) simplified method and Janbu's (Janbu, 1968) simplified method include the interslice normal force, E, but ignore the interslice shear force, X, shown in Figure 2.5.5-6. Bishop's (Bishop, 1955) and Janbu's (Janbu, 1968) simplified methods satisfy only moment equilibrium and horizontal force equilibrium, respectively. The Morgenstern-Price (Morgenstern, 1965) method, however, considers both shear and normal interslice forces, and it satisfies both moment and force equilibrium. These four methods were used to calculate FOSs for constructed slopes at the CCNPP Unit 3 site area.

Dynamic analysis of the slopes can be performed using a pseudo-static approach, which represents the effects of seismic shaking by accelerations that create inertial forces. These forces act in the horizontal and vertical directions at the centroid of each slice, and are defined as:

(Eq. 2.5.5-2)

(Eq. 2.5.5-3)

Where ah and av are horizontal and vertical ground accelerations, respectively, W is the slice weight, and g is the gravitational acceleration constant. The inertial effect is specified by kh and kv coefficients, based on site seismic considerations.

Typical minimum acceptable values of FOS are 1.5 for normal long-term loading conditions and 1.0 to 1.2 for infrequent loading conditions (Duncan, 1996), e.g., during earthquakes.

2.5.5.2.1 Stability of Constructed Slopes

The software Slope/W (Slope/W, 2004) was used for the stability analysis. This software has been independently validated by Bechtel (Slope/W, 2005). The software searches for a critical slip surface by attempting several hundred combinations of surfaces of different shapes. Both static and pseudo-static analyses were performed for the selected cross-sections, allowing the program to search for the critical surface.

A computer analysis was made for expediting computations and to examine several hundred potential slip surfaces. The computer program Slope/W (Slope/W, 2004) was used for the stability analysis. Slope/W is an interactive program with a large number of options to suit the modeling needs of the user. In brief, the initial code for Slope/W was developed by Professor D.G. Fredlund at the University of Saskatchewan in Canada. The PC version became available in the 1980s. Slope/W contains formulation for 10 different methods for evaluating the stability of slopes, each with various assumptions in its development of the respective mathematical model, some of which were described earlier in Section 2.5.5.2, with the main difference being in the treatment of interslice forces. Slope/W contains a variety of options for the shape of trial surfaces, e.g., circular, planar, composite, or block type, and locates the critical surface with the lowest possible FOS. The reasonableness of the surface, however, should be determined by the user as Slope/W, or other similar applications, cannot be expected to make these judgments. Slope/W also allows for the incorporation of forces due to water, as well as negative pore water (suction) and externally applied forces, when needed. Material properties may simply be defined in terms of friction and/or cohesion, or made a function of others parameters, e.g., change with stress. Slope/W has two options for evaluating slopes subjected to rapid loading; namely, pseudostatically or using results from other dynamic analyses such as a companion program that obtains dynamic stresses and pore water pressure. Slope/W offers many other computational options. A complete description of Slope/W and slope stability formulations is given in Slope/W user Manual (SlopeW, 2004) and Krahn (Krahn, 2004).

The effect of surcharge loading was excluded from the analyses. Planned structures are sufficiently set back from edges of slopes so that they do not impose surcharge loading on the slopes, as evident in Figure 2.5.5-2 and Figure 2.5.5-3. The site soils are not considered liquefiable for the seismic conditions of the site; therefore, liquefaction is not applicable to stability of slopes at the site. Liquefaction potential is addressed in detail in Section 2.5.4.8.

For the pseudo-static analysis, the inertial effect coefficient kh=0.125 was used, based on ah=0.125g, as discussed in Section 2.5.4.7. The vertical component, kv, was chosen as 0.063.

Results of the static and pseudo-static slope stability analyses for critical surfaces, i.e., surfaces with the lowest FOS, are shown in Figure 2.5.5-7 through Figure 2.5.5-13. The computed FOSs shown on these figures are based on the Morgenstern-Price (Morgenstern, 1965) method. This method was chosen for its complete consideration of interslice forces as well as force and moment equilibrium. In addition to the Morgenstern-Price (Morgenstern, 1965) method, FOSs were also estimated using the Ordinary (Fellenius, 1936) method, Bishop's (Bishop, 1955) simplified method, and Janbu's (Janbu, 1968) simplified method for comparison, which are all implemented in Slope/W. These FOSs are summarized in Table 2.5.5-1. An examination of the FOSs in Table 2.5.5-1 indicates that for a particular slope, there is no appreciable difference among the FOSs computed by the different methods. The FOSs, based on the Morgenstern-Price (Morgenstern, 1965) method, range from about 1.4 to 1.9 from the static analysis and from about 1.0 to 1.4 from the pseudo-static analysis. The FOSs are further explained below, referencing results obtained from the Morgenstern-Price (Morgenstern, 1965) method.

In the power block and adjacent areas (Cross-sections A through F in Figure 2.5.5-2), all slopes show FOSs greater than 1.8 for the static case and greater than 1.3 for the pseudo-static case. Additionally, in this area, all slopes indicated that the critical sliding surface is very limited and surficial, except for cross-Section B. The static FOS for cross-Section B exceeds 1.8, despite the deep-seated surface indicated. Since all cross-sections analyzed, except cross-Section B, resulted in shallow, sloughing-type slip surfaces, additional analyses were made to evaluate FOSs associated with potential deeper slip surfaces. The deeper slip surfaces were arbitrarily chosen, but forced into deeper soils to encompass a larger volume of the soil mass. The analyses were repeated for all sections that showed shallow slip surfaces in the initial trial. The estimated FOSs for the deeper (forced) surfaces are shown in Table 2.5.5-2; location of deeper surfaces are shown in Figure 2.5.5-14 through Figure 2.5.5-18. As would be expected, these

FOSs are higher than those previously estimated, and they are at least 2.0 for the static case and at least 1.4 for the pseudo-static case, based on the Morgenstern-Price method. These FOSs are consistent with the simple formulation of stability, based on the ratio of the tangent of soil friction to tangent of slope inclination, or a FOS of about 1.9 for □=32 degrees and slope inclination of 18.4 degrees (3H:1V).

In the UHS/CWS makeup intake structures area, at cross-Section G shown in Figure 2.5.5-3, a static FOS of 1.42 and a pseudo-static FOS of 1.02 were estimated with the Morgenstern-Price (Morgenstern, 1965) method, as shown in Figure 2.5.5-13. The slope at this cross-Section is 3:1 H:V, and is approximately 91 ft (27.7 m) high. The preconstruction site exploration did not specifically include borings or other tests in this area for the purpose of evaluating the stability of this slope. The stability evaluation, however, was performed due to a recent design modification that placed the UHS makeup water intake structure and the CWS makeup water intake structure near the shoreline of the Chesapeake Bay. Only the UHS makeup water intake structure is considered a Category I structure. In absence of data specific to the slope in this area, the average soil model (thickness, elevations, properties, etc.), as obtained from the investigation in the power block area, shown in Figure 2.5.5-5, was applied to this slope, including the adopted Elevation 80 ft (24.4 m)for groundwater level, resulting in the referenced FOSs.

As stated previously, typical minimum acceptable values of FOS are 1.5 for normal long-term loading conditions and 1.0 to 1.2 for infrequent loading conditions. In and around the power block area, the calculated FOSs for all slopes exceed 1.8 for the static case and 1.3 for the dynamic case. Accordingly, the slopes in the power block area have sufficient static and dynamic stability. This conclusion is consistent with conditions at the site for the relatively flat slope geometry of 3:1 H:V, groundwater level below the ground surface, and relatively dense/stiff soil conditions. For the slope adjoining the UHS/CWS makeup intake structures, the static FOS is near 1.4 and the dynamic FOS is near 1.0. The FOSs are slightly lower than those typically applied to similar slopes. The lower values are very likely the result of the assumed model for this slope in absence of actual data, such as stratigraphy, groundwater level, etc. Based on expected performance from similar slopes, a 3:1 slope in dense/stiff soils such as those at the site, is expected to result in slightly higher FOSs. It is noted that the horizontal distance between the toe of slope G and the UHS makeup water intake structure is about 160 ft (48.8). Assuming the dynamic FOS is realistic, should this slope fail during a seismic event, and should the soils from the failure have enough energy to reach the UHS makeup water intake structure (a distance of about 160 ft (48.8 m)), the volume of soils reaching this structure could result in loading the adjacent wall of the structure by an equivalent soil height of about 1 ft (0.3 m). The magnitude of this loading is considered small relative to other dynamic loads that this structure is expected to be designed for, such from earthquakes, hurricanes, or tsunamis. Nonetheless, this slope will be the subject of an evaluation once again during the detailed design phase of the project. To obtain refined FOSs for this slope, borings and other tests will be performed in this area, and the slope conditions re-evaluated during detailed design. Should the results at that time indicate unacceptable FOSs, additional measures will be taken to mitigate its impact on the Category I, UHS makeup intake structure, such as by further flattening of the slope, further set back from structures, or other engineering measures.

Results of stability analyses are presented in Table 2.5.5-1 and Table 2.5.5-2. The strength parameters for materials are shown in Figure 2.5.5-5. Forces acting on the slope (slices) are shown in Figure 2.5.5-6. There are no external forces, i.e., surcharge, acting on the slopes. Pore pressures acting within the slope are represented by the groundwater condition adopted at Elevation 80 ft (24.4 m), as shown in Figure 2.5.5-5. The types of failure surfaces are shown on the final stability results in Figure 2.5.5-7 through Figure 2.5.5-13. Units for soil properties shown on these figures are consistent with those shown in Figure 2.5.5-5.

The critical surfaces are shown graphically in Figure 2.5.5-7 through Figure 2.5.5-13. The FOSs associated with these surfaces are identified in Table 2.5.5-1 and Table 2.5.5-2. Locations of sections are shown in Figure 2.5.5-2 and Figure 2.5.5-3.

Dams and embankments, including description of any adverse conditions such as high water levels attributable to the Probable Maximum Flood (PMF), sudden drawdown, or steady seepage at various levels are addressed in Section 2.5.6.

2.5.5.2.1 Stability of Natural Calvert Cliffs

The Calvert Cliffs are steep, near-vertical slopes, formed by erosion processes over the last several thousand years. These processes are addressed in more detail in Section 2.4.9. The on-going erosion results in the cliffs failing along irregular, near-vertical surfaces. The failures are the result of shoreline erosion undermining the cliffs at the beach line. With sufficient undermining, the weight of the overlying deposits that make up the cliffs exceeds their shear strength, resulting in the undermined portion falling to the shoreline. Long-term and short-term processes, e.g., waves, tidal fluctuations, and extreme weather conditions, affect the Calvert Cliffs. The cliffs are estimated to undergo erosion near the CCNPP Unit 3 site area of about 2 ft (0.6 m) to 4 (1.2 m) ft per year, as described in Section 2.4.9.

In the proximity of CCNPP Unit 3, the cliffs rise to elevations in the range of about Elevation 30 ft (9.1 m) to Elevation 100 ft (30.5 m), with a major portion maintaining about Elevation 90 ft (27.4 m), as shown in Figure 2.5.5-1. Given the past performance of the high cliffs, there is no reason to expect their future performance would appreciably differ; therefore, these cliffs are anticipated to continue to be globally stable, owing to the relatively high strength of the soil deposits that make up the cliffs (refer to Section 2.5.4.2 for strength data for these soils). Consistent with the results of the preconstruction exploration, all soils that make up the cliffs also include some level of plasticity, as well as a moderate amount of fines (refer to Table 2.5.4-8 for data), resulting in moderate capillary forces and, therefore, enhanced stability and resistance to erosion.

The easternmost boundary of the CCNPP Unit 3 power block is set back a distance of about 1,000 ft (305 m) from the cliffs, with at least 1,200 ft (365.8 m) to the nearest Category I structure, as shown in Figure 2.5.5-1. This set back area will be free from any major construction, surcharge, re-grading, or other activities that could modify the ground or the loading conditions which would adversely impact the cliffs or their stability. Therefore, they are anticipated to remain unaffected by construction factors.

Although not expected, should the global stability of the cliffs, due to unforeseen conditions, be adversely impacted such that a major cliff failure could ensue, hypothesized failure scenarios may be in the form of (1) a wedge (or a plane) portion of the cliffs sliding into the Chesapeake Bay at an inclined angle, or (2) a portion of the cliffs separate and topple into the Chesapeake Bay. For the wedge-shaped hypothesis, conservatively assuming that an inclined angle of 45 degrees from the base of the cliffs could form a wedge that daylights at the top of the cliffs, only an area of approximately 100 ft (30.5 m) from the cliffs' edge would be impacted by such an unexpected scenario, and the remaining 900-plus ft (274-plus m) setback area would still be intact to provide sufficient global stability to CCNPP Unit 3. For the toppling hypothesis, except for cases associated with erosion that will be discussed below, the hydrogeologic conditions that are prerequisite to this failure situation are not known to exist at the site, such as fractured bedrock or soils with planes of weakness due to fissures, slickensides, faults, or discontinuities; excessive seepage forces that could promote such failures; or prior failure history of the type hypothesized. Therefore, massive toppling failure of the Calvert Cliffs that could have an immediate, adverse impact on CCNPP Unit 3 is not kinematically possible.

The Calvert Cliffs, however, are expected to continue to erode, as they have in the past. Based on the estimated rate of erosion of 2 ft (0.6 m) to 4 ft (1.2 m) annually, at a constant rate, it will take approximately 25 to 50 years to erode about 100 ft (30.5 m) of the cliffs. Or, it would take approximately 125 to 250 years for the cliffs to erode to within a distance of 500 ft (152.4 m) from CCNPP Unit 3 outline (or 700 ft (213.4 m) from any Category I structure). The estimated periods of 125 to 250 years are appreciably more than the anticipated operating life of CCNPP Unit 3; therefore, stability of Calvert Cliffs due to erosion should not pose any immediate risk to the stability of soils supporting CCNPP Unit 3 in its lifetime.}

2.5.5.2.2 Concluding Remarks

Based on analyses provided in this Section, it is concluded that the constructed and natural slopes at the site are sufficiently stable and present no failure potential that would adversely affect the safety of the proposed {CCNPP Unit 3. If final geotechnical results for the area of the UHS makeup water intake structure indicate than any potential slope failure could adversely affect the safety of the proposed CCNPP Unit 3, corrective actions will be taken to preclude this potential slope failure.}

2.5.5.3 Logs of Borings

Log of borings, and associated references, are provided in Appendix 2.5-A.

2.5.5.4 Compacted Fill

{Compacted fill, and associated references, are addressed in Section 2.5.4.5.}

Table 2.5.5-1 {Computed Factors of Safety (FOS) for the Critical Slip Surface} (Page 1 of 1)

Slope Section	Static Analysis				Pseudo-Static Analyiss			
	Ordinary	Bishop	Janbu	M-P	Ordinary	Bishop	Janbu	M-P
A	1.89	1.89	1.89	1.89	1.34	1.34	1.34	1.34
B	1.77	1.84	1.82	1.85	1.26	1.36	1.31	1.41
C	1.89	1.89	1.88	1.89	1.35	1.35	1.34	1.35
D	1.88	1.88	1.88	1.88	1.33	1.34	1.33	1.33
E	1.88	1.88	1.88	1.88	1.33	1.34	1.33	1.33
F	1.96	1.96	1.96	1.96	1.38	1.38	1.38	1.38
G	1.30	1.41	1.35	1.42	0.97	1.01	0.98	1.02

Notes:

Ordinary = Ordinary method
Bishop = Bishop's simplified method
Janbu = Janbu's simplified method
M-P = Morgenstern-Price method

Table 2.5.5-2 {Computed Factors of Safety (FOS) for (Forced) Deeper Slip Surfaces}
(Page 1 of 1)

Slope Section	Static Analysis				Pseudo-Static Analyiss			
	Ordinary	Bishop	Janbu	M-P	Ordinary	Bishop	Janbu	M-P
A	1.79	1.98	1.86	2.00	1.37	1.40	1.37	1.41
B	---	---	---	---	---	---	---	---
C	2.10	2.16	2.10	2.16	1.46	1.51	1.47	1.51
D	1.94	1.99	1.94	1.99	1.38	1.42	1.38	1.42
E	1.98	2.03	1.98	2.03	1.40	1.44	1.40	1.44
F	1.98	2.03	1.98	2.03	1.40	1.44	1.40	1.44
G	---	---	---	---	---	---	---	---

Notes:

Ordinary = Ordinary method
Bishop = Bishop's simplified method
Janbu = Janbu's simplified method
M-P = Morgenstern-Price method
--- indicates no computation

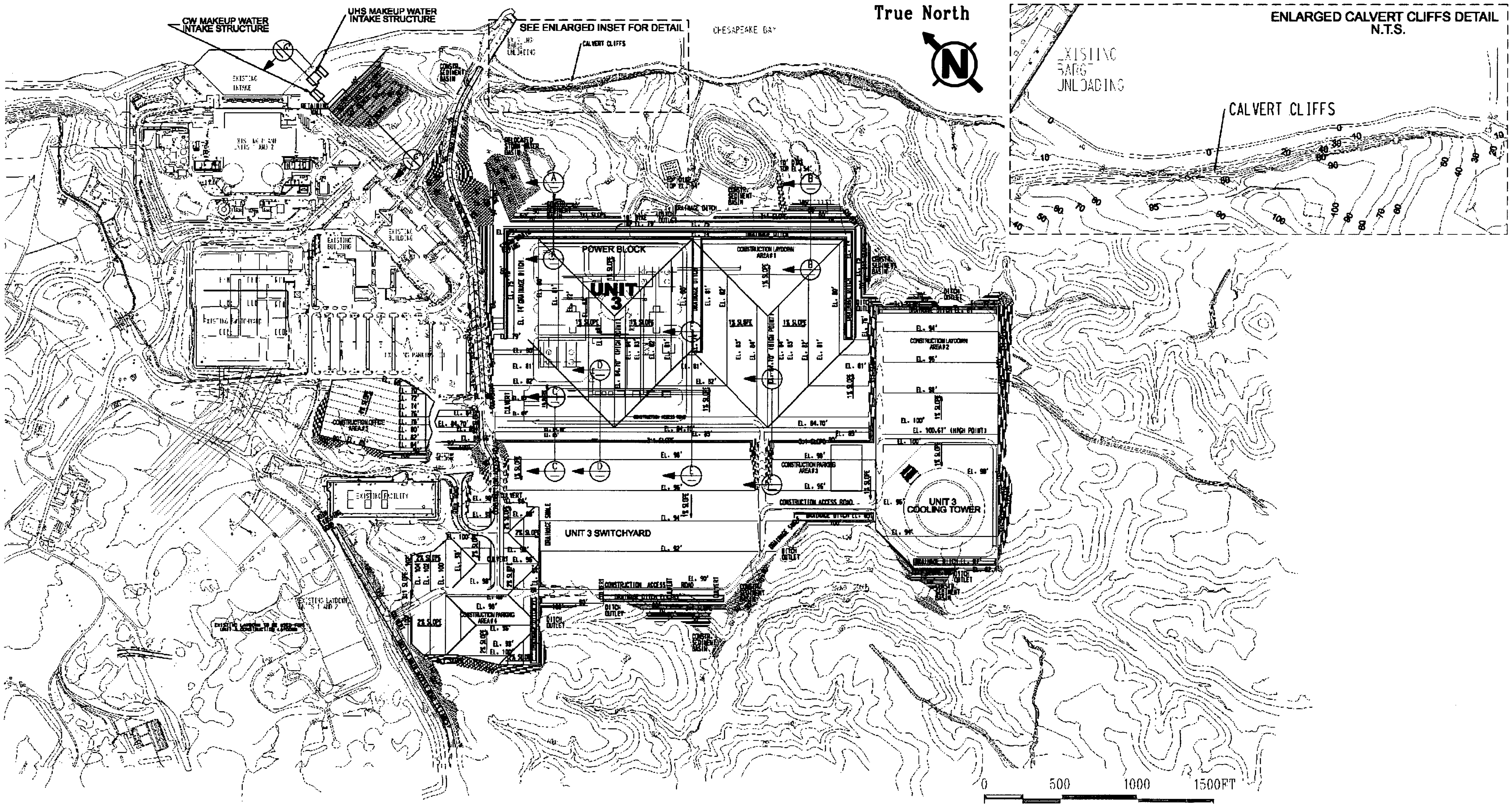


FIGURE 2.5.5-1 **Rev. 0**

SITE GRADING PLAN

CCNPP UNIT 3 FSAR

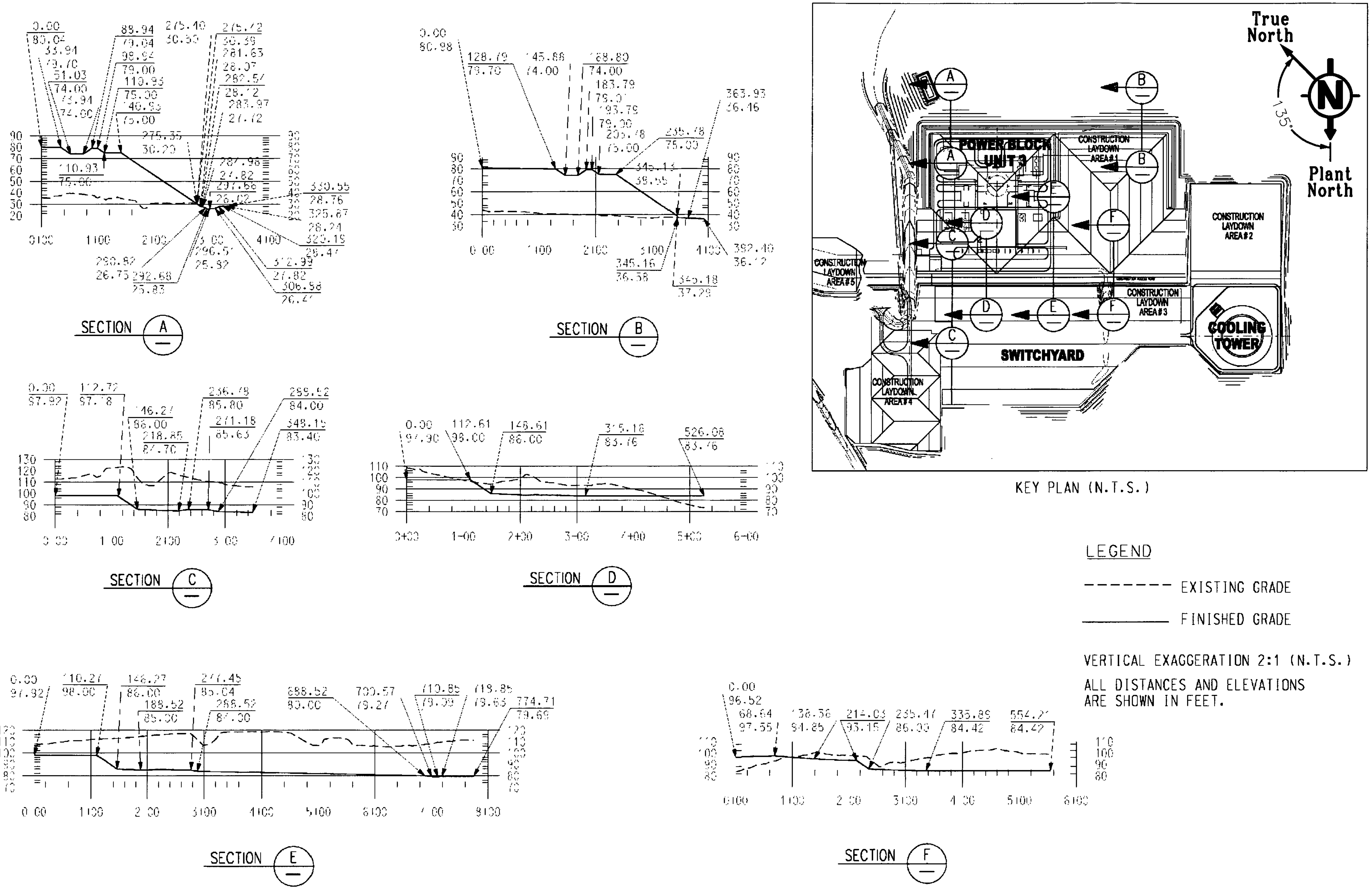
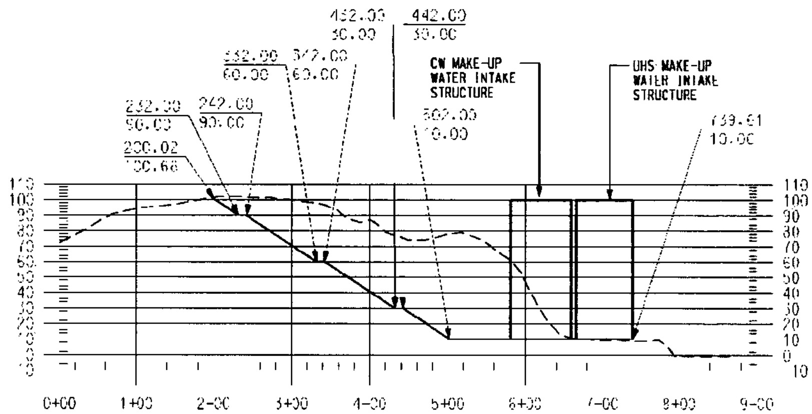


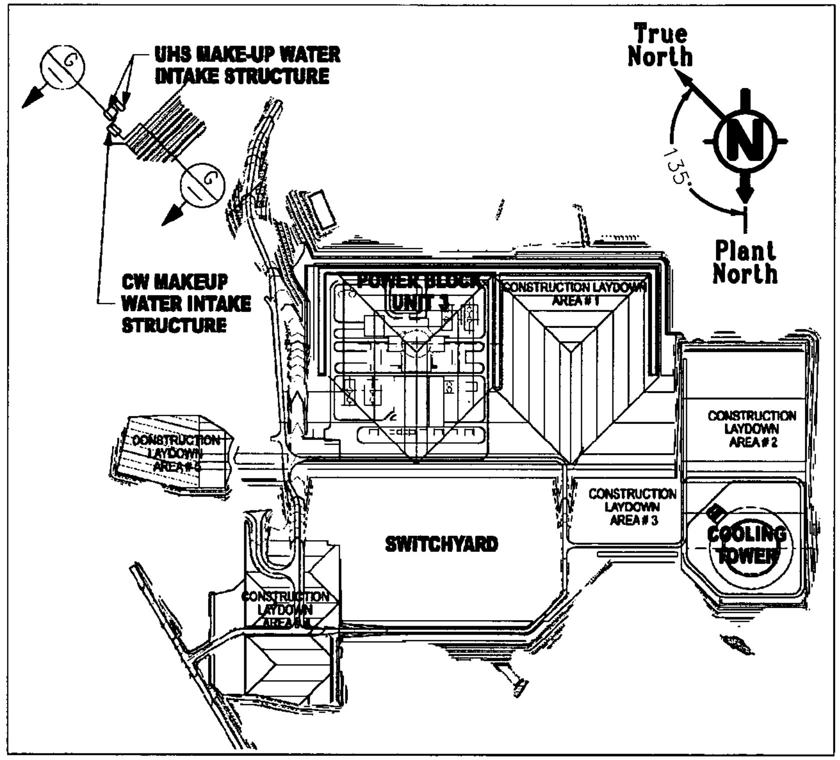
FIGURE 2.5.5-2 **Rev. 0**

CROSS-SECTIONS IN POWERBLOCK AREA

CCNPP UNIT 3 FSAR



SECTION G



KEY PLAN (N.T.S.)

LEGEND

------- EXISTING GRADE

———— FINISHED GRADE

VERTICAL EXAGGERATION 2:1 (N.T.S.)

ALL DISTANCES AND ELEVATIONS ARE SHOWN IN FEET.

FIGURE 2.5.5-3 **Rev. 0**

CROSS-SECTION IN INTAKE STRUCTRES AREA

CCNPP UNIT 3 FSAR

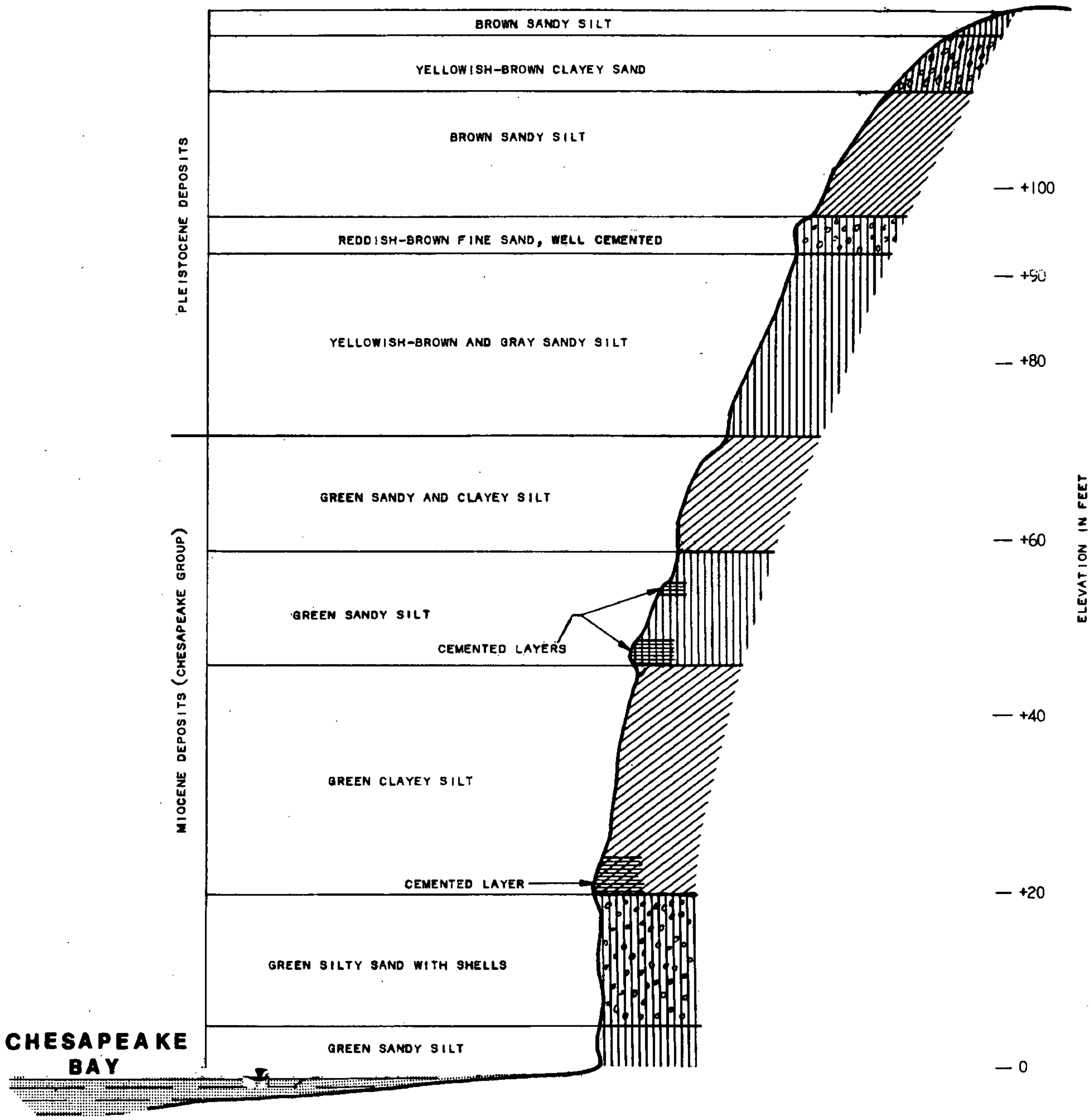


FIGURE 2.5.5-4 **Rev. 0**

SCHEMATIC SECTION OF CALVERT CLIFFS(1)

CCNPP UNIT 3 FSAR

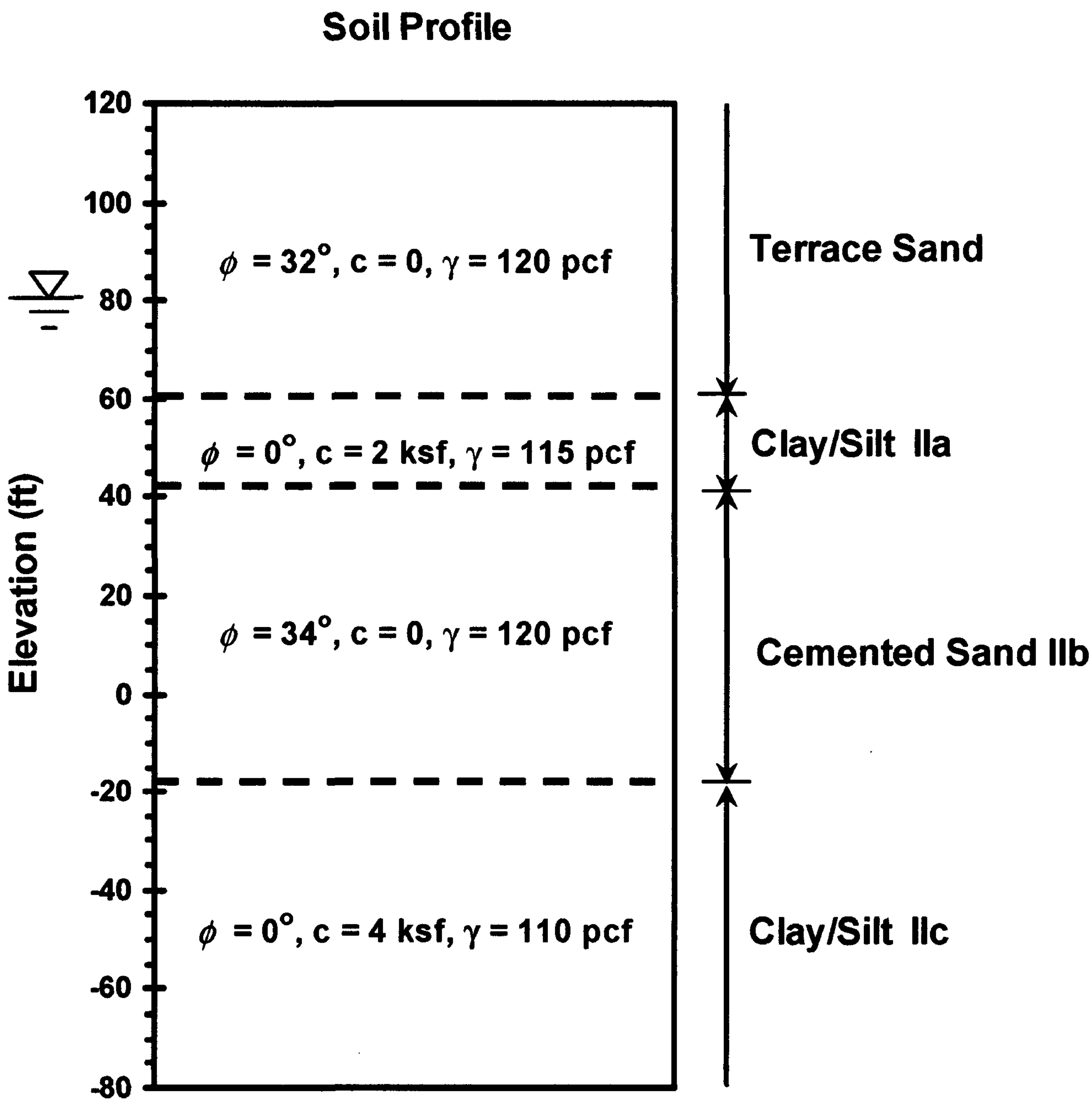
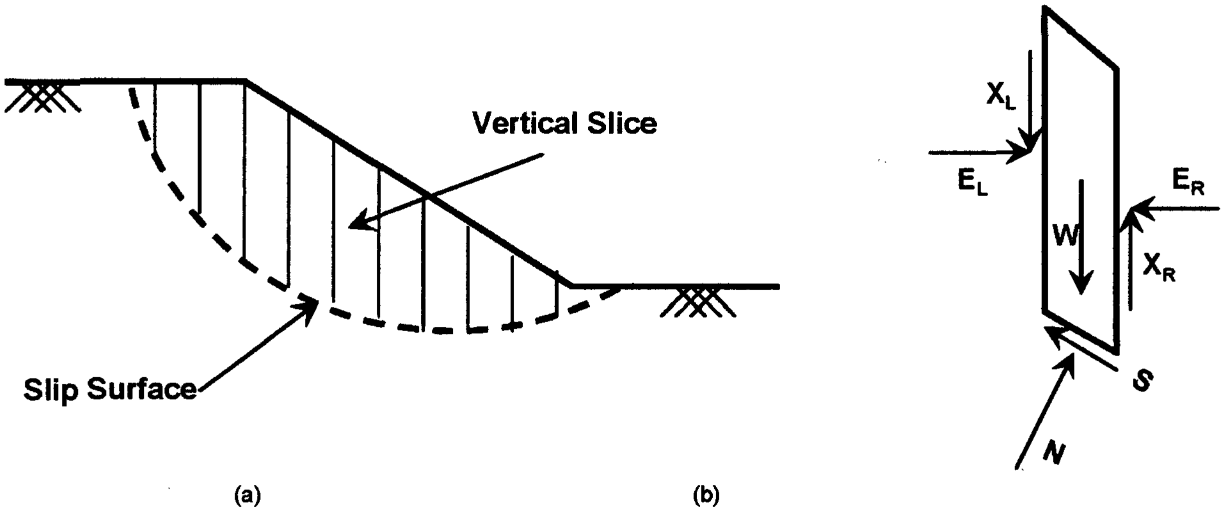


FIGURE 2.5.5-5 **Rev. 0**

SOIL STRATIGRAPHY AND PROPERTIES FOR SLOPE STABILITY ANALYSIS

CCNPP UNIT 3 FSAR



(a) Sliding Mass Discretized into Vertical Slices
(b) Forces on a Single Vertical Slice

FIGURE 2.5.5-6 **Rev. 0**

LIMIT EQUILIBRIUM SLOPE STABILITY ANALYSIS

CCNPP UNIT 3 FSAR

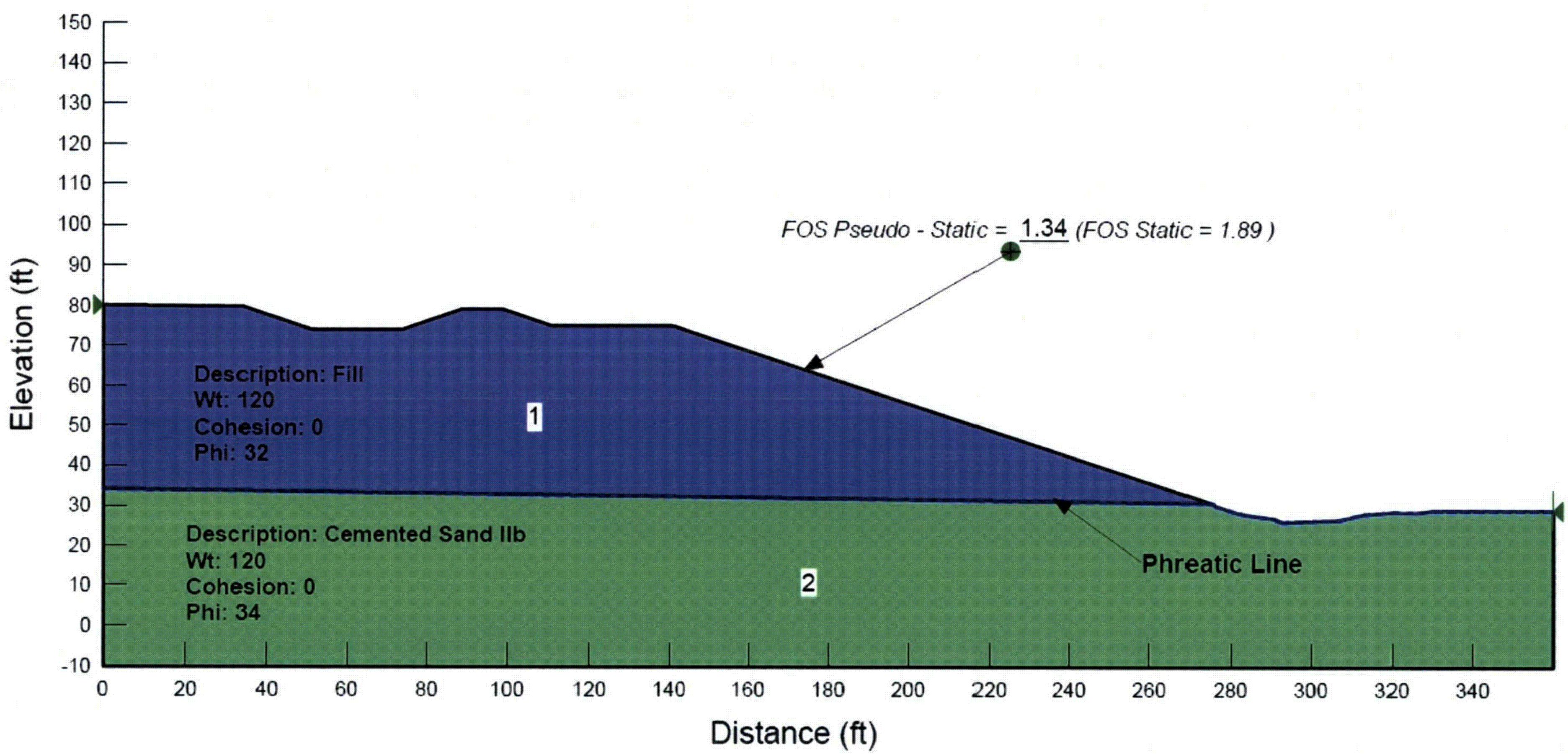


FIGURE 2.5.5-7 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION A

CCNPP UNIT 3 FSAR

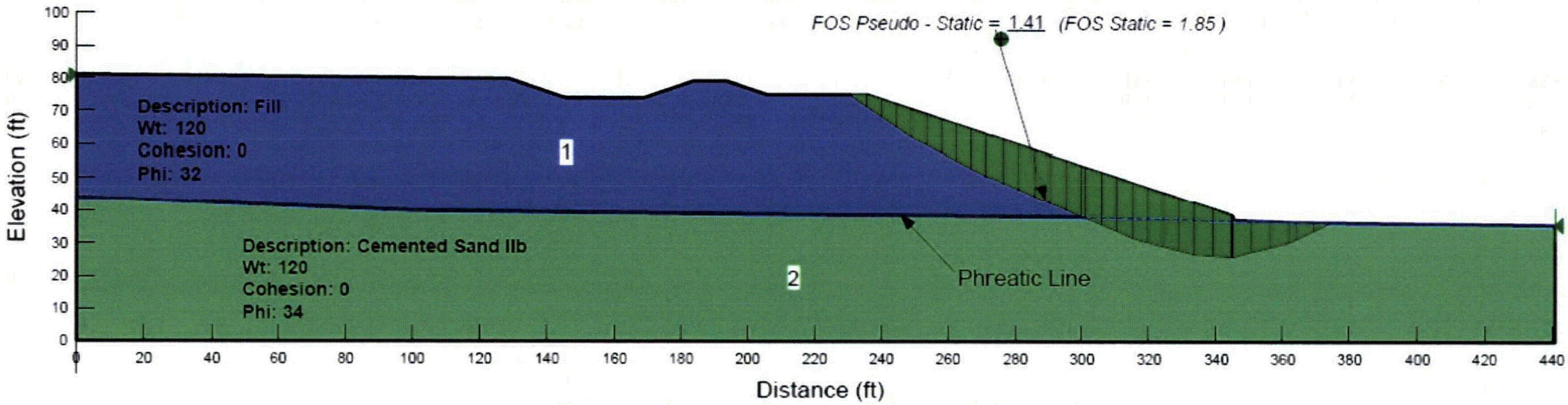


FIGURE 2.5.5-8 Rev. 0

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION B

CCNPP UNIT 3 FSAR

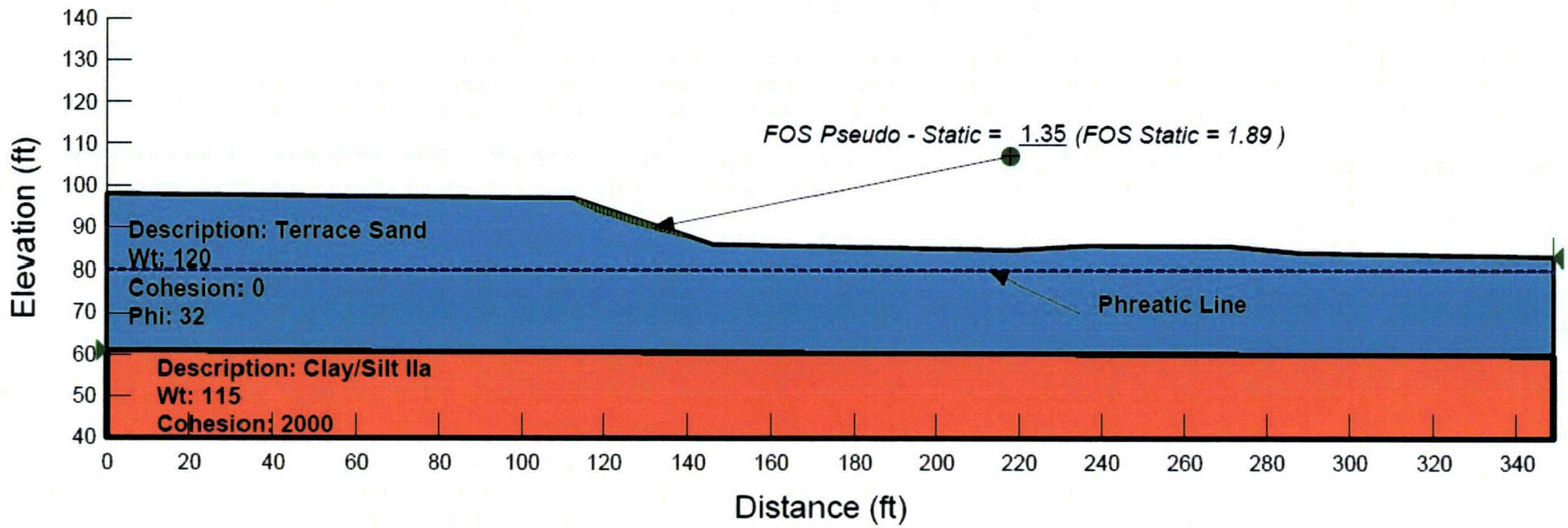


FIGURE 2.5.5-9 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION C

CCNPP UNIT 3 FSAR

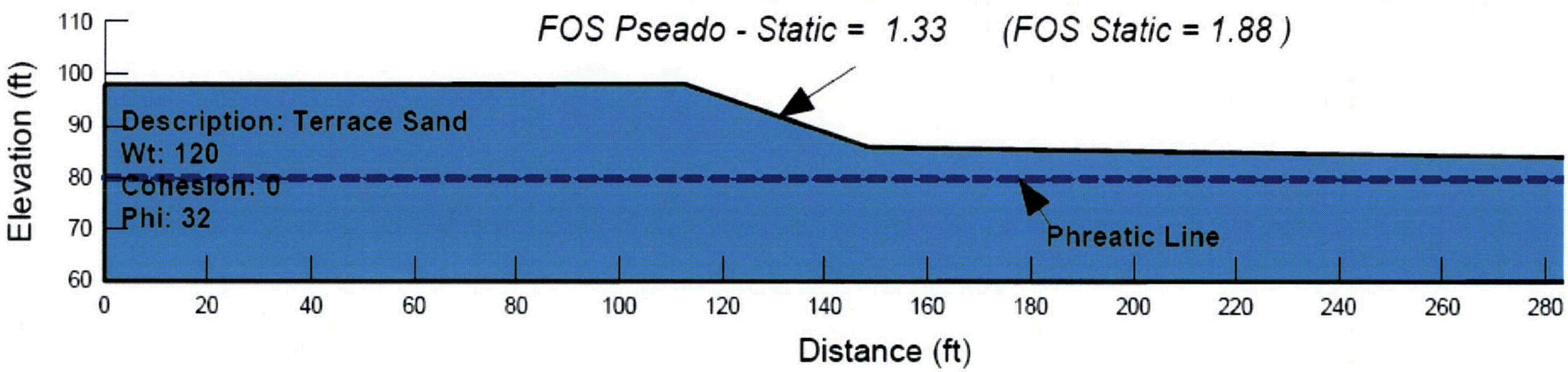


FIGURE 2.5.5-10 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION D

CCNPP UNIT 3 FSAR

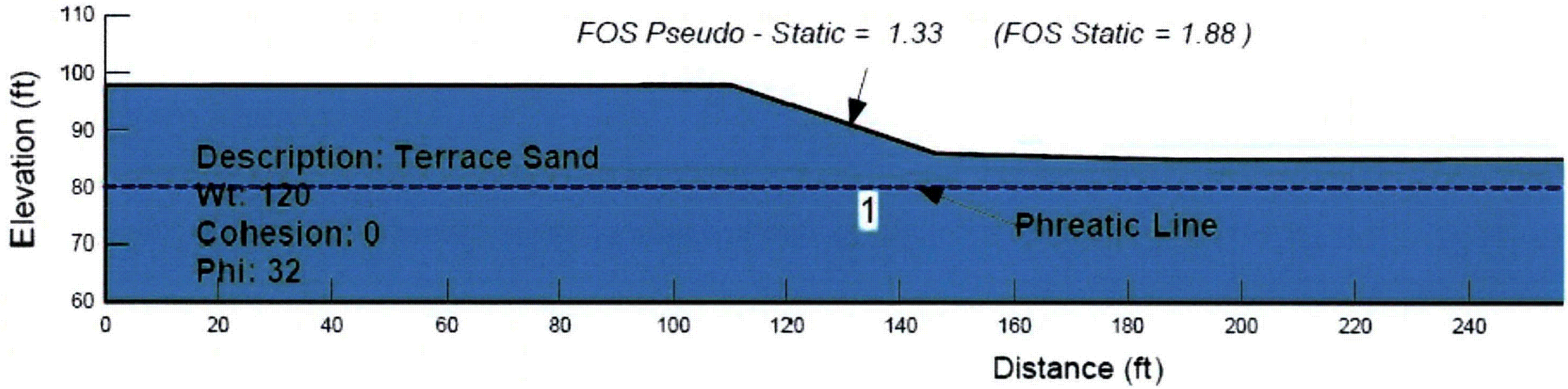


FIGURE 2.5.5-11 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION E

CCNPP UNIT 3 FSAR

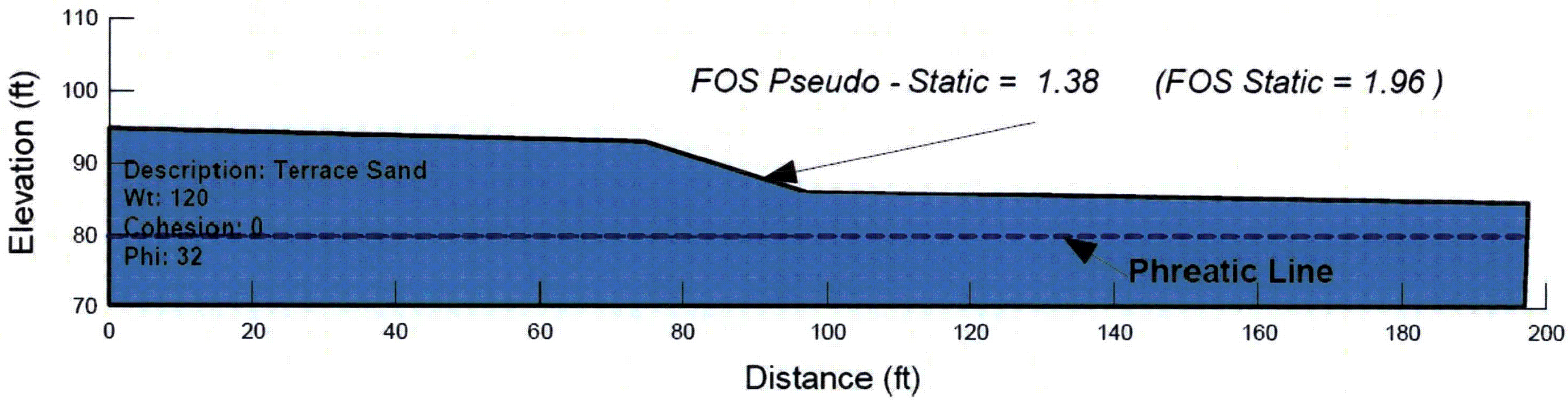


FIGURE 2.5.5-12 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION F

CCNPP UNIT 3 FSAR

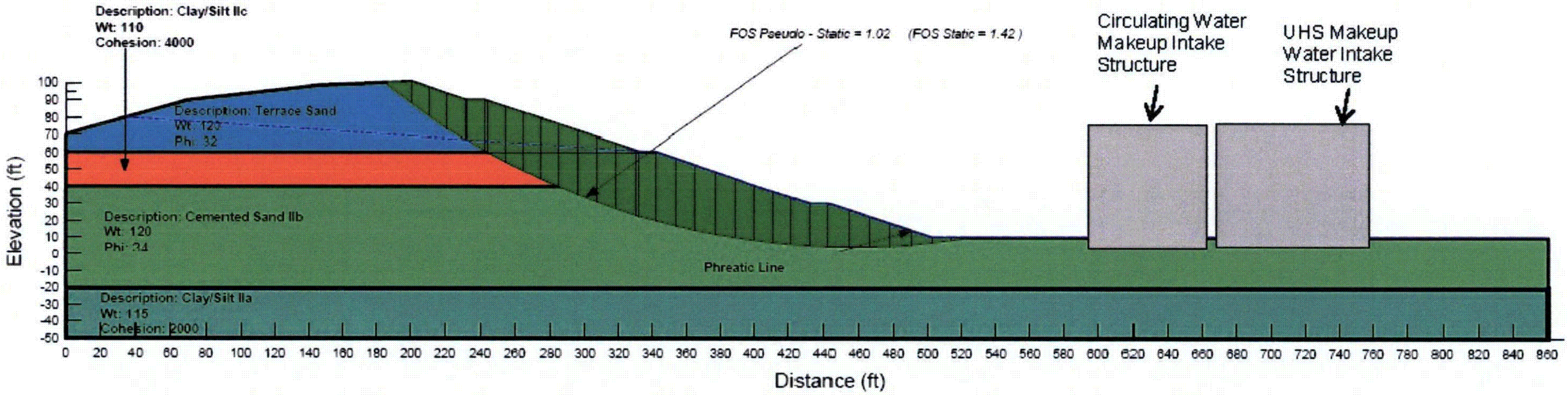


FIGURE 2.5.5-13 **Rev. 0**

STATIC AND PSEUDO–STATIC STABILITY ANALYSES OF SLOPE SECTION G

CCNPP UNIT 3 FSAR

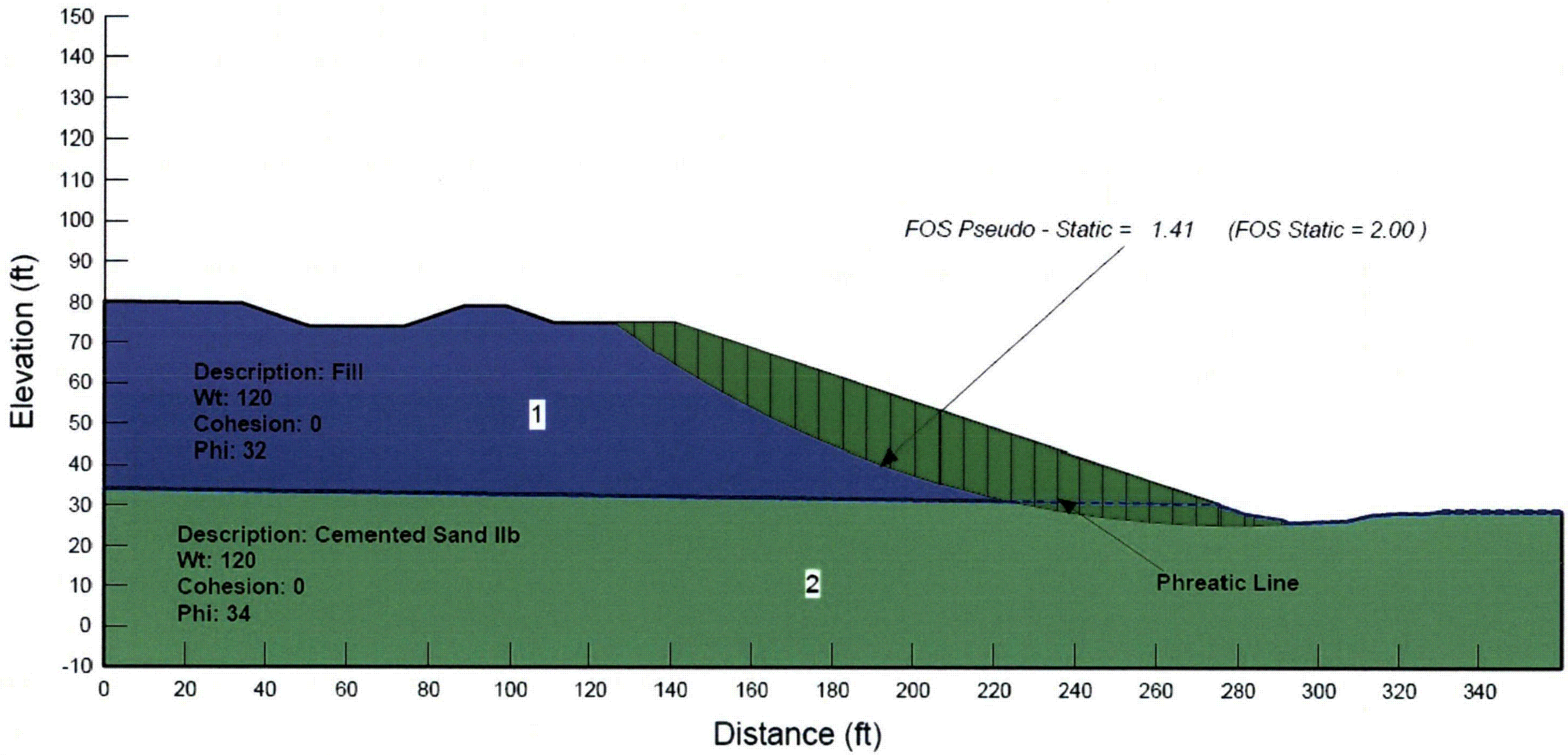


FIGURE 2.5.5-14 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION A (FORCED DEEPER SURFACE)

CCNPP UNIT 3 FSAR

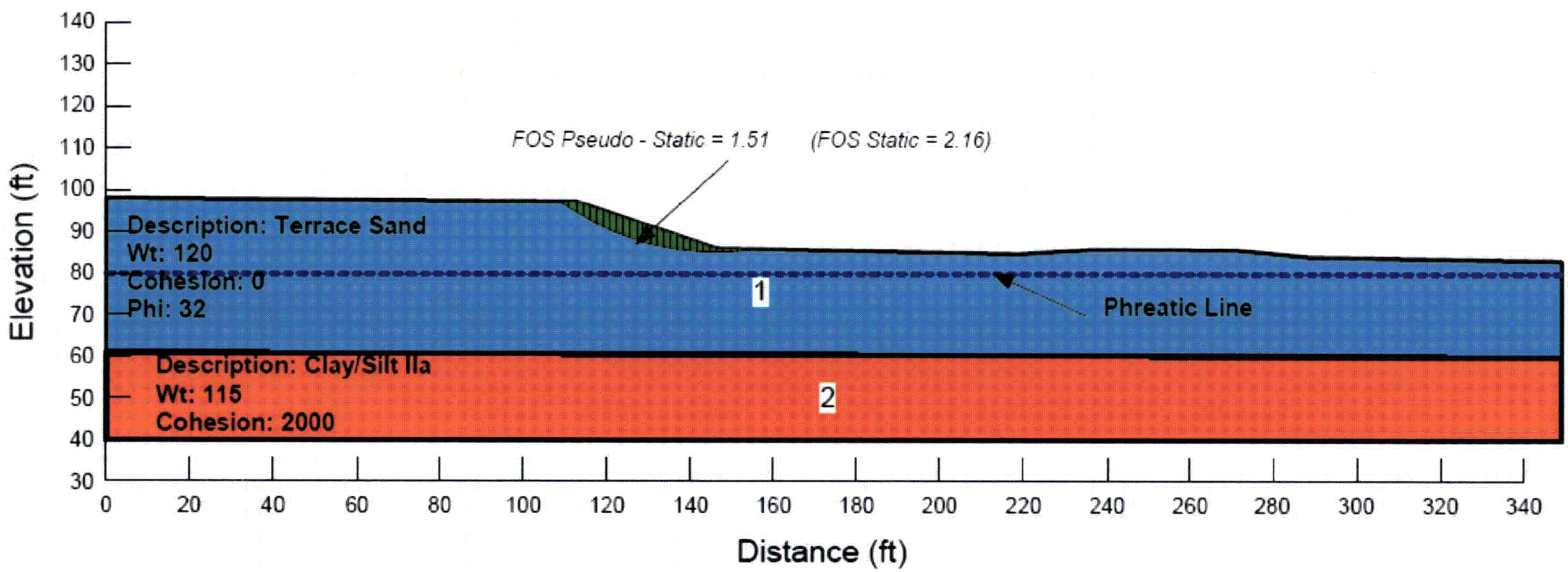


FIGURE 2.5.5-15 **Rev. 0**

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION C (FORCED DEEPER SURFACE)

CCNPP UNIT 3 FSAR

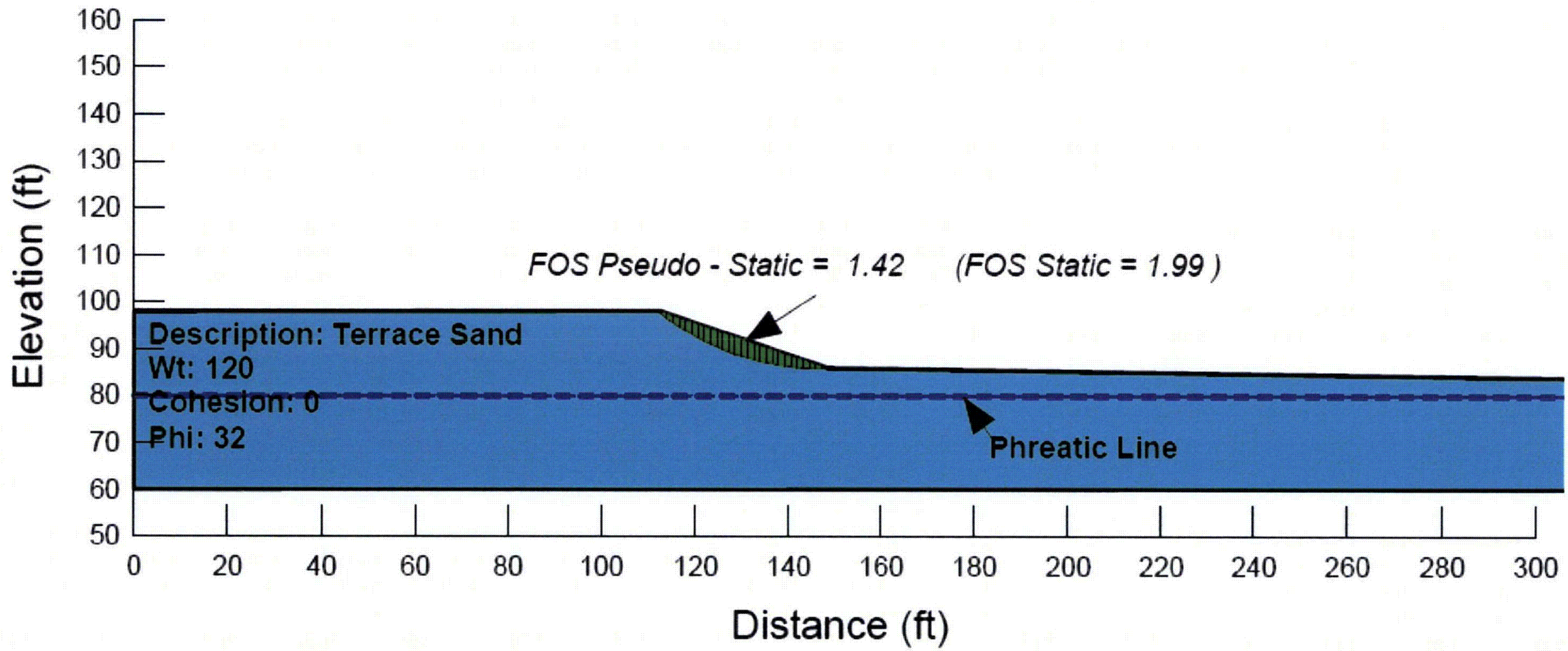


FIGURE 2.5.5-16 Rev. 0

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION D (FORCED DEEPER SURFACE)

CCNPP UNIT 3 FSAR

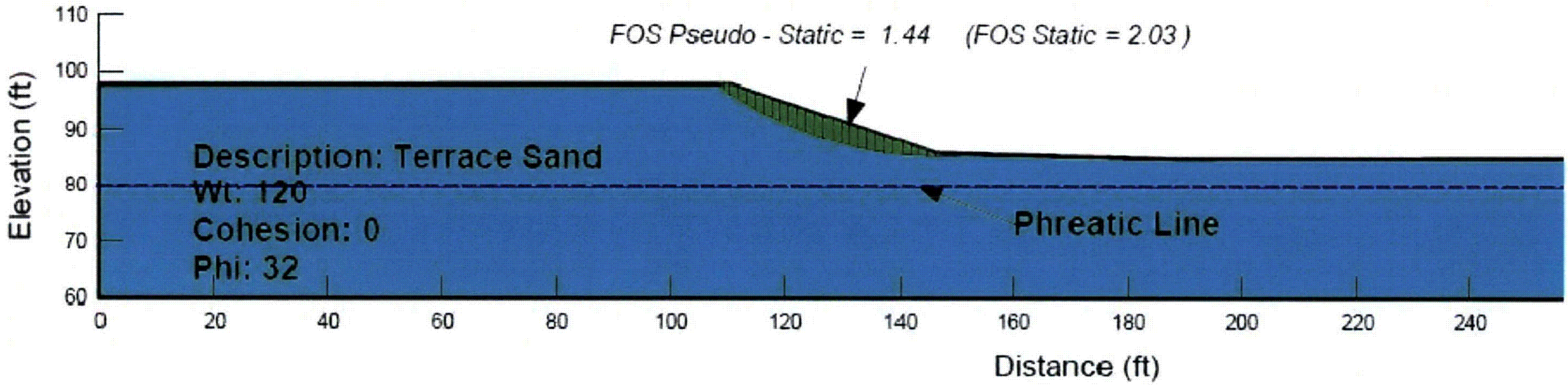


FIGURE 2.5.5-17 Rev. 0

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION E (FORCED DEEPER SURFACE)

CCNPP UNIT 3 FSAR

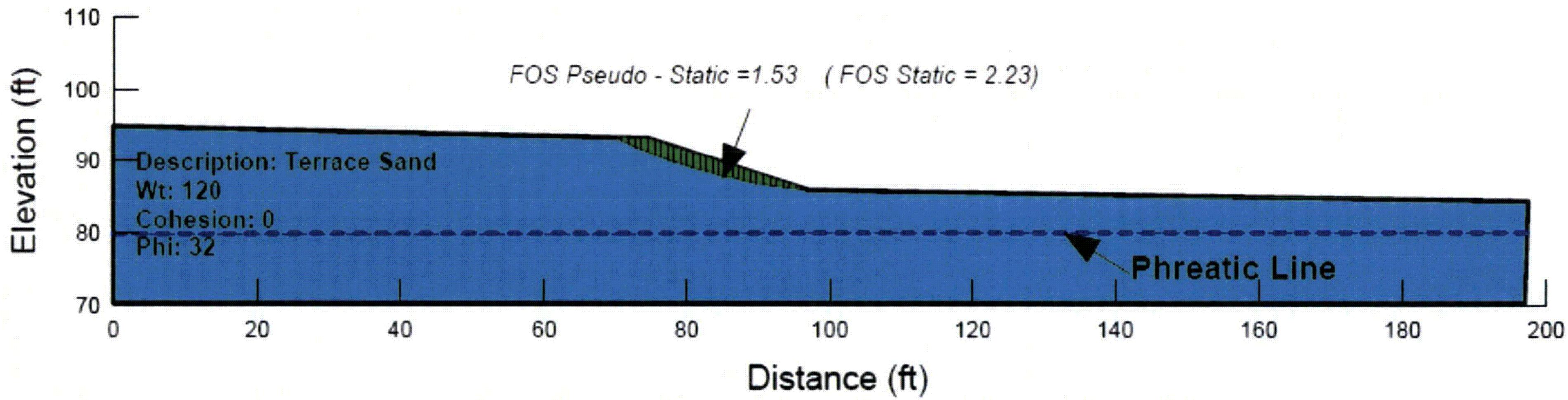


FIGURE 2.5.5-18 Rev. 0

STATIC AND PSEUDO-STATIC STABILITY ANALYSES OF SLOPE SECTION F (FORCED DEEPER SURFACE)

CCNPP UNIT 3 FSAR