Attachment 1

UN#09-427

Enclosure 1

Revision of CCNPP Unit 3 FSAR Sections 2.5.4 and 2.5.5

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

The U.S. EPR FSAR 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 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 CSDRS and the site-specific SSE.

This COL Item is addressed as follows:

{This section addresses site-specific subsurface materials and foundation conditions. 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).

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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

2.5.5.5 References

BGE, 1992. Updated Final Safety Analysis Report, Calvert Cliffs Nuclear Power Plant (Units 1 and 2), Calvert County, Maryland, Docket Numbers 50-317 and 50-318, Baltimore Gas and Electric Company, 1992.

Bishop, 1955. The Use of the Slip Circle in the Stability Analysis of Slopes, A. W. Bishop, Geotechnique, Vol. 5 (1), 7-17, 1955.

Duncan, 1996. State of the art: Limit equilibrium and finite-element analysis of slopes, J. M. Duncan, Journal of Geotechnical Engineering, ASCE, Vol. 122 (7), 577-596, 1996.

Fellenius, 1936. Calculation of Stability of Dams, W. Fellenius, Second Congress on Large Dams Transactions, Vol. 4, 445-462, 1936.

Janbu, 1968. Sl ... ics and Foundation Engineering, The

Krahn, 2004. St ... thodology, First Edition, Revision 1, J. Kra

Morgenstern, 1 ... urfaces, N.R. Morgenstern and V. E. Price, Ge

NRC, 2007. Com ... nts (LWR Edition), Regulatory Guide 1.206, Rev ... h 2007.

Slope/W, 2004. ... v. 6.13, 2004.

Slope/W, 2005. ... 13, A Computer Program for Slope Stability A

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Text pages 2 -1208 through 2-1285

with the attached re-write of sections 2.5.4 and 2.5.5.

2.5.6 REFERENCES

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Table 2.5-25—{Summary of Field Testing Quantities}

Field Test	Standard	Quantity
Test Borings	ASTM D1586/1587	200
Observation Wells	ASTM D5092	47
CPT Soundings	ASTM D5778	74*
Suspension P-S Velocity Logging	EPRI TR-102293	13
Test Pits	N/A	20
Field Electrical Resistivity Arrays	ASTM G57/IEEE 81	4
SPT Hammer Energy Measurements	ASTM D4633	10
Pressuremeter	ASTM D4719	2
Dilatometer	ASTM D6635	2

Note:

* Including additional off-set soundings performed

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Tables 2.5 - 25 through 2.5 - 56
(pages 2 -1318 through 2-1364)

with the attached re-write of sections
2.5.4 and 2.5.5.

Table 2.5-56—{Computed Factors of Safety (FOS) for (Forced) Deeper Slip Surfaces}

Slope Section	Static Analysis				Pseudo-Static Analysis			
	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	---	---	---	---	---	---	---	---
H	---	---	---	---	---	---	---	---

Notes:

Ordinary = Ordinary method

Bishop = Bishop's simplified method

Janbu = Janbu's simplified method

M-P = Morgenstern-Price method

--- indicates no computation

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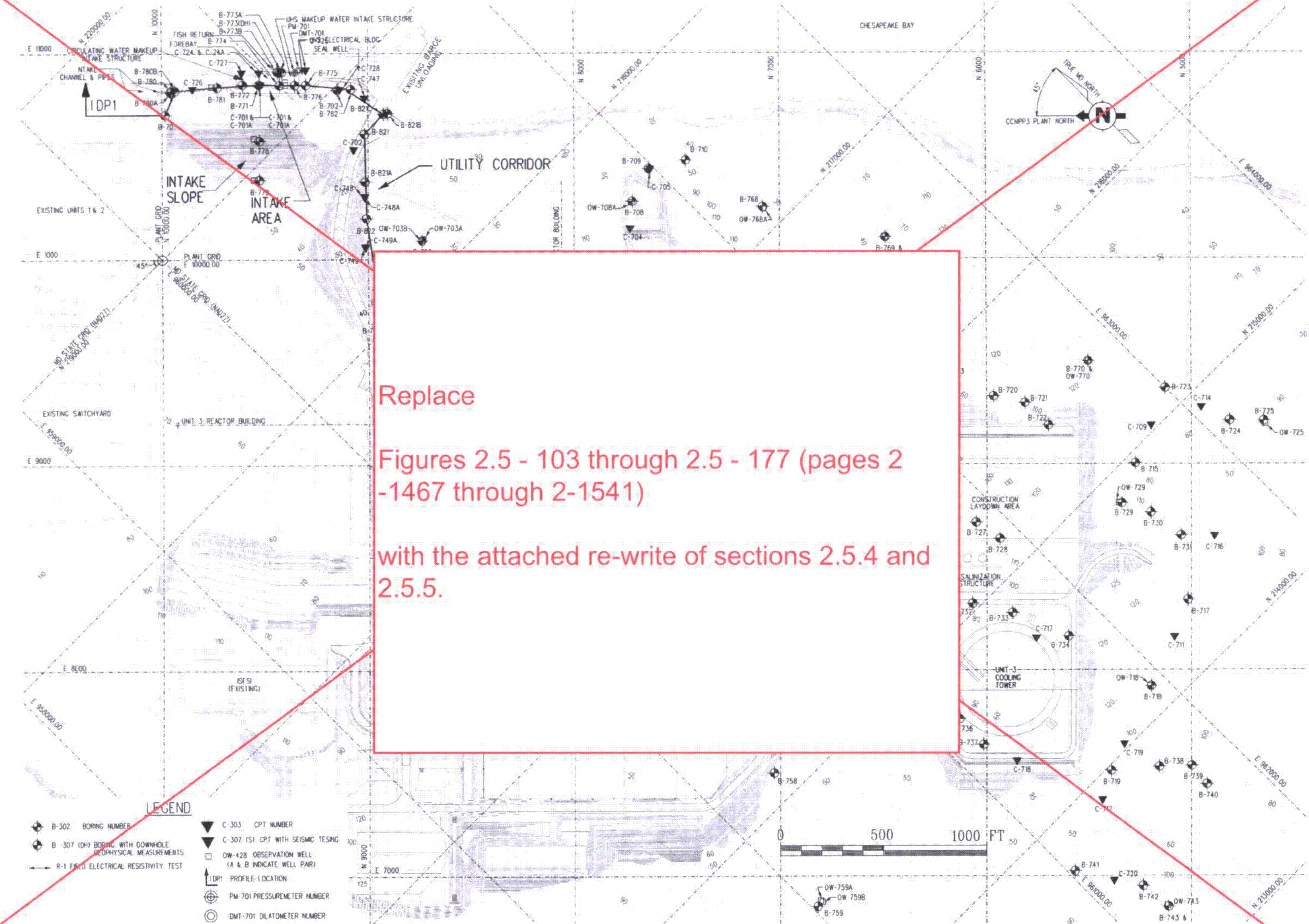
Tables 2.5 - 25 through 2.5 - 56
(pages 2 -1318 through 2-1364)

with the attached re-write of sections 2.5.4 and 2.5.5.

Figure 2.5-103—{Subsurface Investigation Location Plan}



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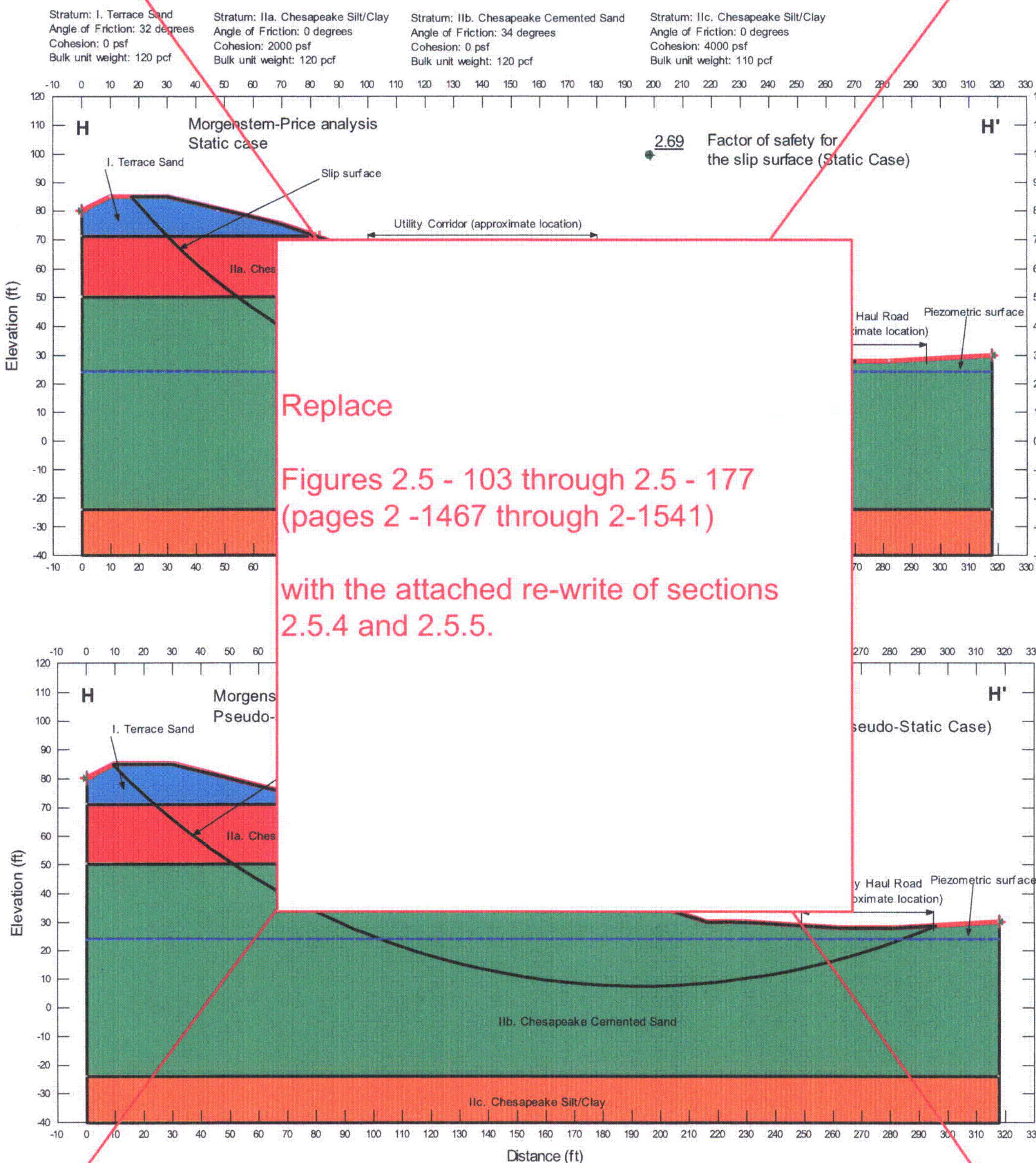
Figures 2.5 - 103 through 2.5 - 177 (pages 2 -1467 through 2-1541)

with the attached re-write of sections 2.5.4 and 2.5.5.

Figure 2.5-177—{Stability Analysis of Slope Section H}



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with the attached re-write of sections 2.5.4 and 2.5.5.

2.5.4 STABILITY OF SUBSURFACE MATERIALS AND FOUNDATIONS

The U.S. EPR FSAR 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 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 CSDRS and the site-specific SSE.

This COL Item is addressed as follows:

{This section addresses site-specific subsurface materials and foundation conditions. 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) (USNRC, 2007a).

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, comparison to CCNPP Units 1 and 2 is limited, but provided when relevant information is available. The information presented in this section is based on results of a site specific subsurface investigation program implemented at the CCNPP Unit 3 site, and evaluation of the collected data, unless indicated otherwise.

Geotechnical and geophysical site investigations have been completed in three stages as follows:

- **Phase I** – Performed in 2006, this is the initial investigation effort and is reported in the Geotechnical Subsurface Investigation Data Reports (Schnabel, 2007a) (Schnabel, 2007b). The investigation includes the boring program for the CCNPP Unit 3 and laboratory testing, including the Resonant Column Torsional Shear (RCTS) tests of the in-situ soils.
- **Phase II** – Performed in 2008, the second phase investigation incorporates the following items:
    - Drilling and sampling of 48 additional Standard Penetration Test (SPT) borings.
    - Installation and Development of 7 additional observation wells.
    - 11 Cone Penetration Tests (CPT) with shear wave velocity measurements.
    - Borehole geophysical including P-S suspension tests in the Intake Area.
    - Two pressuremeter tests.

  Information from the Phase II investigation is presented in several geotechnical and laboratory testing data reports (Schnabel, 2009) (MACTEC, 2009a). The investigation incorporates information from additional borings and additional laboratory testing.
- **Phase III** – Performed in 2009, incorporating the following items:
    - Intake samples laboratory testing, including both static and dynamic RCTS tests.
    - Structural fill static testing, including chemical tests, triaxial tests, grain size tests, and Modified Proctor tests.

- Structural fill dynamic testing (RCTS).

Information from the Phase III investigation is presented in several geotechnical and laboratory testing data reports (MACTEC, 2009b) (MACTEC, 2009c) (MACTEC, 2009d).

The referenced geotechnical reports for the three phases of the investigation are provided in COLA Part 11J: Geotechnical Data Report and COLA Part 11K: Mactec Report.

The CCNNP3 Unit 3 site covers an area of approximately 460 acres. Figure 2.5-103 provides the site utilization plan. The following areas are identified:

1. Powerblock Area – Safety-related facilities in this area include the Reactor Building (RB), Fuel Building (FB) and Safeguard Buildings (Nuclear Island, NI), Essential Service Water Buildings (ESWB), and Emergency Power Generation Buildings (EPGB); other important facilities are the Nuclear Auxiliary Building (NAB), the Radioactive Waste Processing Building (RWPB), the Access Building (AB), and the Turbine Building (TB). The Powerblock Area is enlarged in Figure 2.5-104.

2. Intake Area – Safety-related facilities in this area include the Ultimate Heat Sink Makeup Water Intake Structure (UHS-MWIS), and the Ultimate Heat Sink Electrical Building (UHS-EB), other facilities are the Ultimate Heat Sink Forebay and the Circulating Water Makeup Intake Structure and the Fish Return. The Intake Area is enlarged in Figure 2.5-105.

3. Utility Corridor Area.

4. Construction Laydown Area (CLA).

5. Unit 3 Switchyard.

6. Unit 3 Cooling Basin and Cooling Tower.

The Powerblock, Construction Laydown Area, switchyard and cooling tower and basin are collectively referred to as the CCNPP Unit 3 Area.

The natural topography at the CCNPP site varies throughout the site with differences in elevation up to 100 ft. In the area where CCNPP Unit 3 is planned, ground surface elevations at the time of the exploration ranged from approximately El. 47 ft to El. 121 ft, with an average of 86 ft. The planned elevation (rough grade) in the Powerblock Area ranges from about El. 75 ft to El. 85 ft, with the centerline of Unit 3 at El. 84.7 ft, or approximately El. 85 ft.

In the Intake Area, ground surface elevations at the time of the exploration ranged from approximately El. 7 ft to 12 ft with an average of approximately 9.5 ft. The planned rough grade in the Intake Area is El. 10 ft.

The focus of Section 2.5.4 is the Powerblock Area and the Intake Area. These zones house the safety-related, Seismic- Category I facilities, with the Utility Corridor Area in between. Numerous natural and man-made slopes are identified across the plan. The safety of slopes is addressed in Section 2.5.5.

The subsurface conditions were established from the information contained in the Geotechnical Subsurface Investigation Data Reports from all Phases of the investigation (MACTEC, 2009a) (MACTEC, 2009b) (MACTEC, 2009c) (MACTEC, 2009d) (Schnabel, 2007a)

(Schnabel, 2007b) (Schnabel, 2009). 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 below the ground surface was 138.0 ft at C-302 and 152.4 ft at C-725 (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 the Powerblock Area. Tests identified as 400-series, e.g., B-401 or C-401, are located in an area adjacent to the CCNPP Powerblock Area, hereafter referred to as Construction Laydown Area (CLA). 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, Utility Corridor, Intake Slope, and intake/discharge piping locations. Locations of various test areas are identified in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. The major strata identified from the boring logs are described in detail in the next subsections.

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

The CCNPP Unit 3 is located in the Atlantic Coastal Plain physiographic province. The soils in the site vicinity 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 ft 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.

Detail narrative of the geologic features is provided in Section 2.5.1. 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.}

2.5.4.2 Properties of Subsurface Materials

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.2:

> A COL applicant that references the U.S. EPR design certification will reconcile the site-specific soil properties with those used for design of U.S. EPR Seismic Category I structures and foundations described in Section 3.8.

This COL Item is addressed as follows:

{A comprehensive field investigation and associated laboratory testing has been performed for the CCNPP Unit 3 site. This subsection presents the properties of underlying materials encountered. It is divided into five subsections, as follows.

- Section 2.5.4.2.1 provides an introduction to the soil profile and subsurface conditions,

- Section 2.5.4.2.2 provides a description of the field investigation program, including borings, sampling, and in-situ tests,
- Section 2.5.4.2.3 provides a narrative on the origin of the engineered fill soils samples,
- Section 2.5.4.2.4 provides a description of the laboratory testing program,
- Section 2.5.4.2.5 provides the CCNPP Unit 3 soil properties for analysis and design of foundations.

The description of the field investigation and laboratory testing data incorporate information from all three phases of the investigation (Phase I, II, and III).

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 400 ft of the site soils were the subject of the CCNPP Unit 3 subsurface investigation. In general, the soils at the site can be divided into the following stratigraphic units:

- ***Stratum I: Terrace Sand*** – light brown to brown sand with varying amounts of silt, clay, and/or gravel, sometimes with silt or clay interbedded layers.
- ***Stratum IIa: Chesapeake Clay/Silt*** – light to dark gray clay and/or silt, predominantly clay, with varying amounts of sand.
- ***Stratum IIb: Chesapeake Cemented Sand*** – 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. For the purposes of settlement analysis, Stratum IIb was further divided into three sub-layers. The investigation encountered variation of SPT values both in depth and horizontal distribution. The position of the sub layers beneath the Powerblock Area footprint is variable and this condition needs to be accounted for in a detailed three dimensional settlement analysis. Section 2.5.4.10 provides the details of the settlement model.
- ***Stratum IIc: Chesapeake Clay/Silt*** – gray to greenish gray clay/silt soils, they contain interbedded layers of sandy silt, silty sand, and cemented sands with varying amount of shell fragments.
- ***Stratum III: Nanjemoy Sand*** – primarily dark greenish-gray glauconitic sand with interbedded layers of silt, clay, and cemented sands with varying amounts of shell fragments and varying degrees of cementation.

Figure 2.5-106 provides an idealized soil column for the CCNPP Unit 3 site. The actual depth of layer interfaces varies throughout the site. This condition is revealed by the following subsurface profiles identified on Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105:

Figure 2.5-107	Subsurface profile A-A' at the Powerblock looking east through the NI (local plant coordinates).
Figure 2.5-108	Subsurface profile B-B' at the Powerblock looking east through the EPGBs and NI.

Figure 2.5-109	Subsurface profile C-C' at the Powerblock looking south through the NI and TB.
Figure 2.5-110	Subsurface profile D-D' at the Powerblock looking south through 1EPBG, 3ESWB, and the RWPB.
Figure 2.5-111	Subsurface profile E-E' at the Powerblock looking east through the RWPB, NAB, NI (Safeguard North), 2ESWB and 1ESWB.
Figure 2.5-112	Subsurface profile F-F' at the Intake Area, looking east through the UHS-MWIS and UHS-EB.

The recommendations for soil properties (Section 2.5.4.2.5) to be used for analysis and design of foundation are provided in tabular form for each layer identified. Table 2.5-25 presents the depths and thicknesses of the layers encountered at the site. The data is provided for the entire site and independently for the Powerblock Area and the Intake Area. Information on deeper soils (below 400 ft) was obtained from literature research and it is discussed in Section 2.5.4.7. 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.

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 CLA area (the 300 and 400 series borings) and by a majority of boreholes drilled outside of these two areas including the Intake Slope and the Utility Corridor (the 700 series borings). This stratum was not encountered in low lying areas.

The thickness of Stratum I soils was estimated from the boring logs and CPT logs. In CCNPP Unit 3 area, its thickness with respect to the existing ground surface is shown in Table 2.5-25. The average bottom for Stratum I soils is about El. 62 ft in CCNPP Unit 3 area. Stratum I Terrace Sand does not exist in the Intake Area.

At isolated locations, sandy soils with an appearance similar to Stratum I soils were encountered. Materials that were probably man-made, (hereafter referred to as "fill"), and disturbed soils were encountered, beginning at the existing ground surface at isolated locations at the CCNPP Unit 3 site. These materials were predominantly sand with varying amounts of silt and clay. In the Intake Area (B-701, B-702, B-771 through B-776, B-780 through B-782, and B-821), the depth of these materials varied from approximately 6 to 11 ft below existing grade. They were present at the ground surface and were encountered in 25 borings (B-303, B-309, B-318, B-336, B-340, B-341, B-352, B-356, B-357, B-406, B-409, B-412, B-415, B-419, B-420, B-432, B-437, B-438/A, B-439, B-440, B-701, B-710, B-713, B-768, and B-791). 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 6 ft.

Stratum I soils are 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.

2.5.4.2.1.2 Stratum IIa – Chesapeake Clay/Silt

The Chesapeake Clay/Silt was encountered at all locations except the Intake Area. When present, it was encountered beneath the Terrace Sand, 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 and CPT logs. The thickness of this stratum is presented in Table 2.5-25. Only data from borings that fully penetrated the layer were considered for determination of termination elevations.

The stratum IIa soils were characterized, on average, as medium-high plasticity clays. Their predominant USCS designation was clay of high plasticity and silt of high plasticity (CH and MH); sometimes with silty sand, silty sand to clayey sand, and organic clay. The organic designation was based on laboratory (liquid limit) tests. With less than 1 percent organic matter on average, and observations during sampling, these soils are not considered organic.

2.5.4.2.1.3 Stratum IIb – Chesapeake Cemented Sand

The Chesapeake Cemented Sand stratum was encountered beneath Stratum IIa in all the boreholes except at the Intake Area where it was encountered beneath fill. 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 and termination elevations of this layer are presented in Table 2.5-25. Only data from borings that fully penetrated the layer were considered for determination of termination elevations.

Layer IIb is further subdivided into three sub-layers, as shown by Figure 2.5-106. The layers are denominated Layer 1, Layer 2, and Layer 3. In general, Layer 1 is characterized by standard penetration test (SPT) N-values greater than 20, Layer 2 is characterized by SPT N-values less than 20, and Layer 3 is characterized by SPT N-values greater than 20. Additional information on SPT data is provided in Section 2.5.4.2.2.

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 Phase I investigation samples were classified as organic clay or organic silt, although evidence of high organic content was not present during the field exploration. Organic content testing on three samples indicated an average organic content of 1.4 percent. Eleven Phase II samples from Intake Area borings were tested for organic content. The average organic content in the Intake Area was 1.5 percent. Despite the presence of organic matter in these samples, 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 fines content of 25 percent.

2.5.4.2.1.4 Stratum IIc – Chesapeake Clay/Silt

Underlying the Stratum IIb sands, another Chesapeake Clay/Silt stratum was encountered, although distinctly different from the soils in Stratum IIa. This stratum was encountered in areas and in borings that were sufficiently deep to encounter these soils. 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. The stratum thickness and termination elevations of this Stratum are provided in Table 2.5-25.

For engineering analysis purposes, CCNPP Unit 3 Stratum IIc soils were characterized, on average, as high plasticity clay and silt, with an average PI = 50. 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 observations during sampling, the organic soil designation based on laboratory (Liquid Limit) testing is not representative of these soils, and therefore, they are not considered organic soils. The organic designation may be impacted by the glauconite content in the soils. Organic content testing was performed on 53 Stratum IIc soil samples (all areas). Results indicated organic contents ranging from 1.0 to 9.3 percent with an average of 3.3 percent. The measured values are indicative of the presence of slight organics in these soils.

2.5.4.2.1.5 Stratum III – Nanjemoy 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. It is estimated that 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 El. -315 ft and the termination of the lower clayey portion can be estimated at about El. -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 El. -308 ft and El. -329 ft, respectively.

For engineering analysis purposes, Stratum III soils were characterized, on average, as sand of high plasticity. 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.

2.5.4.2.1.6 Subsurface Materials below 400 Feet

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.

2.5.4.2.2 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" (USNRC, 2003a). References to the industry standards used for field tests completed for the CCNPP Unit 3 subsurface investigation are shown in Table 2.5-26. The details and results of the field investigation are included as COLA Part 11J. 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. A complementary Phase II investigation was performed in 2008 as part of the detailed design of the project, with reference to guidance in Regulatory Guide 1.132 (USNRC, 2003a) to verify subsurface uniformity at locations where coverage was not available in the initial phase of the investigation due to shifting locations of some structures. Results of the additional (Phase II) investigation are presented herein, and in the data report (Schnabel, 2009) (MACTEC, 2009a). Locations of the field tests are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105.

2.5.4.2.2.1 Previous Subsurface Investigations

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 (Bechtel, 1992). 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 El. 70 ft. These soils were estimated to extend to an average El. 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 El. 70 ft and El. -200 ft. These soils were estimated to extend to approximately El. -200 ft based on the CCNPP Unit 3 investigation. CCNPP Units 1 and 2 UFSAR (BGE, 1982) indicates that Eocene deposits lie below El. -200 ft and consist of glauconitic sands. Comparable observations were made on these, and the overlying deposits, from the CCNPP Unit 3 subsurface investigation borings. The 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.

The CCNPP Unit 3 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.

2.5.4.2.2.2 CCNPP Unit 3 Field Investigation

The subsurface investigation program was performed in accordance with the guidance outlined in Regulatory Guide 1.132 (USNRC, 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 (USNRC, 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 Seismic 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 Seismic 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 decreasing the sample spacing in the upper 15 ft and maintaining 5-ft sampling intervals at depths greater than 50 ft, except for the 400-ft borings. Continuous sampling was also performed, and is later described.

Regulatory Guide 1.132 (USNRC, 2003a) provides guidance in selecting the boring depth (dmax) based on a foundation to overburden stress ratio of 10 percent. Regulatory Guide 1.132 (USNRC, 2003a), also indicates that at least one-fourth of the principal borings should penetrate to a depth equal to dmax. 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. Also included were 3 CPT soundings. Borings associated with the Common Basemat extended at least 33 ft below the foundation level. An additional (Phase II) field investigation was completed in 2008 (Schnabel, 2009) (MACTEC, 2009a) in conformance with guidance in Regulatory Guide 1.132.

The current quantity and locations of tests for the combined initial and Phase II investigations, are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. These provide the necessary coverage at the footprint structures, although several of the test locations required relocation during the field investigation to reduce cutting trees, and for accessibility for drilling equipment.

A team consisting of a geologist, a geotechnical engineer, and a member of UniStar 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, (USNRC, 2003a) provides that boreholes with depths greater than about 100 ft should be surveyed for deviation. In lieu of surveying for deviation in boreholes greater than 100 ft, 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, (USNRC, 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 for the initial investigation. However, it was used during the Phase II investigation. X-ray imaging was performed on tube samples selected for RCTS testing.

The Phase I CCNPP Unit 3 subsurface field exploration was performed from April through August 2006; the Phase II exploration was performed from May through December 2008. This work consisted of an extensive investigation to define the subsurface conditions at the project area. The scope of work and investigation methods was 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 200 test borings with SPT sampling and collecting in excess of 275 intact samples (using Shelby push tubes, Osterberg sampler, and Pitcher sampler) to a maximum depth of 403 ft, including 6 borings with continuous SPT samples (B-305, B-409, B-774, B-324, B-417, and B-775), with the first three borings being 150 ft deep each and the last three borings being 100 ft deep each. Note that "continuous sampling" was defined as one SPT sample for every 2.5-ft interval with a one ft distance between each SPT sample. In addition to the 6 continuous borings noted above, 13 borings were continuously sampled between El. 50 ft and El. -20 ft (B-342, B-343, B-344, B-345, B-347, B-348, B-352 through B-357, and B-357A).
- Installing and developing 47 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 74 CPT soundings, including off-set soundings that required pre-drilling to overcome CPT refusal, to a maximum depth of 152 ft, as well as seismic CPT and 37 pore pressure dissipation measurements.
- Conducting 13 P-S Suspension Logging tests to measure dynamic properties.
- Conducting 2-dimensional field electrical resistivity testing along four arrays.

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- 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 13 boreholes.
- Two pressuremeter tests, one in the CCNPP Unit 3 Powerblock Area and another in the Intake Area.
- Two Dilatometer tests, one in the CCNPP Unit 3 Powerblock Area and another in the Intake Area.
- Conducting SPT hammer-rod combination energy measurements on drilling rigs.

Table 2.5-26 provides a summary of the number of field tests performed. 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 COLA Part 11J. 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.

Borings, Standard Penetration Test and Sampling

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, 2006). 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 2.5-27 provides a summary of all test borings performed. The boring locations are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. The boring logs are included in COLA Part 11J. At boring completion, the boreholes were tremie-grouted using cement-bentonite grout.

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, SPT samples were typically obtained at 5 ft intervals. SPT N-values were measured during the sampling and recorded on the boring logs.

SPT N-values in Stratum I soils registered 0 blows/ft (SPT weight of hammer (WOH) or weight of rod (WOR)). 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 CCNPP 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 El. 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 does 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 El. 60 ft. In conclusion, at the CCNPP Unit 3 location, the 5 WOH and WOR results are inconsequential to the stability of Stratum I soils.

The data clearly indicates the need to further subdivide Layer IIb into three sub-stratums. Figure 2.5-113 provides a graphic representation of the SPT distribution in the CCNPP Unit 3 Powerblock Area. Figure 2.5-114 provides equivalent information for the Intake Area. SPT data is summarized in Table 2.5-28. For the Powerblock Area, 177 out of 359 N-values are greater than 63 blows/ft, which is approximately 49 percent of the N-values reported. Out of these 177 values, 153 N-values are 100 blows/ft, which is difficult to clearly portray in scatter plots. The plot does not show clearly these 153 points at a N-value of 100 because the deeper layer overrides those points. Values for analysis and design are provided in Section 2.5.4.2.

Intact 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 2.5-29 provides a summary of undisturbed sampling performed during the subsurface investigation. A total of 375 sample retrievals were attempted. Intact samples are also identified on the boring logs included in COLA Part 11J.

Energy Measurements

Several drill rigs were used for the Phase I and II COL subsurface exploration. SPT hammer energies were measured for each of the drilling rigs used. Energy measurements were made in 10 borings (B-348, B-354, B-356, B-357, B-401, B-403, B-404, B-409, B-744, and B-791). 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 in accordance with ASTM D6066 (ASTM, 2004b). A summary of the measured ETR values for each drill rig is shown in Table 2.5-30 The measured SPT N-values from each boring were adjusted using the appropriate ETR value also shown in Table 2.5-30 for the drill rig used.

The energy measurements were made on the hammer-rod system on drilling rigs used in the subsurface investigation. A Pile Driving Analyzer (PDA) was used to acquire and process the data. 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 90 percent, with an average energy transfer efficiency of 84 percent. Detailed results are presented in COLA Part 11J.

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, 2000b), 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. For the 10-cm^2 base cone, the ASTM D5778 (ASTM, 2005b) specified dimensions for "base diameter," "cone height," and "extension" are a minimum of 34.7 mm, 24 mm, and 2 mm, respectively, compared to the report SGF 1:93E (SGS, 1993) which recommended tolerances of a minimum of 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 74 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 several locations, and often several times at the same sounding. The sounding depths ranged from about 12 ft to 152 ft. Seismic CPT was performed at eight sounding locations. Pore pressure dissipation tests were performed, with 37 results at various depths. Table 2.5-31 provides a summary of CPT locations. The locations are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. The CPT logs, shear wave velocity, and pore pressure dissipation results are contained in COLA Part 11J.

The cone tip resistance, qc, in the Stratum I soils ranged from about 2 to 570 tons per square ft (tsf), with an average of about 120 tsf. The results indicate the qc 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 95 percent, with an average of about 75 percent. Stratum I Terrace Sand was not encountered in CPTs in the Intake Area. In the Utility Corridor it was present at higher elevations.

For Stratum IIa soils, the cone tip resistance values ranged from about 10 to 200 tsf, with an average value of about 50 tsf. Stratum IIa Chesapeake Clay/Silt was not encountered in the Intake Area. The results also indicate a mild increase in tip resistance with depth.

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 qc from the soundings ranged from about 40 to over 600 tsf. The average qc value ranges from 200 to 300 tsf. The results are consistent 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. Stratum IIb Cemented Sand was encountered in the Intake Area with similar but somewhat lower average tip resistance. Average qc value for the Intake Area is approximately 210 tsf. Low SPT N-values and qc 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, qc~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 or no 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, et al., 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 render these zones very porous. In a few borings, loss of drilling fluid was noted, (e.g., in borings B-302, B-309, B-354, B-357, B-357A, B-406, B-414, B-426, B-703, B-710, B-786A and B-790). These 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 B-354, B-357 and B-357A, and 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.

Refusal was also encountered for Stratum IIc soils. Profiles of qc versus elevation are shown in Figure 2.5-115 and Figure 2.5-116 for the Powerblock and Intake Areas respectively. The results suggest relative uniformity in qc 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 profiles in Figure 2.5-113. The qc values for CCNPP Powerblock Area range from about 50 to 100 tsf, with an average of about 75 tsf. Stratum IIc Clay/Silt was encountered in the Intake Area with a slightly lower average tip resistance of 70 tsf.

Observation Wells and Slug Testing

A total of 47 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 2.5-33 provides a summary of the observation well locations and details. The locations are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. 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 47 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 2.5-33 also provides the hydraulic conductivity values. Details on testing are provided in Section 2.4.12. COLA Part 11J 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 2.5-34 provides a summary of the test pit locations. The locations are shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105. COLA Part 11J 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 2.5-35 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 Figure 2.5-103 and Figure 2.5-104. 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 percent 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 percent) 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 contained topographic variation. Finally, IEEE 81 (IEEE, 1983) states that electrodes not be driven into the ground more than 10 percent of the "a" spacing. As discussed above, at some locations electrodes were driven about 2.25 in (or about 12 percent) into the ground. Despite the noted deviations, the collected resistivity values are considered valid and suitable for use.

The results of field resistivity surveys are presented in COLA Part 11J, and summarized in Table 2.5-36.

Suspension P-S Velocity Logging Survey

Borehole geophysical logging was performed in a total of 13 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 COLA Part 11J.

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, B-423, B-773, B-786, and B-821. The measurement at B-786 was performed directly underneath the UHS-MWIS in the Intake Area during the Phase II investigation. 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 COLA Part 11J. Figure 2.5-117 and Figure 2.5-118 present the results of the P-S logging surveys. Figure 2.5-119 provides the test result of the PS log performed in the Intake Area. Overall, the result is consistent with the measurements in the Powerblock Area. Section 2.5.4.2.5.8 and 2.5.4.4 provide the analysis of the P-S data along with the development of the best estimate soil profiles for the Unit 3 Area and the Intake Area.

The suspension P-S velocity logging used a 23-ft 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 high column of soil around the borehole. For quality assurance, analysis is also performed on source-to-receiver data.

Ignoring the measurements above El. 85 ft (approximate finished grade), V_p measurements in Stratum I Terrace Sand ranged from about 850 ft/sec to 5,560 ft/sec, with an increasing trend with depth. V_p measurements in Stratum IIa Chesapeake Clay/Silt ranged from about 3,000 ft/sec to 5,750 ft/sec. V_p measurements in Stratum IIb Chesapeake Cemented Sand ranged from about 2,000 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 V_p values. V_p 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. V_p 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 2.5-37 and Figure 2.5-120) for similar soils. V_p values below about El. 80 ft are typically at or above 5,000 ft/sec; these measurements reflect the saturated condition of the soils below the referenced elevation.

V_s measurements in Stratum IIa Chesapeake Clay/Silt ranged from about 590 ft/sec to 1,430 ft/sec, with typically increasing trend with depth. V_s 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. V_s 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. V_s 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.

The P-S logging results are discussed in detail in Section 2.5.4.4.

Pressuremeter

Pressuremeter testing was performed in pre-drilled boreholes using a cylindrical probe that expanded radially. The deformation of the borehole wall was measured relative to the stress induced by the pressuremeter on the soil. Geotechnical engineers or geologists were on site to inspect the work. One pressuremeter test was performed in the Unit 3 Powerblock Area to a depth of about 360 ft at borehole PM-301. Another pressuremeter test was performed in the Intake Area to a depth of about 150 ft in borehole PM-701. The data are presented in COLA Part 11J. Sixty-seven (67) tests were completed in PM-301 and 29 in PM-701. Almost all of the tests produced useful data, although not all tests could be completely analyzed for all possible parameters. In instances where not all parameters could be determined, this was due to borehole disturbance or uneven expansion of the instrument resulting in less than complete information on the soil.

The pressuremeter used was a digital electronic instrument of the Cambridge design and is a much more sensitive instrument than the Menard type specified by ASTM. The pressuremeter data was analyzed to determine the pressuremeter modulus and limit pressure as determined by ASTM D4719 (ASTM, 2007). Additional analyses were performed to determine the unload/reload modulus which usually included one to three cycles per tests at various strain levels. Strength parameters were determined using modeling techniques. Pressuremeter data has been used as means, among other methodologies, to estimate the elastic modulus for settlement. It is also used to establish the ratio of the Unload/Reload Modulus to the Elastic Modulus.

Table 2.5-38 and Table 2.5-39 provide the data recordings of the pressuremeter tests at PM-301 and PM-701. Figure 2.5-121 shows a graphic representation of the data for the Powerblock and Intake Area in the form of elastic modulus. An average for the site is plotted as references. This information is used as one of the criteria to provide a recommendation for elastic modulus.

Dilatometer

An in-situ penetration and expansion test with a steel dilatometer blade with a sharp cutting edge was incrementally forced into the soil in a generally vertical orientation. At a specified depth a flat circular, metallic membrane is expanded into the surrounding soil. Inspected by a geotechnical engineer or geologist, the soil deformation is measured relative to the stress induced on the soil by the expanding membrane. One dilatometer test was performed in the Powerblock Area to a depth of about 350 ft in boring B-301. Another dilatometer test was performed in the Intake Area to a depth of about 150 ft at boring B-701. Due to the large amount of data, the results of the tests are included only in COLA Part 11J.

2.5.4.2.3 Backfill Investigation

During the Phase III investigation, a backfill characterization study was conducted. Structural fill has been identified and the material sampled was sent to the laboratory to establish their static, chemical, and dynamic properties. The results are evaluated to verify that the candidate backfill materials meet the design requirements for structural fill. The structural fill for CCNPP Unit 3 is sound, durable, well graded sand or sand and gravel, with a maximum 25 percent fines content, and free of organic matter, trash, and other deleterious materials. Backfill and related topics are further addressed in Section 2.5.4.5. It is estimated that about 2 million cubic yards of structural backfill are required.

The field sampling campaign was performed as follows:

- Batch 1: sampling of six buckets from Vulcan Quarry in Havre de Grace, Maryland was performed in September of 2008. Sample testing directive to laboratory was performed on unblended samples.
- Batch 2: sampling of six buckets from Vulcan Quarry. Sample testing directive to laboratory was performed on blended samples. Sample testing directive to laboratory was performed on composite samples.
- Batch 3: eight buckets of CR6, eight buckets of GAB, and six buckets of coarse aggregate- 57 sampled from the Vulcan Quarry on December, 2008. Sample testing directive to laboratory was performed on composite samples.
- Batch 4: seventeen buckets of CR6, GAB, and coarse aggregate-57 sampled from the Vulcan Quarry on March, 2009. Sample testing directive to laboratory was performed on composite samples. Batch 4 was used for Resonant Column Torsional Shear Testing.

2.5.4.2.4 Laboratory Testing Program

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

The detailed results of all laboratory tests performed as part of the subsurface investigation is provided in the following reports:

- Geotechnical Subsurface Investigation Data Report (Schnabel, 2007a), with Phase I laboratory testing program.
- Geotechnical Subsurface Investigation Data Report (Schnabel, 2007b).
- Reconciliation of EPRI and RCTS Results Calvert Cliffs Nuclear Power Plant Unit 3 (Bechtel, 2007), with the RCTS data and analysis for the Powerblock Area.
- Revised Laboratory Testing Results, Rev.2 (MACTEC, 2009a).
- Structural Fill Static Laboratory Testing Results, Rev. 1 (MACTEC, 2009b).
- Structural Fill Dynamic Laboratory Testing Results, Rev.1 (MACTEC, 2009c).

- Intake Samples Laboratory Testing Results, Rev. 1 (MACTEC, 2009d).

The referenced reports are included in COLA Part 11J and COLA Part 11K.

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 in 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.

Testing of index properties included the following items:

- Soil classification,
- Water content,
- Grain size (sieve and hydrometer),
- Atterberg limits,
- Organic content,
- Specific gravity,
- Unit weight.

Chemical tests included:

- pH,
- Chloride content,
- Sulfate content.

Performance and strength tests under static conditions included:

- Consolidation,
- Unconfined compression (UC),
- Unconsolidated-undrained triaxial compression with pore pressure measurement (UU),
- Consolidated-undrained triaxial compression with pore pressure measurement (CU-Bar),
- Direct shear (DS),
- Modified Proctor compaction (Moisture–Density),
- California Bearing Ratio (CBR).

Performance and strength tests under dynamic conditions included:

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- Resonant Column Torsional Shear (RCTS) tests.

Unit weight is also obtained from direct volume/mass measurements from miscellaneous tests. The number of tests performed is provided in Table 2.5-40.

Regulatory Guide 1.138 (USNRC, 2003b) provides guidance for laboratory testing procedures for certain specific tests, including related references. 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.

The soil and rock laboratory tests listed in Regulatory Guide 1.138 (USNRC, 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 were 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.

The following subsections present a summary of the most relevant laboratory testing data. A recommendation of soil properties for use of foundation analysis and design is provided in Section 2.5.4.2.5. The complete set of laboratory test results is included in COLA Part 11J and COLA Part 11K. References are made to property data tables. Each table presents a line item for each of the soil layers and one line item for backfill.

2.5.4.2.4.1 Index Testing

Laboratory index tests and testing for determination of engineering properties were performed on selected samples. Laboratory test quantities are summarized in Table 2.5-40. 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.

Values of index testing are provided in Table 2.5-41 and Table 2.5-42. Figure 2.5-122 and Figure 2.5-123 provide a plot of Moisture Content and Atterberg limits as a function of elevation for the Powerblock and Intake Area respectively. Figure 2.5-124 and Figure 2.5-125 provide the plasticity chart for the Powerblock Area and Intake Area respectively.

2.5.4.2.4.2 Chemical Testing

Chemical testing consisted of pH, chloride, and sulfate tests, performed on selected soil samples collected during the COL exploration. The pH tests were performed on samples in both calcium chloride and deionized water. Seventy-seven sets of chemical tests were performed on soil samples collected from depths ranging from the ground surface to 104 ft below the ground surface. The test results are provided in the data report and summarized in Table 2.5-43.

2.5.4.2.4.3 Performance and Strength Tests under Static Conditions

Summary data of performance and strength properties are presented in the following tables:

- Table 2.5-44 and Table 2.5-45 provide the summary of the consolidation test results for the Powerblock Area and Intake Area respectively.

- Table 2.5-46 and Table 2.5-47 provide the summary of shear strength test results for the Powerblock Area and Intake Area respectively; the tests include unconsolidated-undrained triaxial, consolidated-drained triaxial, unconfined compression and direct shear.

- Table 2.5-48 provides the results of Modified Proctor tests for the samples tested for backfill. These samples have been selected based on performance under compaction tests and RCTS tests (Section 2.5.4.2.4.4).

2.5.4.2.4.4 Resonant Column Torsional Shear Tests (RCTS)

Testing was performed on resonant column and torsional shear (RCTS) equipment to measure the material properties (shear modulus and material damping in shear) of soil specimens. The RCTS equipment used is of the fixed-free type, with the bottom of the specimen fixed and shear stress applied to the top. Both the resonant column (RC) and torsional shear (TS) tests were performed in a sequential series on the same specimen over a shearing strain range from about 10^{-4} percent to about 1 percent, depending upon specimen stiffness. RCTS testing was performed on each soil specimen at selected confining pressures of 0.25, 0.5, 1, 2, and 4 times the estimated effective stress. Testing at each successive stage (i.e., confining pressure condition) occurred after the specimens were allowed to consolidate at each pressure step. At each level of shear strain amplitude, the shear modulus and material damping ratio were determined.

EPRI curves were fitted to the data to provide the recommendation (EPRI, 1990). For the Powerblock Area, the EPRI curve fitting is provided in the report "Reconciliation of EPRI and RCTS Results, Calvert Cliffs Nuclear Power Plant Unit 3" (Bechtel, 2007), and is included as COLA Part 11J. Section 2.5.4.2.5 provides a detailed discussion about the criteria for selection of strain dependant property curves based on generic curves and site specific laboratory information.

RCTS testing was performed for the samples in the Powerblock Area, the Intake Area, and Backfill. Table 2.5-49 provides a list of the RCTS samples tested and their index properties. The following samples were used for RCTS testing. The associated figure shows the results for that specific sample.

- Powerblock Area
  - B-437-6 (13.5'), Figure 2.5-126
  - B-301-10 (33.5'), Figure 2.5-127
  - B-305-17 (39.5'), Figure 2.5-128
  - B-404-14 (52.0'), Figure 2.5-129
  - B-401-31 (138.5'), Figure 2.5-130
  - B-401-67 (348.5'), Figure 2.5-131
  - B-401-48 (228.5'), Figure 2.5-132
  - B-301-78 (385.2'), Figure 2.5-133
  - B-306-17 (68.0'), Figure 2.5-134
  - B-409-15 (35.0'), Figure 2.5-135
  - B-404-22 (83.5'), Figure 2.5-136

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    - B-401-42 (198.5'), Figure 2.5-137
    - B-409-39 (95.0'), Figure 2.5-138

- Intake Area
    - B-773-2 (15.9'), Figure 2.5-139
    - B-773-3 (27.0'), Figure 2.5-140
    - B-773-4 (37.0'), Figure 2.5-141
    - B-773-5 (47.0'), Figure 2.5-142
    - B-773-6 (57.0'), Figure 2.5-143
    - B-773-7 (66.1'), Figure 2.5-144
    - B-773-9 (87.0'), Figure 2.5-145
    - B-773-11 (107.0'), Figure 2.5-146
    - B-773-13 (127.0'), Figure 2.5-147
    - B-773-15 (147.0'), Figure 2.5-148

- Backfill
    - CR6 Composite (Bulk), Figure 2.5-149
    - GAB Composite (Bulk), Figure 2.5-150
    - CR6 Vulcan Average (Bulk), Figure 2.5-151

The backfill low strain RCTS test shear wave velocity measurements are used to aid in the development of the best estimate velocity profiles. These measurements are provided in Table 2.5-50. The confining pressures in the test ranged from 0.5 ksf to 17.3 ksf. Since the backfill will be placed near the surface in the uppermost 43.5 feet, and an increase in confining pressures is expected from building facilities, the relevant results correspond to the confining pressures reported in Table 2.5-50.

2.5.4.2.5 Soil Properties for Foundation Analysis and Design

Sections 2.5.4.2.2, 2.5.4.2.3, and 2.5.4.2.4 provide a comprehensive summary of the results from field and laboratory testing. This section uses the data retrieved and develops soil properties to be used for foundation analysis and design. The selection of properties takes into account the wealth of information generated from the field and laboratory, and is developed based on simplified soil profiles that are derived with the use of common geotechnical engineering principles and engineering judgment.

Figure 2.5-106 shows the general soil profile for the CCNPP Unit 3 Site. The profile is applicable throughout the site, though at the Intake Area, due to the difference in elevation and proximity to the shoreline, the Stratum I Terrace Sand and Stratum IIa Chesapeake Clay/Silt are not present. Instead, a man made fill sits on top of Layer IIb Chesapeake Cemented Sand. Figure 2.5-112 shows the conditions at the Intake Area.

The soil properties provided in this section are applicable to the soil layers portrayed by Figure 2.5-106. The settlement analysis for the CCNPP3 Unit 3 Site accounts for a three-dimensional representation of the subsurface conditions. Details of the settlement analysis are provided in Section 2.5.4.10.

2.5.4.2.5.1 General Classification and Index Properties

Stratum I soils are 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.

Stratum IIa soils are characterized as medium-high plasticity clays. Their predominant USCS designation was clay of high plasticity and silt of high plasticity (CH and MH); sometimes with silty sand, silty sand to clayey sand, and organic clay. The organic designation was based on laboratory (liquid limit) tests. With less than 1 percent organic matter on average, and observations during sampling, these soils are not considered organic.

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).

Stratum IIc soils are characterized as high plasticity clay and silt, with an average PI = 50. Their predominant USCS designation was clay of high plasticity and silt of high plasticity (CH and MH), however, sometimes silty sand, clay, and organic clay classifications were indicated. Based on observations during sampling, the organic soil designation based on laboratory (Liquid Limit) testing is not representative of these soils, and therefore, they are not considered organic soils.

Stratum III soils are characterized as sand of high plasticity. 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.

Table 2.5-51 provides the USCS classification of soils and index properties for each stratum. Unit weights were determined based on numerous unit weight tests performed on specimens during different types of tests such as unit weight, triaxial, RCTS. The USCS classification is based on the predominant classification of tested samples.

2.5.4.2.5.2 Chemical Properties

Table 2.5-43 provides the data obtained for the CCNPP Unit 3 site. Guidelines for interpretation of chemical test results are provided in Table 2.5-52, based on the following consensus standards, API Recommended Practice 651 (API, 2007), Reinforced Soil Structures (FHWA, 1990), Standard Specification for Portland Cement (ASTM 2005b), Manual of Concrete Practice (ACI, 1994), and Standard Specification for Blended Hydraulic Cement (ASTM, C595). From the average values of available results shown in Table 2.5-43, the field resistivity surveys in table 2.5-12, and guidelines in Table 2.5-52, 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 typically being 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." Few chemical test results are available from Stratum IIc; however, that should be of no special importance because no Seismic 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. 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, based on the available information, Seismic Category I structures (or piping) may encounter Stratum IIc soils in the Intake Area. Nevertheless, all natural soils at the site will be considered aggressive to concrete, requiring protection if placed within these soils.

2.5.4.2.5.3 Performance Properties Under Static Conditions

The required performance properties under static conditions are the following:

- C_r - Recompression index,
- C_c - Compression index,
- e_o - Initial void ratio,
- p'_c - Preconsolidation pressure,
- OCR- Overconsolidation ratio,
- c_v - Coefficient of consolidation.
- k - Permeability (hydraulic conductivity),

The selected values for the consolidation properties are based on average parameters obtained from laboratory testing. Permeability is obtained from well field tests and development and calibration of hydrogeologic models. Details of the tests and models are provided in Sections 2.4.12 and 2.4.13. Hydraulic conductivity for backfill is based on laboratory results of tests performed on bulk samples. Table 2.5-53 provides the soil performance properties for each stratum.

2.5.4.2.5.4 Strength Properties Under Static Conditions

The required strength properties under static conditions are the following:

- N - Standard Penetration Test (SPT) Resistance (N);
- c' - Cohesion under drained conditions;

- ϕ' - Friction angle under drained conditions;
- c - Cohesion under undrained conditions;
- ϕ - Friction angle under undrained conditions;
- s_u - Undrained shear strength.

Table 2.5-28 provides the SPT test data. The average SPT N corrected values are used.

The shear strength parameters are based on laboratory testing data. Table 2.5-54 provides the strength properties for each stratum.

2.5.4.2.5.5 Elastic Properties Under Static Loading

The required elastic properties of soil under static loading are the following:

- E - Elastic modulus (large strain).
- $E_{u/r}$ - Unload/Reload Elastic modulus.
- $E_{u/r}/E$- Ratio of to unload/reload Elastic modulus to Elastic modulus .
- G - Shear modulus (large strain).
- ν - Poisson's ratio.

The elastic moduli significantly impact settlement estimates and therefore numerous methods have been applied to estimate these parameters. The Shear modulus (G) and elastic modulus (E) are estimated for each soil strata using the following three criteria:

1. Geophysical test results: Shear wave velocities (V_s), P-wave velocities (V_p), and Poisson's ratios from borehole surveys are used to estimate the shear modulus (G) and Elastic modulus (E) at depth intervals between 1.6 ft and 1.7 ft below the ground surface. The geophysical survey data are grouped based on the soil strata. Average G and E values and their corresponding standard deviations of each soil layer are estimated. The G and E values estimated based on the geophysical tests correspond to very low strain values; therefore, they are reduced to account for the material's strain softening due to higher strains. The moduli are determined from elasticity theory equations:

$$G = \rho V_s^2$$
$$E = 2G(1+\nu)$$

   The value of the static Poisson Ratio is adopted from typical values reported in the literature (Salgado, 2008).

2. Pressuremeter testing data obtained from two borehole locations are used to calculate the shear modulus (G) and elastic modulus (E) for each soil layer. Results from Pressuremeter testing correspond to high strain values, therefore, it is expected that the elastic modulus values fall in the lower bound range.

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3. Elastic modulus is calculated using different correlations as a function of corrected SPT N-values and undrained shear strength (s_u):

$E = 18N_{60}$	Coarse grained Materials (Davie, et al.,1988)
$E = \beta_0\sqrt{OCR} + \beta_1 N_{60}$	Coarse grained materials (Coduto, 2001)
$E = 450s_u$	Fine grained materials (Davie, et al., 1988)
$E = 2G(1+\nu), G = s_u$	Fine grained materials (Senapathy, et al., 2001)

Table 2.5-55 provides the estimates of elastic modulus using the previously listed criteria.

The unload/reload modulus ($E_{u/r}$) is required for the estimation of heave and of settlement between excavation and reload. The pressuremeter test data were used to estimate the ratio of unload/reload modulus. The data provided by Table 2.5-38 and Table 2.5-39 indicate that the unload/reload values are consistently above 3.0, with average values above 4.0 and in many instances higher than 6.0. Due to the uncertainty involved in settlement computations and the uncertainty in relating pressuremeter data to actual field conditions it is prudent to adopt a conservative approach. Therefore, the maximum value for the $E_{u/r}/E$ ratio adopted is 3.0 except when the minimum recorded value for a given layer is higher than 3.0. In those instances the minimum value of $E_{u/r}/E$ is adopted. Table 2.5-56 shows the minimum, average, and maximum values of the $E_{u/r}/E$ ratio reported from pressuremeter testing. Table 2.5-57 provides the static elastic properties for each stratum.

By establishing a limit of 3.0, the previous criterion is conservative for the estimation of total settlements. By using a larger value than 3.0 whenever $(E_{u/r}/E)_{min}$ is larger, the previous criterion is conservative for the estimation of tilt. This approach accounts for the asymmetric topographic conditions and the effect that they have on the unloading throughout the footprint of the foundation. Additional explanation is provided in the settlement analysis in Section 2.5.4.10.

2.5.4.2.5.6 Earth Pressure Coefficients

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 on the following relationships (Lambe, et al., 1969):

Active Earth Pressure Coefficient: $K_a = \tan^2\left(45 - \frac{\Phi'}{2}\right)$

Passive Earth Pressure Coefficient: $K_p = \tan^2\left(45 + \frac{\Phi'}{2}\right)$

At Rest Earth Pressure Coefficient: $K_0 = 1 - \sin(\Phi')$

The values for earth pressure coefficients for each stratum are provided in Table 2.5-58.

2.5.4.2.5.7 Sliding Coefficient

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), the sliding coefficient, tangent δ, for each stratum is provided in Table 2.5-58.

2.5.4.2.5.8 Low Strain Dynamic Properties

The low strain dynamic properties are the basis to develop the Best Estimate soil profile for the purposes of site amplification analysis. The following properties are discussed:

- γ - Moist unit weight;
- G_o - Low strain shear modulus;
- V_S - Shear wave velocity;
- V_p - Compression wave velocity;
- ν - Poisson's Ratio;

The moist unit weight is obtained directly from the index properties. Based on all 10 suspension P-S velocity measurements, an average V_s profile was estimated for the upper 400 ft. Poisson's ratio values were determined based on the V_p and V_s measurements. The measurement of dynamic properties reflects the conditions for the approximately upper 400 ft of the site, or to about El. -317 ft. Information on deeper soils, as well as bedrock, was obtained from the available literature.

Shear wave velocity measurements were made using a seismic cone at ten soundings (C-301, C-304, C-307, C-308, C-401, C-404, C-407, C-408, C-724, and C-725). The measurements were made at 3.3 ft (1 m) 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. 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 COLA Part 11J and COLA Part 11K.

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. It is also well established that the P-S logging technique is specifically designed to measure wave velocities and is a superior measurement technique when compared to the CPT.

The best estimate of the shear and compression shear wave velocity profiles are presented by the following four figures:

1. Figure 2.5-166, showing the best estimate velocity profiles in the Powerblock Area;
2. Figure 2.5-167, showing the best estimate velocity profiles in the Powerblock Area, after placement of fill;
3. Figure 2.5-168, showing the best estimate velocity profiles in the Intake Area;
4. Figure 2.5-169, showing the best estimate velocity profiles in the Intake Area, after placement of fill;

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In these four figures, 0 depth corresponds to site grade, El 83 ft.

The following apply to the best estimate profiles and the previous figures:

- The figures indicate the position of the groundwater. For the Powerblock Area, the groundwater level at the site has an approximate depth of 16 ft. Once construction is finalized, due to new drainage patterns the expected depth of the groundwater is 30 ft. A detailed discussion related to groundwater is provided in Section 2.4.12.

- The shear wave velocity of the fill has been estimated by adjusting the low strain dynamic properties measured by the RCTS tests to the field conditions. Table 2.5-50 provides the RCTS test results for the range of confining pressures that will prevail after backfill placement. Based on the results, a three-step velocity profile is proposed, as shown by the four previously listed figures. The shear wave velocity for the the backfill below the EPGB is 900 fps. This value is below the 1,000 fps specified in the U.S. EPR FSAR. This constitutes a departure. The lower shear wave velocity will be used in the soil-structure interaction analysis in section 3.7.

- For the Intake Area, the best estimate is based in the P-S logging measurement of boring B-773. The shear wave velocity in Stratum II-C, Chesapeake Clay/Silt is consistent with the measurements at the Powerblock Area, though slightly lower with a value of 1150 fps, as opposed to 1250 fps. The measurement at B-773 reached a depth of approximately 150 ft. The values for deeper strata are taken from the best estimate profile in the Powerblock Area.

- The development of the deep soil column, location of bedrock, and location of the 9,200 fps horizon was based on the study of geologic conditions and deep well exploration records in the site vicinity. A detailed discussion with the basis for parameter selection is provided in the following paragraphs.

To develop the deep soil velocity profile, 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. 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). Top of rock elevation at the CCNPP site is estimated, and adopted, at approximately El. -2,446 ft which corresponds to a depth of about 2,531 ft. 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 Unit 3 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 2.5-152. 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 V_s 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 Figure 2.5-153 and Figure 2.5-154.

The bottom of the measured V_s profile in the upper 400 ft fits well with the Chester data for which a soil's Poisson's ratio = 0.4 was used, whereas, in the case of Lexington Park data, the bottom of the measured 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 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 V_s profile measured with suspension logging at comparable elevations. A similarly good fit is obtained for the Lexington Park data when a Poisson's ratio of 0.45 is used.

Although geologically the soils at the Chester and Lexington Park sites are quite comparable, 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 intervals 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 2.5-155. 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. 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 2.5-156.

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 Figure 2.5-153 and Figure 2.5-154 (near the soil-rock interface) are enlarged for clarity and are shown in Figure 2.5-157 and Figure 2.5-158 for the Poisson's ratios shown.

A comparison of the shear wave velocity 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 2.5-157. 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 2.5-159, showing the shear wave velocity profile versus elevation in bedrock. From this figure, starting at V_s 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 2.5-159 was adopted to define the velocity profile for the rock. The recommended velocity profile for bedrock begins with V_s = 5,000 ft/sec at the soil-rock interface, as indicated from the sonic data, transitioning to 9,200 ft/sec in the steps shown in Figure 2.5-159. The top of rock elevation was adjusted to conform to the estimated rock elevation for the CCNPP Unit 3 site, or El. -2,446 ft. (Refer to Section 2.5.1).

Accordingly, based on measured data in the upper 400 ft and data obtained from available literature in areas surrounding the CCNPP site, the shear wave velocity profile in soils at the CCNPP Unit 3 site is shown in Figure 2.5-166 and Figure 2.5-167. For the Intake Area the profiles are provided in Figure 2.5-168 and Figure 2.5-169. The profiles in the figures are considered as the design shear wave velocity profiles. Tabular data related to velocity profiles is provided in Table 2.5-59 and Table 2.5-60 for the Powerblock and Intake Area respectively.

2.5.4.2.5.9 Strain Dependant Properties

The strain dependant properties for the CCNPP3 project are developed by fitting generic curves to the site specific data reported by RCTS tests. EPRI curves from EPRI TR-102293 were used as generic curves (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.

A detailed description of the RCTS curve fitting process is provided in the report "Reconciliation of EPRI and RCTS Results, Calvert Cliffs Nuclear Power Plant Unit 3" (Bechtel, 2007), and is included as COLA Part 11J.

The strain dependent properties are first developed for the Powerblock Area. After fitting EPRI curves to the RCTS data in the Powerblock, the resulting curves were used as a starting point to fit the data of the Intake Area and develop properties for that zone. The damping ratio curves are truncated at 15 percent, consistent with the maximum damping values that will be used for the site response analysis. The backfill RCTS results were used to develop strain dependent properties following the same fitting approach and using EPRI curves for granular soils. The following tables and figures provide the strain dependant properties for the CCNPP project:

- Table 2.5-61 and Figure 2.5-170 provide the properties for the Powerblock Area.
- Table 2.5-62 and Figure 2.5-171 provide the properties for the Intake Area.
- Table 2.5-63 and Figure 2.5-172 provide the properties for Backfill.

Bedrock Properties

The two velocity profiles for the Chester and Lexington Park sites (Figure 2.5-157 and Figure 2.5-158), indicate the presence of "hard" rock (identified with V_s = 9,200 ft/sec). 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 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 rock unit weight was estimated from the available literature (Deere, et al., 1996), as 162 pcf.

2.5.4.3 Foundation Interfaces

Subsurface profiles (at the corresponding locations shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105) depicting the inferred subsurface Stratigraphy with the location of the plant's facilities are presented in the following figures:

- Subsurface and excavation profile Powerblock Area A-A': Figure 2.5-160.
- Subsurface and excavation profile Powerblock Area B-B': Figure 2.5-161.

- Subsurface and excavation profile Powerblock Area C-C′: Figure 2.5-162.
- Subsurface and excavation profile Powerblock Area D-D′: Figure 2.5-163.
- Subsurface and excavation profile Powerblock Area E-E′: Figure 2.5-164.
- Subsurface and excavation profile Powerblock Area F-F′: Figure 2.5-165.

Excavation and dewatering issues are addressed in Section 2.5.4.5. Settlement and bearing capacity are discussed in Section 2.5.4.10. Slope stability analysis is discussed in Section 2.5.5.

2.5.4.4 Geophysical Surveys

Section 2.5.4.2.2 provides a description of the geophysical surveys performed. Section 2.5.4.2.5.8 provides a detailed description of the interpretation and recommendation of properties for dynamic soil profiles. The main goal of the surveys was to gather the information to provide a recommendation for velocity profiles underneath foundation footprints.

The best estimate of the shear and compression shear wave velocity profiles are presented by the following four figures:

1. Figure 2.5-166, showing the best estimate velocity profiles in the Powerblock Area.
2. Figure 2.5-167, showing the best estimate velocity profiles in the Powerblock Area, after placement of fill.
3. Figure 2.5-168, showing the best estimate velocity profiles in the Intake Area.
4. Figure 2.5-169, showing the best estimate velocity profiles in the Intake Area, after placement of fill.

2.5.4.5 Excavation and Backfill

Sections 2.5.4.5.1 through 2.5.4.5.4 are added as a supplement to the U.S. EPR FSAR.

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 estimated that approximately 3.5 million cubic yards (cyd) of materials will be moved during earthworks to establish the site grade.

The materials 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. 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 included in COLA Part 11J and Part 11K. The results clearly indicate that there are both plastic and non-plastic soils included in Stratum I soils, including material designated as fill. These fill 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, the suitable portions of the excavated soils are used for site grading purposes, with very little, if any, remaining to be used as structural fill.

It is estimated that about 2 million cyd of structural backfill are needed. Therefore, structural fill will be obtained from off-site borrow sources. An off-site borrow source of structural fill for CCNPP Unit 3 has been identified, Vulcan Quarry in Havre de Grace, Maryland. Details of the engineering and chemical properties of the backfill are provided in Section 2.5.4.2.4.

2.5.4.5.2 Extent of Excavations, Fills, and Slopes

In the area of CCNPP Unit 3, the current ground elevations range from approximately El. 50 ft to El. 120 ft, with an approximate average El. 88 ft. The finished grade in CCNPP Unit 3 Powerblock Area ranges from about El. 75 ft to El. 85 ft; with the centerline of Unit 3 at approximately El. 85 ft. Earthwork operations are performed to achieve the planned site grades, as shown on the grading plan in Figure 2.5-173. 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 2.5-173. Seismic Category I structures with their corresponding foundation are:

- Nuclear Island Common Basemat (El. 41.5).
- Emergency Power Generating Building (El. 76).
- Essential Service Water Buildings (El. 61.0).
- Ultimate Heat Sink Makeup Water Intake Structure (El. -26.5).
- Ultimate Heat Sink Electrical Building (El. -10.5).

Excavation profiles (at the corresponding locations shown in Figure 2.5-103, Figure 2.5-104, and Figure 2.5-105) are shown in:

- Subsurface and excavation profile Powerblock Area A-A': Figure 2.5-160.
- Subsurface and excavation profile Powerblock Area B-B': Figure 2.5-161.
- Subsurface and excavation profile Powerblock Area C-C': Figure 2.5-162.
- Subsurface and excavation profile Powerblock Area D-D': Figure 2.5-163.
- Subsurface and excavation profile Powerblock Area E-E': Figure 2.5-164.
- Subsurface and excavation profile Intake Area F-F': Figure 2.5-165.

These figures illustrate that excavations for foundations of Seismic 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 Intake Area. In the Intake Area, the foundations are 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 derive support from these soils, variations in the top of this stratum were evaluated, reflected as elevation contours for the top of Stratum IIb in CCNPP Unit 3 and in CLA areas, as shown in Figure 2.5-174. The variation in top elevation of these soils is very little, approximately 5 ft or less (about 1 percent) across each major foundation area. The extent of excavations to final subgrade, however, is determined during construction based on observation of the actual soil conditions encountered and verification of their suitability for foundation support. Once subgrade suitability in Stratum IIb soils is confirmed, the excavations are backfilled with compacted structural fill to the foundation level of structures or, if necessary, lean concrete is placed as a leveling mat. Subsequent to foundation construction, the structural fill is 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 Section 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, are graded on an inclination not steeper than 2:1 horizontal:vertical (H:V) or even extended to inclination 3:1 H:V, if found necessary, and having a factor of safety for stability of at least 1.30 for static conditions.

Excavation for the Ultimate Heat Sink Makeup Water Intake Structure and the Ultimate Heat Sink Electrical Building is different than that for other CCNPP Unit 3 structures, as shown in Figure 2.5-165. Given the proximity of this excavation to the Chesapeake Bay, this excavation is made by installing a sheetpile cofferdam that not only provides excavation support but also aids with the dewatering needs. This is addressed further in Section 2.5.4.5.4.

2.5.4.5.3 Compaction Specifications

Testing of structural backfill is described in Section 2.5.4.2.4. For foundation support and backfill against walls, structural fill is compacted to minimum 95 percent of its maximum dry density, as determined based on the Modified Proctor compaction test procedure (ASTM, 2002). The fill is compacted to within 3 percent of its optimum moisture content.

Fill placement and compaction control procedures are addressed in a technical specification 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 and moisture content testing frequency, e.g., a minimum of one test per 10,000 square ft of fill placed.

The backfill supplier will submit samples of backfill prior to placement to perform tests such as Modified Proctor, grain size and chemical properties. The number of samples should adequately cover each of the backfill supply batches. Samples should be collected in accordance with ASTM D75. Each sample should be representative of the material from a single source. Testing will be performed by an independent qualified laboratory.

Samples from each placement lift (usually 8 feet) will be extracted from the placed fill. Careful inspection during fill placement will be enforced and sample collection will be prioritized and fill placement progress interrupted if required. The number of samples will be sufficient to adequately represent the area coverage of the backfill for each lift. The number of required collection samples will be indicated by the testing specification

Once fill is placed, and prior to beginning of foundation work, the following in-situ tests will be performed to verify strength and dynamic properties:

- Standard Penetration Tests, since the N value is extremely useful to correlate to other strength and dynamic properties.
- In-Situ, conventional downhole test, to measure shear wave velocity as a function of depth. The downhole test is preferred to the PS-Logging since casing and grouting will be required to maintain the integrity of the hole.
- In-Situ, surface wave shear wave velocity measurements.

2.5.4.5.4 Dewatering And Excavation Methods

Groundwater control is required during construction. Groundwater conditions and dewatering are addressed in Sections 2.4.12.5 and 2.5.4.6.

Given the soil conditions, excavations are performed using conventional earth-moving equipment, likely using self-propelled scrapers with push dozers, excavators and dump trucks. 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 are needed due to localized conditions or due to deeper foundation elevations such as at the Ultimate Heat Sink 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 are planned as open cut. Upon reaching the final excavation levels, all excavations are cleaned of any loose materials, by either removal or compaction in place. All final subgrades are inspected and approved prior to being covered by backfill or concrete. The inspection and approval procedures are addressed in the foundation and earthworks specifications developed during the detailed design stage of the project. These specifications include measures, such as proof-rolling, excavation and replacement of unsuitable soils, and protection of surfaces from deterioration.

As discussed in Section 2.5.4.5.2, excavation for the Ultimate Heat Sink Makeup Water Intake Structure requires the installation of a sheetpile cofferdam. The sheetpile structure extends from the ground surface to a depth of about 50 ft. The full scope of the sheetpile cofferdam is 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) is monitored in excavations for selected Seismic Category I structures. Rebound estimates are addressed in Section 2.5.4.10. Monitoring program specifications are developed during the detailed design stage of the project. The specification document addresses 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 excavation, dewatering, and foundation construction.

2.5.4.6 Groundwater Conditions

Sections 2.5.4.6.1 through 2.5.4.6.5 are added as a supplement to U.S. EPR FSAR.

2.5.4.6.1 Groundwater Conditions

Details of available groundwater conditions at the site are given in Section 2.4.12. The shallow (surficial) groundwater level in the CCNPP Unit 3 area ranges from approximately El. 68 to El. 85.7 ft, or an average El. of 80 ft. This elevation is considered as the in-situ, current condition groundwater elevation. Similarly, the groundwater level associated with the deeper hydrostatic surface was found to range from approximately El. 16 ft to El. 42 ft, with an average El. of 34 ft. Available observation well data indicate the groundwater table in the Intake Area is at about El. 7 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 away from excavations, i.e., through runoff prevention measures and/or ditches. There are no Seismic Category I foundations planned within the upper water-bearing soils. Groundwater in the powerblock after construction is expected to be at El. 55. Additional detail is provided in Section 2.4.12.

2.5.4.6.2 Dewatering During Construction

Temporary dewatering is required for groundwater management during construction. On the basis of defined groundwater conditions, groundwater control/construction dewatering is needed at the site during excavations for the Powerblock Area foundations. Groundwater associated with seepage in the shallow (upper) zones (Surficial aquifer) is controlled through site grading and/or a system of drains and ditches, as previously discussed. This may also consist of more positive control, including a series of sumps and pumps strategically located in the excavation area to effectively collect and discharge the seepage that enters the excavation, in addition to ditches, drains, or other conveyance systems.

The drainage ditches are installed below grade level, at the peripheries, as the excavation progresses. These ditches are oriented in approximately north-south and east-west directions, i.e., at excavation corners or more frequently as warranted during construction. Once at the final subgrade, stone-filled drains are installed in the excavation interior for control of upward seepage, if any. These drains are in turn connected to exterior ditches and sumps. Each sump is equipped with a pump of sufficient capacity for efficient groundwater removal. Based on the estimated lateral groundwater flow rate derived in Section 2.4.12.5, a total of four pumps with capacity of 100 gpm each will be used for the dewatering.

Temporary dewatering is required for the excavation of the Ultimate Heat Sink Makeup Water Intake Structure and other neighboring structures. A sheetpile cofferdam, designed to aid with dewatering, needs to be extended into low permeability soils; however, some level of groundwater control is still required to maintain a relatively "dry" excavation during construction. As a minimum, pumps are installed to control and/or lower the groundwater level inside the cofferdam. Given the limited excavation size, one 100 gpm pump is sufficient for control of groundwater in this excavation.

Additional auxiliary pumps are available for removal of water from excavations during periods of unexpected storm events. The groundwater level in excavations will be maintained at a minimum of 3 ft below the final excavation level.

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 2.4.15. 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 2.5.4.2.3. 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 2.5-33.

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.

Sections 2.5.4.7.1 through 2.5.4.7.3 are added as a supplement to the U.S. EPR FSAR.

2.5.4.7.1 Soil Velocity Profiles

Section 2.5.4.2.5.8 provides a detailed discussion of the selected soil velocity profiles used for the site response analysis. Details of the site response analysis are provided in Section 2.5.2.

2.5.4.7.2 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.

2.5.4.7.3 Acceleration Time History For Soil-structure Interaction Analysis

A spectrum-compatible acceleration-time history was developed for use with the velocity profile described in Section 2.5.4.8. 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 Safe Shutdown Earthquake (SSE) (refer to Sections 2.5.2.5 and 2.5.2.6), a zero depth peak ground acceleration of 0.084 g associated with a magnitude **M**5.5 earthquake was computed. However, the SSE of the CCNPP Unit 3 uses a peak ground acceleration of 0.15 g.

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 (USNRC, 2003c). The soil properties and profiles utilized are those described in Section 2.5.4.2.

Sections 2.5.4.8.1 through 2.5.4.8.6 are added as a supplement to the U.S. EPR FSAR.

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.

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.

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 (USNRC, 2003c) would not add additional information to the planning and conduct of the subsurface investigation; therefore, was not conducted.

A common stratigraphy was adopted for the purpose of establishing soil boundaries for liquefaction evaluation. The adopted stratigraphy was that shown generically in Figure 2.5-106 and also by the velocity profiles shown in Figure 2.5-167 and Figure 2.5-169. 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 2.5.4.7.3, the resulting peak ground acceleration for the site was found to be 0.084 g associated with a magnitude **M**5.5 earthquake. For conservatism, a peak ground acceleration of 0.15 g 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, et al., 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 (Youd, et al., 2001) for evaluating the liquefaction potential of soils at the CCNPP Unit 3 site. Given the adequacy of these methods in assessing

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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, V_s, 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). 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, et al., 2001). A magnitude scaling factor (MSF) of 1.93 was used in the calculations based on the adopted earthquake magnitude and guidelines in Youd (Youd, et al., 2001). Below are examples of liquefaction resistance calculations using the available SPT, V_s, and CPT data in the Powerblock Area and Intake Area. Calculations were performed mainly using spreadsheets, supported by spot hand-calculations for verification.

2.5.4.8.4 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, et al., 2001), based on corrected SPT N-values $(N_1)_{60}$, including corrections based on hammer-rod combination energy measurements at the site. The soils at CCNPP site include clean granular soils with $(N_1)_{60}30$ that are considered too dense to liquefy and are classified as non-liquefiable (Youd, et al., 2001). Similarly, corrections were made for the soils fines contents, based on average fines contents and the procedure recommended in Youd (Youd, et al., 2001).

The collected raw (uncorrected) SPT N-values are shown in Figure 2.5-113 and Figure 2.5-114. SPT data from the figures were used for the liquefaction FOS calculations for over 2000 SPT N-value data points. The results are shown in Figure 2.5-176 for the Powerblock Area and Figure 2.5-177 for the Intake Area.

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 2.5-113 and Figure 2.5-114 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.

Of the over 2,000 SPT N-value data points for which FOS values were calculated, no points resulted with FOS<1.1 below foundation grade.

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, or are at locations where no structures are planned. Hence, the low FOSs are not a concern for these samples. Based on SPT data, there is no potential for liquefaction for the CCNPP3 Unit 3 Powerblock and Intake Areas.

2.5.4.8.5 FOS Against Liquefaction Based on Shear Wave Velocity Data

Similar to the FOS calculations for the SPT values, equivalent clean-sand $CRR_{7.5}$ values, based on V_s measurements, were calculated following recommendations in Youd (Youd, et al., 2001). Soils at the CCNPP site include soils with normalized shear wave velocity (V_{S1}) exceeding a value of 215 m/s (705 fps). Clean granular soils with V_{S1} larger than 215 m/s (705 fps) are considered too dense to liquefy and are classified as non-liquefiable (Youd, et al., 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/s (705 fps) and 200 m/s (656 fps) for fines contents of less than 5 percent and larger than 35 percent, respectively. As such, when values of V_{S1} are larger than 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 2.5-118 and Figure 2.5-119 for the Powerblock and Intake Areas respectively. 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 V_s data points were used for the FOS calculations. The results showing FOS against liquefaction using the shear wave velocity data are provided in Figure 2.5-178 and Figure 2.5-179.

The results show that all calculated FOSs exceeded 1.1 with significant margin; 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. Based on shear wave velocity data, there is no potential for liquefaction for the CCNPP Unit 3 Powerblock and Intake Areas.

2.5.4.8.6 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 Powerblock CPT data was reviewed and correlated with the applicable SPT data and compared with guidelines in Robertson (Robertson, et al., 1988). This review process verified the CPT data by correlation to the CCNPP Unit 3 site-determined SPT values.

The equivalent clean-sand $CRR_{7.5}$ value, based on CPT tip measurements, was calculated following recommendations in Youd (Youd, et al., 2001), based on normalized clean sand cone penetration resistance $(q_{c1N})_{cs}$ and other parameters such as the soil behavior type index, Ic.

Cone tip resistance values from CPT soundings are shown in Figure 2.5-115 and Figure 2.5-116 for the Powerblock and Intake Areas respectively. 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 El. -80 ft. Tip resistance measurements were made at 5-cm intervals (~2-in). The results showing FOS against liquefaction using the CPT data are provided in Figure 2.5-180 and Figure 2.5-181 for the Powerblock and Intake Areas, respectively. For completeness, all data points, including data for clay soils, were included in the calculation, despite their known high resistance to liquefaction.

Only data points in the upper layers resulted in FOS>1.1. 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, et al., 2001). The soils in Stratums I and IIa will be removed during construction. In addition an extremely conservative margin is adopted by using a PGA value of 0.15 g. Based on CPT data, there is no potential for liquefaction for the CCNPP3 Powerblock and Intake Areas.

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2.5.4.8.7 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. 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, et al., 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 5.0. 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 2.5-166 through Figure 2.5-169, 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.

2.5.4.8.8 Potential for Liquefaction of Backfill

Section 2.5.4.5 describes material specifications and compaction for structural fill. For foundation backfill, compaction will be done to 95 percent of Modified Proctor optimum dry density. The fill will be compacted to within 3 percent of its optimum moisture content.

Liquefaction in an engineered fill is not an issue if the recommended compaction practices are followed. Liquefaction occurs in loose sands and/or silts with poor gradation. An engineered fill is a compacted and well graded soil structure. Compaction practices need to be monitored during construction. Liquefaction of granular engineered fills will be prevented by assuring that the fill specifications are met during the implementation stages. Particular attention will be placed on the grain size and compaction requirements to ensure the specifications are fully met. Specifications for fill will include requirements for an on-site testing laboratory for quality control, especially material gradation and plasticity characteristics, the achievement of specified moisture-density criteria, fill placement/compaction, and other requirements to ensure that the fill operations conform to the earthwork specification for CCNPP Unit 3.

Regardless of the non-liquefiable nature of engineered fills, the liquefaction potential was also evaluated with the shear wave velocity approach. Figure 2.5-167 indicates that the values for the backfill are 790, 900, and 1080 fps. The 790 fps backfill will not be exposed to saturated conditions since it only corresponds to the first six ft from the surface. The results of the analysis are shown in Figure 2.5-182. Based on shear wave velocity data, there is no potential for liquefaction for the CCNPP3 backfill.

2.5.4.8.9 Concluding Remarks on Liquefaction Analysis

It is evident, from the collective results, that soils at the CCNPP Unit 3 site are overly-consolidated, geologically old, and sometimes even cemented. 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, et al., 2001). Additionally, in the liquefaction evaluation, the effects of age, overconsolidation, and cementation were ignored. These factors 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 is not 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.Limited man-made fill may be already present at the CCNPP Unit 3 site at isolated locations. These soils will be removed during construction.These activities, further improve the liquefaction resistance of soils at the site.

2.5.4.8.10 Regulatory Guide 1.198

Before and during the liquefaction evaluation, guidance contained in NRC Regulatory Guide 1.198 (USNRC, 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 (USNRC, 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 (USNRC, 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 (USNRC, 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 El. 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 beyond El. 55 ft in the future given the relief and topography of the site, promoting drainage. Similarly, NRC Regulatory Guide 1.198 (USNRC, 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 (USNRC, 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 (USNRC, 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 Site Characteristics

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" (USNRC, 2007b) with reference to NUREG/CR-6728 (REI, 2001) and ASCE/SEI 43-05 (ASCE, 2005). 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 a soil Uniform Hazard Spectra (UHS), accounting for site-specific conditions using Approach 2A of NUREG/CR-6769 (USNRC, 2002). 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 CCNPP Powerblock Area is graded to establish the final site elevation, which will range from about El. 81 ft to 85 ft. An average grade elevation of 83 ft is assumed. The Reactor, Safeguards, and Fuel Buildings are seismic Category I structures and are supported on a common basemat. For a basemat thickness of 10 ft and top of basemat about 31.5 ft below grade, the bottom of the basemat would be 41.5 ft below the final site grade, or El. 41.5 ft. The common basemat has an irregular shape, approximately 80,000 square feet (sq ft) in plan area, with outline dimensions of about 363 ft x 345 ft. For bearing capacity and settlement estimation, a representative foundation is used. Table 2.5-64 presents the values for elevation, depth, area, and loads of the seismic Category I structures and the main structures in the Powerblock area. This information is also shown in Figure 2.5-183.

Construction of the common basemat requires an excavation of about 41 to 42 ft (from approximately El. 83 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 4 in is estimated due to excavation for the common basemat, and is expected to take place concurrent with the excavation. Ground rebound is monitored during excavation. The heave estimate is made based on the elastic properties of the CCNPP site soils and the response to the unloading of the ground by the excavation. The magnitude and rate of ground heave is a function of, among other factors, excavation speed and duration that the excavation remains open. Other factors remaining unchanged, shorter durations culminate in smaller values of ground heave.}

2.5.4.10.1 Bearing Capacity

The U.S. EPR FSAR 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 foundation basemats of Seismic Category I structures have the capacity to support the bearing pressure with a factor of safety of 3.0 under static conditions.

This COL Item is addressed as follows:

{The ultimate bearing capacity of safety-related buildings for the Powerblock and Intake Areas is estimated using the closed form solutions proposed by Vesic (Vesic, et al., 1975) and Meyerhof (Meyerhof, et al., 1978). Factors of safety are obtained for different soil profile cases and compared with standard practice allowable values.

The soil profiles of CCNPP Unit 3 and Intake Areas are used in the analysis in order to determine the corresponding layer thickness and material properties. Stratum thicknesses and elevations are presented in Table 2.5-25.

Weighted average values of soil parameters are used in the analysis; weight factors are based on the relative thickness of each stratum within a specific depth (i.e. depth equal to the least lateral dimension of the building).

The water table in the Powerblock Area is conservatively considered to be at El. 83 ft, which corresponds to the average grade surface elevation. For the Intake Area, the water table is considered to be at El. 10 ft, which also corresponds to the average grade surface elevation. With the higher groundwater level, the bearing capacity estimate will be more conservative since overburden resistance is diminished by increased buoyant effect.

Average values of the soil strength parameters (c', ϕ', s_u, γ) are considered in the analysis. Average unit weights are calculated using data from the entire CCNPP Unit 3 area (limited number of samples were available for strength parameters in the Powerblock Area, therefore data from the Construction Laydown Area (CLA) area are included in the calculation of the average values). Sand layers present a relatively low cohesion due to the presence of fine particles, based on laboratory tests results. However, for this analysis the cohesion for sand layers is conservatively not considered ($c' = 0$).

The ultimate static bearing capacity of a footing supported on homogeneous soils can be estimated using the following equation (Vesic, et al., 1975):

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + \frac{1}{2}\gamma' B' N'_{} s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma r_\gamma + qN_q s_q d_q i_q g_q b_q$$

Where:

q_{ult} → Ultimate bearing capacity;

c → Cohesion;

N_c, N_g, N_q → Bearing capacity factors;

$$N_q = e^{\pi \tan\phi}\tan^2(45 + \phi/2);$$
$$N_c = (N_q - 1)\cot\phi;$$
$$N_\gamma = 2(N_q + 1)\tan\phi;$$

ϕ → Friction angle;

s_c, s_γ, s_q → Foundation shape correction factors;

d, i, g, b → Shape, depth, and inclination factors;

r_γ → Foundation size correction factor;

γ' → Effective unit weight of foundation media;

B' → Effective foundation width;

Three different cases are considered in the analysis:

a. Soil subsurface including all strata: For this case, weighted average values of the strength parameters are used based on relative thickness of each stratum in the zone between the bottom of the footing and a depth B below this point, where B is the least lateral dimension of the building. For this case, effective soil parameters are used (drained conditions). (Vesic, et al. 1975)

b. Soil subsurface considering only stratum IIb Chesapeake Cemented Sand. Soil parameters of this layer are used for the entire depth. For this case, effective soil parameters are used (drained conditions). (Vesic, et al. 1975)

c. The ultimate static bearing capacity of a footing supported on a dense sand stratum over a soft clay stratum can be estimated using the punching shear failure with a circular slip path (Meyerhof, et al., 1978):

$$q_{ult} = q_{u,b} + \frac{2\gamma_1 H_t^2}{B}\left(1 + \frac{2D}{H_t}\right)K_{ps}\tan\phi_1 - \gamma_1 H_t \le q_{ut}$$
$$q_{u,b} = c_2 N_{c_2}\zeta_{c_2} + \frac{1}{2}\gamma'_2 B' N_{\gamma_2}\zeta_{\gamma_2} r_\gamma + \gamma'_1 (H_t + D) N_{q_2}\zeta_{q_2}$$
$$q_{ut} = c_1 N_{c_1}\zeta_{c_1} + \frac{1}{2}\gamma'_1 B' N_{\gamma_1}\zeta_{\gamma_1} r_\gamma + \gamma'_b D N_{q_1}\zeta_{q_1}$$

Where:

q_u → Ultimate bearing capacity;

$q_{u,b}$ → Ultimate bearing capacity of a very thick bed of the bottom soft clay layer;

q_{ut} → Ultimate bearing capacity of upper dense sand layer;

γ'_1 → Effective unit weight of the upper sand layer;

γ'_2 → Effective unit weight of the lower clay layer;

γ'_β → Effective unit weight of backfill;

ϕ_1	→	Friction angle of upper sand layer;
ϕ_2	→	Friction angle of lower clay layer;
c_1	→	Cohesion of upper sand layer;
c_2	→	Cohesion of lower clay layer;
H_t	→	Depth from footing base to soft clay;
D	→	Depth from of footing base below ground surface;
K_{ps}	→	Punching shear coefficient;
B'	→	Effective foundation width;
ζ_q, ζ_c, ζ_γ	→	Geometry Factors;
N_c, N_γ, N_q	→	Bearing capacity factors;

Buildings are considered to have an equivalent rectangular foundation with the same area and moment of inertia as the original footprint shape. The analysis is preformed using uniformly distributed loads in all buildings. For the NI Common Mat, an average uniform load is used including the loads from the Reactor, Safeguard and Fuel Buildings. The vertical load imposed by adjacent structures is conservatively not included in the calculation of bearing capacity of each building, only the surcharge imposed by the backfill is considered.

The vertical loads and dimensions of the buildings that comprise the NI common mat are not symmetrical. This will result in overturning moments around the centroid of the common mat that will reduce the contact area of the foundation and hence the bearing capacity. To account for this reduction in the contact area, an effective area is used in the bearing capacity equations. The length (L) and width (B) of the foundation's footprint are reduced in proportion to the eccentricity of the resultant vertical force. For the CCNPP3 NI common mat the asymmetry in dimensions and static loads is not significant; the effective area is approximately 98% of the total area.

The Meyerhof model represents a more realistic approach to calculate the bearing capacity of the soil subsurface at CCNPP 3, by considering a dense sand layer overlying a softer clay layer. This model considers a punching shear failure mechanism between both layers.

A summary of the calculated allowable static and dynamic bearing capacities using both the layered and the homogeneous soil conditions are presented in Table 2.5-65. A factor of safety of 3.0 for static loads (dead plus live loads) and 2.0 for dynamic loading are typically considered to be acceptable.

Table 5.0-1 of the U.S. EPR FSAR identifies the soil bearing capacity as a required parameter to be enveloped, defined as a minimum static bearing capacity of "22,000 lb/ft^2 in localized areas at the bottom of the Nuclear Island basemat and 15,000 lb/ft^2 on average across the total area of the bottom of the Nuclear Island basemat." and a "minimum dynamic bearing capacity of 34,560 lb/ft2 at the bottom of the NI basemat."

The static bearing capacity is above the localized 22 ksf requirement and the dynamic bearing capacity is above the 34.56 ksf requirement.

For static and dynamic loading conditions, and based on a factor of safety of 3.0 (static) and 2.0 (dynamic), the site provides adequate allowable bearing capacity.}

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2.5.4.10.2 Settlement

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.10.2:

> A COL applicant that references the U.S. EPR design certification will verify that the differential settlement value of ½ inch per 50 ft in any direction across the foundation basemat of a Seismic Category I structure is not exceeded. Settlement values larger than this may be demonstrated acceptable by performing additional site specific evaluations.

This COL Item is addressed as follows:

{The surface topography and subsurface conditions of the CCNPP Unit 3 Powerblock Area make the estimation of settlement and building tilt complex. The objective of the settlement analysis of the CCNPP Powerblock Area is to provide an estimate of the time dependant settlement and heave distribution throughout the footprint of the Powerblock Area, including maximum settlement and tilt estimated for each of the facilities.

The settlement analysis of the CCNPP Powerblock Area was carried out under the following premises:

- Develop a three-dimensional model capable of capturing irregular subsurface conditions, realistic foundation footprint shapes, and asymmetric building loads;
- Perform a time-dependant simulation, that provides settlement and tilt estimates as a function of time through and after construction;
- Incorporate a construction sequence and examine the behavior of settlement and tilt as buildings are erected;
- Account for asymmetric topography, by recognizing that reloading time to original consolidation pressure after excavation will be variable throughout the foundation footprint;
- Perform the settlement analysis simultaneously for the NI and adjacent facilities, including the detached safety related structures (EPBG and ESWB);

2.5.4.10.2.1 Settlement Calculation Methodology

In order to address the issues described above, a Finite Element Method (FEM) model of the subsurface and structural interfaces was developed. The FEM has the capability of providing a numerical solution to the general equations of elasticity in continuous media. The settlement analysis of the CCNPP Powerblock Area is performed with the computer application PLAXIS 3D Foundation v2 (PLAXIS3D) (DUTP, 2007). The application has been validated and verified under the Paul C. Rizzo Associates, Inc. (RIZZO) Quality Assurance Program. The settlement computations have also been performed under RIZZO QA Program.

PLAXIS3D provides a FEM solution of the virtual work equation defining equilibrium conditions and natural boundary conditions in a differential equation form. The program calculates displacements with the use of numerical integration methods. In addition to the typical capabilities of a general FEM application for elastic solids, PLAXIS incorporates advanced constitutive models, (stress vs. strain relationships) that are capable of simulating the response of soils to external loading. Such response includes both elastic/elastoplastic displacement and consolidation. This feature makes PLAXIS3D a unique application for the analysis of foundation systems and its applicability to the CCNPP Powerblock settlement problem is ideal. The application allows for the elaboration of a three-dimensional representation of the subsurface conditions and the building geometries. The model is capable of capturing variation of soil properties below the footprints of the foundation and therefore it is possible to better assess differential settlement. All structures in the Powerblock Area are modeled simultaneously and load increments are applied in different steps in time.

The Mohr-Coulomb constitutive model is selected for the analysis. Other soil hardening constitutive models introduce further sophistication to account for the stress-dependancy of the stiffness, but are slightly less conservative when compared to the Mohr-Coulomb model. This analysis accounted for increased unload and reload elastic moduli with the use of conservative ratios applied at different time steps during the unloading and loading sequence. This approach provided a better understanding of the effect that irregular topographic conditions had in settlement and tilt. Further details are provided in the following sections.

2.5.4.10.2.2 Settlement and Heave Analysis in the CCNPP Powerblock Area

The settlement analysis of the Powerblock Area is based on an FEM model of approximately 2500 ft x 2500 ft x 840 ft (Length x Width x Depth). The area occupied by the buildings is approximately 1100 ft by 1100 ft. There are 42,130 nodes in the model. The boundary conditions for the sides of the model included allowing the vertical displacement, and restraining the two horizontal displacement components. The bottom of the model was restrained in vertical and horizontal directions. The free drainage conditions for consolidation were adapted on the model boundaries. Since the model boundaries were far enough from the loaded areas, the primary direction for the water flow is the vertical direction. In other words, the sides of the model are far enough from the loaded areas so that the consolidation behavior is not impacted by the free-drainage conditions implemented on the sides of the model.

Soil profiles, such as those shown by Figure 2.5-107, were taken as the basis for the geotechnical input of the FEM model. In addition, data from boreholes B-311, B-313, B-334, B-335, B-344, and B-357A were included to adequately represent the three-dimensional nature of the model. PLAXIS3D interpolates information between borehole locations to obtain the three-dimensional representation of the subsurface conditions, as shown in Figure 2.5-184. The figure presents a reduced version of one of the excavation profiles to illustrate how the FEM geometry conforms to the subsurface conditions. The CCNPP Powerblock Area model is a comprehensive mathematical representation of the physical conditions at the site.

The analysis depth is approximately twice the width of the NI foundation footprint. Therefore, given the dimensions of the NI common basemat, the model depth was extended to El. -760 ft. This was achieved by extending the Nanjemoy sand (the continuous soil layer deeper than -208 ft elevation) to the bottom of the model.

Two separate models were developed for the CCNPP Powerblock Area:

1. An Excavation and Dewatering Model (ED Model).

2. Construction and Post-Construction Model (CPC Model).

Heave Analysis: Excavation and Dewatering (ED Model)

On saturated soils, prior to excavation, it is necessary to dewater the excavation area. As water is extracted from the voids, soils will consolidate and settlement due to dewatering will take place. In addition, soils beneath dewatered areas will experience increased loading as consolidation of upper layers takes place. The effect that dewatering has on settlement depends on the soil properties, the hydrogeologic conditions, and to some extent on the pumping rates.

At the CCNPP Powerblock Area, the Stratum IIa Chesapeake Clay/Silt isolates the upper surficial aquifer from the layers beneath. The surficial aquifer is confined by the first clay layer and it does not influence the soils at and beneath foundation elevation. Therefore, dewatering will not produce settlement at the foundation level. In consequence soils will not experience increased stress due to dewatering and such increase need not be accounted for as an excess consolidation pressure as it is typically done if the surficial aquifer was not confined.

Heave will be experienced after excavation and the ED FEM model was used to estimate its magnitude. For this model, the Powerblock Area was divided in three zones considering different average ground elevations for each zone. The subdivision was performed based on the site topography information, as shown in Figure 2.5-185. The zones are:

- Zone I: low areas North East (Plant Local Coordinate System) with an average ground elevation of 60 ft;
- Zone II: South areas (Plant Local Coordinate System) with an average ground elevation of 80 ft;
- Zone III: high areas with an average ground elevation of 105 ft.

The division was done to capture the difference in heave resulting from different depths of excavation. As shown by the resulting variable heave distribution in Figure 2.5-186, the effect of topography is adequately captured. As expected, the magnitude of heave is directly related to the surface topography. Between the end of excavation and the beginning of construction, the maximum reported heave at the center of containment (Point C) is 4.7 in. Most of the heave is elastic and is experienced immediately after excavation. Table 2.5-66 provides heave results for the four locations shown in Figure 2.5-186.

Once excavation is completed, the foundation surface will be prepared for the placement of foundations. Settlement in the following sections will be reported from the beginning of construction or the initial reloading of the soil.

Settlement Analysis: Construction and Post-Construction (CPC Model)

The CPC model was designed to evaluate the settlements during and after construction. This model is not a continuation of the ED model. The excavation and dewatering stages included in ED model were assumed to be completed, and the excess pore pressure generated due to excavation and dewatering fully dissipated. As previously stated, settlement will be reported from the beginning of construction and beyond. The analysis also assumes that the ground surface was re-leveled after the immediate heave. As previously stated, long term heave is a small fraction of the total displacement when compared to the immediate elastic value.

The initial effective stress condition for the CPC model was in accordance with the post-excavation overburden geometry. The model assumes an average surface Elevation of 83 ft. The effect of asymmetric topography is evaluated by performing sensitivity analysis on the value of the initial ground surface elevation (i.e., initial overburden stress). A detailed discussion is provided later in this Section.

The building loads were applied in eight sequential steps as specified by Table 2.5-67. The table corresponds to the construction schedule. The loading sequence is also shown in Figure 2.5-187. Settlement analysis is conducted at the application of each step, accounting for both immediate and consolidation settlements. After the application of the last loading sequence and finalization of construction, partial rewatering occurs in the construction area. The final groundwater elevation is El. 55 ft. The construction schedule affects the timing of the settlement and tilt during construction. However, end values will be similiar if variations that are typical during construction take place.

Backfill between El. 41.5 ft and El. 83 ft was placed in the first five steps indicated by Table 2.5-67 as follows:

1. During Step 1, backfill is placed between El. 41.5 ft and El. 48 ft.
2. During Step 2, additional backfill is placed between El. 48 ft and El. 61 ft.
3. During Step 3, additional backfill is placed between El. 61 ft and El. 66 ft.
4. During Step 4, additional backfill is placed between El. 66 ft and El. 76 ft.
5. During Step 5, additional and final backfill is placed between El. 76 ft and El. 83 ft.

The groundwater elevation in the Powerblock Area was modeled at El. 38 ft during construction to account for dewatering. Around the Powerblock Area, the groundwater elevation was maintained at El. 69 ft. For the post-construction conditions, groundwater elevation in the Powerblock Area was increased up to El. 55 ft and remained constant at that level, while the groundwater elevation around the Powerblock Area remained at El. 69 ft. Post construction groundwater levels will have little impact on the construction settlement.

The stiffness of the foundation mats is also accounted for in the analysis. As the construction proceeds, the deflection pattern of the foundations is expected to be closer to the rigid body motion due to the additional stiffness introduced into the foundation by the structure itself. The stiffness of the foundation mat was transitioned from an initial value based on a 10 ft thick concrete mat to a stiff, rigid-body like condition at the end of construction.

The soil properties used in the settlement analysis are provided in Section 2.5.4.2.5. The soil properties that directly impact the settlement analysis are:

- Unit Weight,
- Permeability and Coefficient of Consolidation,
- Strength parameters, used in the Mohr-Coulomb constitutive model,
- Elastic Modulus and Poisson Ratio,
- Ratio of Unload/Reload Modulus to Elastic Modulus.

The elastic modulus in the deeper Nanjemoy Sand was increased linearly, as a function of depth from its estimated value of 3,170 ksf at the interface with Layer IIC. The value of E at the lower boundary of the FEM model is 4,600 ksf, which corresponds to a rate increase of 2.6 ksf/ft. The increase was performed according to the following relationship (DUTP, 2007) (Schanz, et al., 1999) applicable to a sand with no cohesion:

$$E = E_{ref}\sqrt{\frac{(1-\sin\phi)\sigma'_1}{p_{ref}}}$$

Where:

E	→	Elastic modulus at desired depth (El. -760 ft, end of FEM model);
E_{ref}	→	Reference elastic modulus, calculated with effective vertical stress at El. -207.5 (Nanjemoy Sand top horizon elevation)
ϕ	→	Friction angle (40°);
p_{ref}	→	Reference pressure (100 pressure units);
σ'_1	→	Effective vertical stress;

During the analysis, it was required to account for the asymmetric distribution of surface topography throughout the Powerblock Area. This condition is especially important for the NI common basemat. Figure 2.5-175 clearly shows that the existing surface grade at the NI changes up to 50 ft in elevation. At the lower portions, the construction of the plant will reach the original pre-consolidation pressure relatively soon. On the contrary, for high elevation points, this condition will be reached at later stages into the construction. During the first six steps of construction, some points throughout the foundation footprint will be experiencing reloading, while others are subject to loads that are higher than the original overburden pressure. This fact will have direct influence in the estimation of tilt. The topographic conditions suggest that there is potential for the NI common basemat to present additional tilt towards the North or North East (Local Coordinates) direction along the cross section indicated in Figure 2.5-175.

In order to incorporate the influence of surface topography into the settlement estimates, sensitivity on the initial average surface elevation was performed according to the following cases:

1. *Settlement Representative of Low Surface Elevation Zones:* The unloading/reloading modulus was used until the end of the second loading step, when the reloading for the North East part of the Powerblock Area is expected to be completed. For Step three the elastic modulus value was reverted to its lower counterpart (loading Elastic modulus). This case represents the stress-stiffness correspondence for the parts of the Powerblock Area with an initial pre-excavation ground surface of about El. 60 ft.

2. *Settlement Representative of Medium Surface Elevation Zones:* The unloading/reloading modulus was used until the end of the third and fourth loading steps. These cases represent the stress-stiffness correspondence for the parts of the Powerblock Area with an initial pre-excavation ground surface of about El. 80 ft. These two cases cover the elevation range of most of the Powerblock Area.

3. *Settlement Representative of High Surface Elevation Zones:* The unloading/reloading modulus was used until the end of the fifth loading step, when reloading is expected to be completed for the totality of the footprint area. This case represents the stress-stiffness correspondence for the parts of the Powerblock Area with an initial pre-excavation ground surface of about El. 105 ft.

By performing the settlement analysis under multiple scenarios, it is possible to assign the most representative case for each point throughout the foundation footprint, and obtain a reliable estimate of the increase of tilt for each structure, specifically the NI. Figure 2.5-188 provides a conceptual representation of the three cases previously described. Depending on the original surface elevation with respect to plant grade, each zone throughout the footprint will be best represented by one of the three cases.

Settlement Analysis Results

The following plots and tables are provided for the purposes of presenting settlement and tilt estimates:

- Figure 2.5-190: Settlement vs. Time for center point of NI;

  This figure presents the calculation of settlement for cases that consider different initial elevations of surface topography. As previously discussed, revert from reloading to loading modulus occurs sooner for low elevation points and therefore the low elevation case indicates larger settlement. Using conservatism, the case that best represents settlement at center point of containment is the case denominated "Medium Elevation E Revert (2)". According to this case, total settlement at centerline of the reactor building is estimated at 12.7 in.

  Tilt across the NI, especially running West to East and South West to North East (Local Plant coordinates) will be heavily influenced by the variation of surface topography throughout the NI footprint. The relevance of such influence is directly related to the difference in settlement reported by the analysis cases shown in Figure 2.5-190.

- Figure 2.5-189: Settlement contour plot from FEM model (Medium Elevation Topography);

  The contour plots provide the incremental settlement from the Medium Elevation E Revert(2) case, reported after the application of each loading sequence. The maximum settlement for the NI footprint is estimated at 12.7 in. The plots shows the influence that the Nuclear Island has over the rest of the buildings. In general, the Powerblock Area will present a tilt tendency from the perimeter to the center of the footprint. Long term settlement beyond construction will be influenced by secondary consolidation and rewatering.

- Table 2.5-68: Settlement vs. Time for center point of each foundation (Medium Elevation Topography) and Figure 2.5-191, Settlement at the Center Point of Safety Related Buildings;

- Table 2.5-68 presents the tabular data of settlement under the footprint of each facility from the Medium Elevation E Revert(2) case. As expected, the Fuel Building and NI present the highest settlement. Figure 2.5-191 is the graphical representation of the settlement data provided by Table 2.5-68.

- Figure 2.5-192: Settlement tracking cross-sections;

  Tilt was recorded for several cross sections, as indicated by Figure 2.5-192. The selection of the cross-sections was done to assure that maximum tilt is captured.

- Figure 2.5-193 : Foundation base settlement for four sections of the NI and Turbine Building;

The figure indicates how the foundation settles after each step of the construction sequence. The results in the figure correspond to data resulting from the topography case thatconservatively provides settlement at the centerline of the reactor ("Medium Elevation E Revert (2)"). Reported tilt is later adjusted for topography. Thus, the values of tilt reported by Figure 2.5-193 represent a lower bound estimate of tilt for some sections and the upper bound for others.

The differential settlement between the NI and TB is provided after each loading step. Since both facilities are founded on different basemats, a discontinuity shows the magnitude of the differential settlement. The same condition applies between the NI and the NAB. The differential settlement between the NI and these two adjacent facilities is estimated to be in the order of one inch. Tilt between the NAB and the NI occurs in opposite directions, tilting towards each other. This condition needs to be accounted for in the final design and construction.

- Figure 2.5-194: Maximum tilt vs. time for NI;

This plot provides the lower bound and upper bound estimates for tilt across each of the cross sections of the Nuclear Island. In general, the lower bound is represented by the tilt estimate resulting from the medium elevation topography analysis and the upper bound is obtained by introducing an adjustment in tilt that is proportional to a fraction of the settlement difference between the cases "Low Elevation E Revert" and "High Elevation E Revert" (See Figure 2.5-190). Adjustments for topography increase or decrease the values of tilt. Either of the following cases may present itself:

1. Tilt Increase: Tilting due to loads and subsurface condition occurs from points of high surface topography to points of low surface topography. Cross-section BB' (on Figure 2.5-192) running West to East (Local Plant Coordinates) is the most representative section with this condition. Adjustment for topography will increase tilt estimate since the East portion of the section is more representative of the low topography cases. These points are expected to present higher settlement when compared to average elevation values. In the same way, the high surface elevation points at the other end (West) will likely present less settlement than the average topography case.

2. Tilt Decrease: Tilting due to loads and subsurface condition occurs from points of low surface topography to points of high surface topography. This condition presents itself when the highest settlement is recorded in high elevation points and low settlement in low elevation points. Such is the case for cross-section CC' (on Figure 2.5-192). In the section, tilt is measured from the North East corner to the South West covering both the NAB and RB.

3. No Tilt Adjustment: This case occurs when the cross section runs through constant elevation. This is the case of cross-section DD' (on Figure 2.5-192) running North West to South East (Local Plant Coordinates) as shown in Figure 2.5-188.

The proportional tilt adjustment fraction is obtained by:

a. Determining the difference in elevation between the starting point and end point of each section;

b. Dividing the previous value over the elevation difference between the cases "Low Elevation E Revert" and "High Elevation E Revert"

c. Obtaining the tilt associated with asymmetric topographic conditions by multiplying the previous value (Step b) by the difference in settlement between the cases Low Elevation E Revert" and "High Elevation E Revert" and dividing over the Section Length

d. Reducing the factor obtained in Step (c) above by 10 percent. This reduction is introduced as it is expected that both foundation and soil will lessen the difference recorded between the extreme topography analysis cases. This reduction could be more significant but a conservative value is adopted, since there is uncertainty involved.

The results of this correction are shown in Figure 2.5-194. Once adjusted for topography, estimated tilt approaches 1 in per 50 feet.

- Table 2.5-69: Maximum recorded tilt for the structures in the Powerblock Area.
- Figure 2.5-195: provides the settlement underneath each facility corresponding to the cases that analyze the sensitivity on surface topography. Low elevation points will have an increase in settlement after adjustment and high elevation points will see their settlement estimates reduced.

Long Term Settlement (Creep and Rewatering)

Long term settlements related to secondary consolidation or rewatering are estimated to be very small and both aspects will counteract each other. The stress increase induced by loading are consistently lower than the pre-consolidation condition. At CCNPP the ratio of final applied stress to the preconsolidation pressure always remains below 0.7 for the Stratum IIc Chesapeake Clay layer. The effective stress is always in the recompression range and secondary settlement is not significant (Terzaghi, et al., 1995).

Settlement Monitoring

Heave or rebound of the excavation bottom, the effect of dewatering and the effect of Nuclear Island basemat loading during construction will be monitored. This is necessary to confirm that the rate of settlement is consistent with the estimates. A settlement monitoring program will be developed during detailed design. The settlement monitoring program will consist of three primary elements:

- Piezometers to measure pore pressures in Stratum IIb and IIC. Vibrating wire piezometers are preferred for this purpose as they are adequately sensitive and responsive and easily record positive and negative changes on a real time basis.
- Settlement monuments placed directly on concrete, preferably on the mud mat and on the corners of the structures at grade that are accessible with conventional surveying equipment.
- Settlement telltales if monuments are not practical or if fills are used over consolidation type soils and it is necessary to monitor settlement of the consolidation type soils independent of the consolidation of the fill. Telltales can be used after backfill is placed,

Monitoring locations will be distributed at corners of facilities and throughout the perimeter of the Nuclear Island. Monitoring points will be placed to relate settlement measurements to sections (such as those indicated by Figure 2.5-192) so that actual settlement can be compared directly to model results.

Plots showing Movement (settlement or heave) versus Time will be maintained along with Estimated Load versus Time curves.

Conclusions – Settlement Analysis

The analysis and careful examination of the settlement results provide the following conclusions apply.

- Total average settlement at the end of construction beneath the Reactor Building footprint is estimated at 12.7 in. Settlement for other facilities is provided in Table 2.5-68 and Figure 2.5-195 for the medium topography case.
- Long term settlements related to secondary consolidation or rewatering are estimated to be very small and both aspects will counteract each other.
- Maximum tilt for each building is provided in Table 2.5-69. Maximum tilt is highest for Section CC′ of the NI running from south west to north east (Local Coordinates), and Section BB′ running west to east. NI tilt adjusted for topography is shown in Figure 2.5-194.

  Differential settlement or tilt depends on (1) the asymmetric nature of loads, (2) the irregular thickness of the subsurface strata, and (3) the asymmetry in surface topography. The first two are naturally captured by the FEM simulation. The third, influence of asymmetric topography, is captured by means of sensitivity analyses.
- The differential settlement between the NI and TB is provided after each loading step. Since both facilities are founded on different basemats, a discontinuity shows the magnitude of the differential settlement. The same condition applies between the NI and the NAB. The differential settlement between the NI and these two adjacent facilities is estimated to be in the order of one to two inches. Tilt between NAB and RB occurs in opposite directions, and both facilities tilt towards each other. This condition needs to be accounted for in the final design and construction.
- Groundwater is below foundation grade during construction. After construction, groundwater is expected to rise to El. 55. The settlement estimates are not sensitive to variations in the groundwater rebound level, if such variations are in the order of plus or minus ten feet.

The U.S. EPR FSAR Section 2.5.4.10.2 identifies differential settlement as a required parameter to be enveloped, defined as "½ inch per 50 ft in any direction across the foundation basemat of a Seismic Category I structure" and that "values larger than this may be demonstrated acceptable by performing additional site specific evaluations."

The estimated differential settlements do not meet the U.S. EPR FSAR requirement of ½ inch per 50 ft (or 1/1,200); however, additional site specific evaluations will be performed to demonstrate their acceptability, as follows.

To verify that foundations perform according to estimates, and to provide an ability to make corrections, if needed, major structure foundations are monitored for rate of movement during and after construction.

Foundations are designed to safely tolerate the anticipated total and differential settlements. Additionally, engineering measures are incorporated into design for control of differential movements between adjacent structures, piping, and appurtenances sensitive to movement, consistent with settlement estimates. This includes the development and implementation of a monitoring plan that supplies and requires evaluation of information throughout construction and post-construction on ground heave, settlement, pore water pressure, foundation pressure, building tilt, and other necessary data. This information provides a basis for comparison with design conditions and for projections of future performance.

The estimated differential settlements represent departures from the U.S. EPR FSAR requirements. Additional discussion of the acceptability of these estimated differential settlements is provided in Section 3.8.5.

2.5.4.10.2.3 Settlement in the Intake Area

The settlement model in the Intake Area is developed in a similar form. The model is much simpler and the influence of neighboring structures is negligible. The size of the foundation is very small compared to the variability in layer thickness throughout the footprint. Soil layers, as shown in Figure 2.5-165 are horizontal. There is no additional complication introduced by asymmetric topography. The loading sequence for the Intake Area facilities is applied in a single step. Figure 2.5-196 provides the FEM model for the UHS MWIS and UHS EB facilities.

The total settlement at the end of construction for the facilities in the Intake Area is provided in Table 2.5-70. The maximum total settlement is 3.5 in and the maximum estimated tilt is 0.7 in/50 ft.

The 0.7 inch estimate for the Electrical Building will be mitigated during construction. The UHS settlement model applies building loads simultaneously. The tilt reported for the EB is influenced by the adjacent structures, but no construction baseline correction is performed. This correction cannot be applied in the model. However, in reality, The EB will be erected after the adjacent deeper founded structures. The foundation surface will be leveled prior to beginning construction of the building, This releveling will reduce tilt.}

2.5.4.10.3 Uniformity and Variability of Foundation Support Media

The U.S. EPR FSAR includes the following COL Item in Section 2.5.4.10.3:

> A COL applicant that references the U.S. EPR design certification will investigate and determine the uniformity of the underlying layers of site specific soil conditions beneath the foundation basemats. The classification of uniformity or non-uniformity will be established by a geotechnical engineer.

This COL Item is addressed as follows:

{Three criteria are identified in the U.S. EPR FSAR for establishing uniformity in foundation support media, namely, 1) presence of soil and rock, 2) dip angle of soil layers, and 3) shear wave velocity. Each is addressed below:

1. Foundations of all Seismic Category I structures at the CCNPP Unit 3 site are supported on compacted structural fill which is in turn supported on natural soils. Bedrock at the site is very deep, at about 2,500 ft below ground surface. Given the considerable depth to bedrock, non-uniform foundation conditions resulting from combined soil-rock support are not applicable to foundations at the CCNPP Unit 3 site.

2. Detailed subsurface information is presented in Section 2.5.4. Stratigraphic profiles indicate that the stratigraphic lines delineating various soil units have gentle slopes, mostly sloping about 1 to 2 degrees. This is consistent with the regional dip of 1 to 2 degrees in Coastal Plain deposits (refer to Section 2.5.1 for more details). However, at isolated CCNPP Unit 3 locations, stratigraphic units dip steeper, up to about 10 degrees which may be due to inherent assumptions in developing the stratigraphic lines or paleochannels and/or irregular erosional surfaces. Regardless, these steeper angles are less than the dip angle of 20 degrees from the horizontal identified in the U.S. EPR FSAR as the criterion for determining levelness of layers. On this basis, the soil layers at the CCNPP Unit 3 site are considered horizontal. However, the settlement analysis accounts for the variability in the soil media with the implementation of a FEM model as discussed in Section 2.5.4.10.4.

3. Classification of uniformity (or non-uniformity) in foundation support media resides with the geotechnical engineer, per the U.S. EPR FSAR. Shear wave velocity (V_s) measurements are used for this determination because they are a) in-situ measurements reflecting the natural ground conditions and b) important input to the safety evaluation of structures such as in soil-structure interaction and seismic analyses. The V_s values were evaluated to a depth of 344 ft below the Nuclear Island (NI) foundation basemat, corresponding to El. -300 ft. The 344 ft value was selected based on the three U.S. EPR FSAR criteria of: 1) 1.5 times an equivalent radius of foundation basemat, 2) 1.0 times the maximum foundation basemat dimension, or 3) no less than 200 ft below the bottom of the foundation basemat; with criterion (2) selected as the governing condition for the CCNPP Unit 3 NI basemat for its greater dimension. Minor appendages and protrusions in the irregularly-shaped U.S. EPR NI foundation were ignored in selecting the 344 ft value. The variations in shear wave velocity have been properly accounted for in the dynamic analysis by means of a best estimate soil profile.

Based upon the above, CCNPP Unit 3 is considered a Uniform Site.

2.5.4.10.4 Site Investigation for Uniform Sites

No departures or supplements.

2.5.4.10.5 Site Investigations for Non-uniform Sites

No departures or supplements.

2.5.4.10.6 Earth Pressure

Section 2.5.4.10.6 is added as a supplement to the U.S. EPR FSAR.

Static and seismic lateral earth pressures are addressed for below-grade walls. Seismic earth pressure diagrams are structure-specific. They are only addressed generically herein. Specific earth pressure diagrams are developed for specific structures based upon each structure's final configuration. Passive earth pressures are not addressed; they are excluded for conservatism for general purpose applications. Engineering properties for structural fill are used to estimate earth pressures. The properties of backfill are provided in Section 2.5.4.2.5.9. Structural backfill material is verified to meet the design requirements prior to use during construction. A surcharge pressure of 500 psf applied at the ground surface is assumed. The validity of this assumption will be confirmed during detailed design. Lateral pressures due to compaction are not included; these pressures are controlled by compacting backfill with light equipment near structures.

In developing the earth pressure diagrams, the following are assumed:

- Ground surface behind walls is horizontal,
- The side of the wall in contact with the backfill is vertical and there is no friction between the backfill and the wall,
- Retaining walls designed for the active earth pressure are allowed to move laterally, and building walls designed for the at-rest condition are prevented from moving laterally.
- Properties of backfill relevant to the earth pressure calculations are unit weight and angle of shearing resistance. These are provided in Table 2.5-51 and Table 2.5-54 respectively. The values are obtained from laboratory testing of backfill bulk samples and these are 145 pcf and 40°.
- Active and at rest earth pressure coefficients are provided in Table 2.5-58. These values are: $k_A = 0.22$, and $k_0 = 0.36$;
- For active and surcharge pressures, earthquake-induced horizontal ground accelerations are addressed by the application of k_h g. Vertical ground accelerations (k_v g) are considered negligible and are ignored (Seed, et al., 1970). A seismic horizontal acceleration of 0.15 g is conservatively assumed (consistent with the plant SSE.)

2.5.4.10.6.1 Static Lateral Earth Pressures

The static active earth pressure is estimated with the following equation (Lambe, et al., 1969):

$$p_{AS} = K_{AS}\gamma z$$

Where:

p_{AS}	→	Static active earth pressure;
K_{AS}	→	Active earth pressure coefficient from Table 2.5-58;
γ	→	Unit Weight of backfill;
z	→	Depth below ground surface;

The static at-rest earth pressure is estimated with the following equation (Lambe, et al., 1969):

$$p_{0S} = K_{0S}\gamma z$$

Where:

p_{0S}	→	At rest earth pressure;
K_{0S}	→	At rest earth pressure coefficient from Table 2.5-58;
γ	→	Unit Weight of backfill;

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z	→	Depth below ground surface;

Hydrostatic pressure is accounted for by assuming Groundwater Level at El. 55 ft, which is 13.5 ft above foundation level of the NI.

2.5.4.10.6.2 Seismic Lateral Earth Pressure

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

$$p_{AE} = \Delta K_{AE}\gamma(H-z)$$

Where:

p_{AE}	→	Active seismic pressure;
ΔK_{AE}	→	Coefficient of active seismic earth pressure ($K_{AE} - K_{AS}$);
K_{AE}	→	Mononobe-Okabe coefficient of active seismic earth thrust

$$K_{AE} = \frac{\cos^2(\phi'-\theta)}{\cos^2\theta\left(1+\sqrt{\frac{\sin\phi'\sin(\phi'-\theta)}{\cos\theta}}\right)^2}$$

θ	→	$\theta = \tan^{-1}(k_h)$
k_h	→	Seismic coefficient (0.15 g)
γ	→	Unit Weight of backfill;
H	→	Below-grade height of wall;
z	→	Depth below the top of the backfill;

The value ΔK_{AE} can be estimated as 0.75 k_h for k_h values less than about 0.25 g, regardless of the angle of shearing resistance of the backfill (Seed, et al., 1970).

The seismic at-rest pressure ΔK_{0E}, for below-grade walls for Category I structures is evaluated using a method that recognizes the frequency content of the design motion, limited building wall movements due to the presence of floor diaphragms, and uses the soil shear wave velocity and damping as input (Ostadan, 2004). To predict lateral seismic soil pressures for below-grade structural walls resting on firm foundations and assuming non-yielding walls, the method involves the following steps:

1. For conservatism, define the ground motion as the CCNPP Unit 3 Safe Shutdown Earthquake (SSE) peak ground acceleration. This value is the maximum spectral acceleration of the site specific spectra (See Section 3.7).

2. Compute the total mass for a representative Single Degree of Freedom (SDOF) system using Poisson's ratio and the mass density of the soil, m:

$$m = \frac{1}{2}\frac{\gamma}{g}H^2\psi_v$$

Where:

γ/g	→	Total mass density of the structural backfill;
H	→	Height of wall
ψ_v	→	Factor to account for Poisson's ratio (ν), with $\mid \nu \mid = 0.3$ adopted for structural backfill

$$\psi_v = \frac{2}{\sqrt{(1-\nu)(2-\nu)}}$$

3. Obtain the lateral seismic force as the product of the total mass obtained from Step 2, and 0.15 g.

4. Obtain the maximum lateral seismic soil pressure at the ground surface by dividing the lateral force obtained from Step 3 by the area under the normalized seismic soil pressure, or 0.744 H.

5. Obtain the soil pressure profile by multiplying the maximum pressure from Step 4 by the following pressure distribution relationship:

$$p(y) = -0.0015 + 5.05y - 15.84y^2 + 28.25y^3 - 24.59y^4 + 8.14y^5$$

Where:

y	→	Normalized height ratio (y/H). "y" is measured from bottom of the wall and y/H ranges from a value of zero at the bottom of the wall to a value of 1.0 at the top of the wall.

For well-drained backfills, seismic groundwater pressures need not be considered (Ostadan, 2004). Since granular backfill is used for the project, only hydrostatic pressures are taken into consideration. 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<0.1g), seismic groundwater considerations can be ignored.

Representative earth pressure diagrams are provided in Figure 2.5-197.

2.5.4.11 Design Criteria

No departures or supplements.

2.5.4.12 Techniques to Improve Subsurface Conditions

Major structures derive support from the very dense cemented soils or compacted structural backfill. Given the planned foundation depths and soil conditions at these depths, no special ground improvement measures are warranted. Ground improvement is limited to excavation of unsuitable soils, such as existing fill or loose/soft soils, and their replacement with structural backfill or lean concrete. It also includes 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

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dense or stiff foundation soils, is the primary focus of earthworks during foundation preparation. These measures include groundwater control, use of appropriate measures and equipment for excavation and compaction, subgrade protection, and other similar measures.

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2.5.5 STABILITY OF SLOPES

The U.S. EPR FSAR 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 (e.g., cuts, fill, embankments, dams, etc.), of which failure could adversely affect the safety of the plant.

This COL Item is addressed as follows:

{This section addresses the stability of constructed and natural slopes. 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). 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.

Information on site conditions and geologic features is provided in Section 2.5.1. Section 2.5.4 presents a discussion of the properties of the underlying soil and the backfill.

All elevations referenced in this section are based on National Geodetic Vertical Datum of 1929 (NGVD 29).

Sections 2.5.5.1 through 2.5.5.5 are added as a supplement to the U.S. EPR FSAR.

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

Site grading for CCNPP Unit 3 structures will include such areas as the powerblock, switchyard, cooling tower (collectively identified as the CCNPP Unit 3 area), the intake area and the utility corridor between the CCNPP Unit 3 area and the intake area. The powerblock includes the Reactor Building, Fuel Building, Safeguard Buildings, Emergency Power Generating Building (EPGB), Essential Service Water Building (ESWB), Nuclear Auxiliary Building (NAB), Access Building, Radioactive Waste Building, Turbine Building, Fire Protection Building and Switchgear Building. The intake area includes the Ultimate Heat Sink Makeup Water Intake Structure (UHS MWIS), Circulating Makeup Water Intake Structure (CW MWIS), UHS Electrical Building, Forebay and Fish Return. All the safety related structures are in these two areas. Natural ground surface elevations within the powerblock range from approximately Elevation 47 ft to Elevation 121 ft, and approximately Elevation 8 ft to Elevation 11 ft within the intake area, as shown in Figure 2.5-103. The centerline of the CCNPP Unit 3 powerblock is graded to approximately Elevation 85 ft. The finished grade in each major area will be approximately:

- Powerblock: Elevation 80 ft to Elevation 85 ft.

- Intake Area: Elevation 10 ft.
- Switchyard: Elevation 90 ft to 98 ft.
- Cooling Tower: Elevation 94 ft to 100 ft.
- Utility Corridor: Elevation 80 ft near proposed CCNPP Unit 3 to Elevation 8 ft nearthe Barge Slip.

Locations of these areas and associated structures, and a schematic of the overall grading configuration, are shown in Figure 2.5-198. The site grading within the powerblock will require both cut and fill, currently estimated at approximately 40 ft and 45 ft, respectively. The cut and fill operations will result in permanent slopes around the powerblock and Category I structures in the powerblock area. The maximum height of new slopes in the area of CCNPP Unit 3 powerblock is approximately 50 ft, located on the eastern side of the powerblock, sloping down from the powerblock.

The hill to the west of the intake area is approximately 90 ft high with a slope towards the east. The intake slope is constructed such that its toe is at least 100 ft from the intake structures.

An access road connects the CCNPP Unit 3 area and the Intake area. The cooling-water pipes and electrical duct banks are routed along the same alignment. This area is referred to as the 'Utility Corridor'. The maximum height of the slopes along the Utility Corridor is about 45 ft (from the road elevation 30 ft to top of slope elevation 75 ft).

Permanent slopes, whether cut or fill, will have an inclination of approximately 3:1 (horizontal to vertical). Earthworks for slope construction, including fill control, compaction, testing, etc. are addressed in Section 2.5.4.5.

Seven cross-sections that represent the typical site grading configuration were selected for evaluation based on location (e.g., proximity to Category I structures), slope geometry (e.g., height), and soil conditions. These cross-sections and their locations are shown in Figure 2.5-198 through Figure 2.5-200. Sections A, C, D and E are located in the powerblock area, Section B in the Construction Layout Area (CLA), Section F extends across the Utility Corridor, and Section G extends across the Intake Slope and Intake area. 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 west of the steep cliffs known as the Calvert Cliffs, as shown in Figure 2.5-198. These cliffs make up the Chesapeake Bay shoreline and reach elevations as high as 100 ft at their closest point to the CCNPP Unit 3 powerblock area. Stability of the Calvert Cliffs is discussed in Section 2.5.5.2.

2.5.5.1.3 Exploration Program and Geotechnical Conditions

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

A geotechnical subsurface investigation was performed to characterize the upper 400 ft of soil at the CCNPP Unit 3 site. The site geology, based on geotechnical borings beneath the CCNPP Unit 3 site is comprised of fluvial and marine deposits that are about 2500 ft thick. Only the deposits in the upper 150 ft are of interest for the slope stability analyses. The subsurface, in the upper 150 ft, is 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

Identification of soil layers was based on their physical and engineering characteristics. The characterization of the subsurface materials was based on a suite of tests consisting of standard penetration tests (SPT), in-soil borings including auto-hammer energy measurements, geophysical testing, and laboratory testing. Figure 2.5-106 provides an idealized profile for CCNPP Unit 3. Overall, the subsurface conditions encountered throughout the site are relatively uniform, as presented in detail in Section 2.5.4.

The first two soil layers, Terrace Sand and Chesapeake Clay/Silt IIa are not adequate foundation strata for safety related structures or facilities that will impose high contact pressures. These soils are susceptible to unacceptable levels of both elastic and long-term settlements. These soils will be removed in the powerblock area and replaced with Category I structural fill.

Based on the information provided in Section 2.4.12, in the powerblock area, shallow and deep groundwater regimes are present. For conservatism, the average groundwater level of Elevation 80 ft was chosen for slope stability evaluation in the powerblock, where in-situ soils were present. In locations where Category I structural fill replaced in-situ soils, the groundwater level was chosen as 55 ft. In the Intake Area, Intake Slope and Utility Corridor, the groundwater conditions are also based on the subsurface investigation and monitoring of observation wells. For conservatism, the groundwater levels in the Intake Area, Intake Slope and Utility Corridor were chosen as Elevations 10 ft, 37 ft and 24 ft, respectively. In naturally low-lying areas, that is, in area with ground surface elevations lower than 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 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.

The geotechnical parameters for the purpose of slope stability evaluation are based on material properties derived from the data collected during the exploration program. For the evaluation of the Utility Corridor, material properties based on data from the powerblock area were conservatively selected.

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. This approach results in a Factor of Safety (FOS) that can be defined as (Duncan, 1996):

$$\text{FOS} = \frac{\text{Shear Strength of Soil}}{\text{Shear Stress Required for Equilibrium}}$$

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 Morgenstern-Price method (Morgenstern, 1965). These methods are routinely used for the evaluation of slopes, and their limitations and advantages are well documented. The main differences are:

1. Static equilibrium equations.
2. Interslice forces that are included in the analysis.
3. Assumed relationship between the interslice shear and normal forces.

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

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

$$F_h = \left(\frac{a_h}{g}\right)W = k_h W$$

$$F_v = \left(\frac{a_v}{g}\right)W = k_v W$$

Where a_h and a_v 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 k_h and k_v 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 slope stability analysis was performed using SLOPE/W (GEO-SLOPE, 2007). SLOPE/W 2007 has been independently validated and verified using the Ordinary (Fellenius, 1936), Bishop's (Bishop, 1955) and Morgenstern-Price methods. 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 select the critical surface.

The initial code for SLOPE/W was developed by Professor D. G. Fredlund at the University of Saskatchewan in Canada. During the 1980s, the PC version became available. 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 these assumptions 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 porewater (suction) and externally applied forces, when needed. Material properties may simply be defined in terms of unit weight, friction and/or cohesion, or made a function of other parameters, e.g., change in stress. SLOPE/W has two options for evaluating slopes subjected to rapid loading; namely, pseudo-statically or using results from other dynamic analyses such as a companion program that obtains dynamic stresses and porewater pressure. A complete description of SLOPE/W and slope stability formulations is given in SLOPE/W user manual (GEO-SLOPE, 2007).

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 slope. The location and relative positions of safety-related structures to slopes in Sections A', G' and G" for the powerblock and intake area are shown in Figure 2.5-201 and Figure 2.5-202. 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 in the CCNPP Unit 3 site, the inertial effect coefficient $k_h = 0.15$ was used, based on $a_h = 0.15g$, as discussed in Section 2.5.4.7. The vertical component, k_v, was chosen as 0.075.

In the static analysis, a Mohr-Coulomb failure criterion based on effective stress conditions was used. For the sand layers, it is assumed that the effective cohesion, c', is equal to zero. This is a conservative approach which yields a lower factor of safety (FOS). The sand layers at the site contain varying amounts of clay and silt as shown in the boring logs provided in COLA Part 11J: Geotechnical Subsurface Investigation Data Report. The effective friction angle (ϕ') for the sand layers is based on standard penetration and cone penetration tests correlations, direct shear and CIU-bar triaxial compression tests. For the clay/silt layers, c' and ϕ' were obtained from the CIU-bar triaxial compression and direct shear tests.

Two cases were considered for the dynamic analysis:

- A Mohr-Coulomb failure criterion based on total stress conditions was used, to account for the hydrostatic pressure buildup. For the sand layers, total strength parameters (cohesion, c, and friction angle, ϕ) were obtained from CIU triaxial compression and direct shear tests. For the clay/silt layers, the undrained shear strength, s_u, obtained from Unconsolidated Undrained (UU) and Unconfined Compression (UC) tests was used (Table 2.5-54).

- A Mohr-Coulomb failure criterion based on effective stress conditions, using the same parameters as in the static analysis.

Material properties for the slope stability analysis are presented for the powerblock, utility corridor, and the intake slope and intake area in Table 2.5-71.

Result of the static and pseudo-static slope stability analyses for critical surfaces, that is, surfaces with the lowest FOS, are shown in Figure 2.5-203 through Figure 2.5-211. In these figures, TSA and ESA represent total stress analysis and effective stress analysis, respectively. The computed FOSs shown on these figures are based on the Morgenstern-Price method

(Morgenstern, 1965). Various runs were conducted on each slope to determine the lowest FOS. Sloughing or surficial failures that appeared during analyses were evaluated and disregarded when appropriate. For Sections A and B in the CCNPP Unit 3 area, two cases were considered: a) groundwater at the boundary between structural backfill and Chesapeake Sand, and b) groundwater located at Elevation 55 ft within structural backfill. In addition to the Morgenstern-Price method (Morgenstern, 1965), FOSs were also calculated using the Ordinary method and Bishop's simplified method (Bishop, 1955) for comparison. All three methods are implemented in SLOPE/W. The FOSs for these methods are summarized in Table 2.5-72, for effective stress and total stress conditions. The Ordinary method errs on the conservative side and yields lower FOSs because all interslice forces are ignored and only moment equilibrium is satisfied. The Bishop's method considers moment equilibrium and the normal interslice force. The Morgenstern-Price method considers moment and force equilibrium, and the interslice normal and shear forces. Both Bishop's and Morgenstern-Price methods yield higher FOSs.

An examination of the FOSs in Table 2.5-72 indicates that for the pseudo-static analyses (dynamic), the effective stress conditions yields lower FOSs. However, total stress conditions are more representative of dynamic conditions at the site since porewater pressures do not have time to dissipate. Results reported hereafter for pseudo-static analyses are based on total stress conditions.

In the powerblock and adjacent areas (Cross-sections A through E in Figure 2.5-199), all slopes show FOSs greater than 1.8 for the static case and greater than 1.6 for the pseudo-static case, based on the Morgenstern-Price method (Morgenstern, 1965), as shown in Figure 2.5-203 through Figure 2.5-209.

Along the Utility Corridor, at Cross-section F shown in Figure 2.5-200, a static FOS of 2.34 and a pseudo-static FOS of 2.82 was obtained with the Morgenstern-Price method, as shown in Figure 2.5-210.

In the intake area, at Cross-section G shown in Figure 2.5-200, a static FOS of 2.05 and a pseudo-static FOS of 1.93 were obtained using the Morgenstern-Price method, as shown in Figure 2.5-211.

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. The calculated FOSs for all slopes exceed the minimum acceptable values. Therefore, the slopes in the powerblock, intake area and utility corridor have sufficient static and dynamic stability against slope failure.

There are no dams or embankments that would affect the CCNPP Unit 3. Probable Maximum Flood (PMF) at the CCNPP Unit 3 area is accounted for by assuming a high groundwater level of 37 ft at the Intake Slope. A maximum flood level of 36.6 ft is postulated, this would only affect the Intake Slope.

2.5.5.2.2 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 to 4 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 to Elevation 100 ft, with a major portion maintaining about Elevation 90 ft, as shown in Figure 2.5-198. 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, resulting in moderate capillary forces and, therefore, enhanced stability and resistance to erosion.

The easternmost boundary of the CCNPP Unit 3 powerblock is set back a distance of about 1,000 ft from the cliffs, with at least 1,200 ft to the nearest Category I structure, as shown in Figure 2.5-198. 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 from the cliffs' edge would be impacted by such an unexpected scenario, and the remaining 900-plus ft 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 to 4 ft annually, at a constant rate, it will take approximately 25 to 50 years to erode about 100 ft of the cliffs. Or, it would take approximately 125 to 250 years for the cliffs to erode to within a distance of 500 ft from CCNPP Unit 3 outline (or 700 ft from any Category I structure). The estimated period of 125 to 250 years is 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.

2.5.5.2.3 Concluding Remarks

Based on analyses provided in this Section, 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.

2.5.5.3 Logs Of Borings

Logs of borings, and associated references, are provided in COLA Part 11J: Geotech Data Report.

2.5.5.4 Compacted Fill

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

2.5.5.5 References

BGE, 1992. Updated Final Safety Analysis Report, Calvert Cliffs Nuclear Power Plant (Units 1 and 2), Calvert County, Maryland, Docket Numbers 50-317 and 50-318, Baltimore Gas and Electric Company, 1992.

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GEO-SLOPE, 2007. Stability Modeling with SLOPE/W 2007, An Engineering Methodology, Second Edition, Geo-Slope/W International Ltd., 2007.

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NRC, 2007. Combined License Applications for Nuclear Power Plants (LWR Edition), Regulatory Guide 1.206, Revision 0, U.S. Nuclear Regulatory Commission, March 2007.}

2.5.6 REFERENCES

No departures or supplements.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-25—{Summary Thickness and Termination Elevation}

ENTIRE SITE		Thickness [feet]			Termination Elevation [feet]		
		Min	Max	Avg	Min	Max	Avg
Stratum I - Terrace Sand		1	68	28	32	82	61
Stratum IIa - Chesapeake Clay/Silt		4	36	19	5	67	43
Stratum IIb - Chesapeake Cemented Sand	Layer 1	3	69	24	-2	46	22
	Layer 2	3	55	23	-17	30	0
	Layer 3	4	39	16	-31	-9	-22
Stratum IIc - Chesapeake Clay/Silt		190	195	193	-215	-208	-211
Stratum III - Nanjemoy Sand		>101*	>115*	>108*	-	-	-
** Data based on borings B-301 and B-401*							
POWERBLOCK AREA		**Thickness [feet]**			**Termination Elevation [feet]**		
		Min	**Max**	**Avg**	**Min**	**Max**	**Avg**
Stratum I - Terrace Sand		1	52	21	45	79	62
Stratum IIa - Chesapeake Clay/Silt		4	30	18	34	55	45
Stratum IIb - Chesapeake Cemented Sand	Layer 1	8	45	26	3	43	20
	Layer 2	4	55	23	-17	28	-3
	Layer 3	5	39	16	-31	-9	-23
Stratum IIc - Chesapeake Clay/Silt		190	190	190	-208	-208	-208
Stratum III - Nanjemoy Sand		>101*	>101*	>101*	-	-	-
** Data based on borings B-301*							
INTAKE AREA		**Thickness [feet]**			**Termination Elevation [feet]**		
		Min	**Max**	**Avg**	**Min**	**Max**	**Avg**
Stratum I - Terrace Sand (NP)		-	-	-	-	-	-
Stratum IIa - Chesapeake Clay/Silt (NP)		-	-	-	-	-	-
Stratum IIb - Chesapeake Cemented Sand	Layer 1	5	5	5	3	3	3
	Layer 2	3	31	15	-12	-1	-8
	Layer 3	9	24	15	-28	-17	-22
Stratum IIc - Chesapeake Clay/Silt		>13	>141	>57	-	-	-
Stratum III - Nanjemoy Sand		-	-	-	-	-	-
** Data based on borings B-775 (NP) Not Present*							

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Table 2.5-26—{Summary of Field Tests}

Field Test	Standard	Number of Tests
Test Borings	ASTM D1586/1587	200
Observation Wells	ASTM D5092	47
CPT Soundings(1)	ASTM D5778	74*
Suspension P-S Velocity Logging	EPRI TR-102293	13
Test Pits	N/A	20
Field Electrical Resistivity Arrays	ASTM G57/IEEE 81	4
SPT Hammer Energy Measurements	ASTM D4633	10
Pressuremeter	ASTM D4719	2
Dilatometer	ASTM D6635	2

Notes:
- (1) Includes additional off-set soundings

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Table 2.5-27—{Summary of As-Conducted Boring Information}

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Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
			North	East		
B-301	403.0	-308.5	217024.1	960815.1	94.5	9/15/2006
B-301A	350.0	-253.3	217011.1	960816.8	96.7	11/21/2008
B-301B	120.0	-23.2	217002.6	960819.2	96.8	11/21/2008
B-302	200.0	-123.6	217122.2	960767.0	76.4	9/15/2006
B-303	200.0	-112.6	217016.9	960867.7	87.4	9/15/2006
B-304	200.0	-132.0	217188.6	960896.9	68.0	9/15/2006
B-305	151.5	-79.5	217166.3	960686.7	72.0	9/15/2006
B-306	150.0	-31.4	217024.3	960681.8	118.6	9/15/2006
B-307	201.5	-82.2	216955.3	960690.1	119.3	9/15/2006
B-308	150.0	-42.9	216906.7	960771.3	107.1	9/15/2006
B-309	150.0	-49.9	216949.2	960890.7	100.1	9/15/2006
B-310	100.0	-8.4	217081.4	960616.6	91.6	5/15/2006
B-311	150.0	-91.6	217268.6	960771.8	58.4	9/15/2006
B-312	99.5	-44.2	217293.0	960740.0	55.3	5/15/2006
B-313	150.0	-99.3	217372.3	960713.7	50.7	9/15/2006
B-314	100.0	-47.2	217321.9	960654.5	52.8	9/15/2006
B-315	100.0	-34.5	217184.7	960559.4	65.5	9/15/2006
B-316	100.0	8.1	216767.2	960864.4	108.1	9/15/2006
B-317	100.0	-5.6	217094.7	961249.2	94.4	5/15/2007
B-318	200.0	-102.2	217019.3	961227.2	97.8	5/15/2006
B-319	100.0	2.9	216963.6	961123.0	102.9	9/15/2006
B-320	150.0	-43.6	216943.5	961044.1	106.4	5/15/2006
B-321	150.0	-79.3	217152.5	960333.2	70.7	5/25/2006
B-322	100.0	-10.1	217170.0	960202.7	89.9	9/15/2006
B-323	200.0	-92.5	217028.0	960060.9	107.5	9/15/2006
B-324	101.5	3.7	216906.4	960114.4	105.2	9/15/2006
B-325	100.0	-15.0	216949.0	960549.7	85.0	9/15/2006
B-326	100.0	3.1	216859.2	960652.3	103.1	9/15/2006
B-327	150.0	-63.1	216865.7	960573.4	86.9	9/15/2006
B-328	150.0	-73.7	216828.9	960493.2	76.3	9/19/2006
B-329	100.0	-25.2	216800.4	960379.4	74.8	9/19/2006
B-330	100.0	-14.5	216715.4	960523.7	85.5	9/15/2006
B-331	100.0	-31.7	216970.6	960481.8	68.3	9/15/2006
B-332	100.0	-34.6	217127.4	960400.5	65.4	9/15/2006
B-333	98.8	-9.3	216657.0	960386.2	89.5	9/15/2006
B-334	100.0	-13.3	216515.5	960556.6	86.8	9/15/2006
B-335	100.0	-0.5	216732.7	960703.3	99.5	5/15/2006
B-336	100.0	-3.1	216632.9	960750.3	96.9	9/15/2006
B-337	100.0	-28.2	217257.9	960264.4	71.8	9/15/2006
B-338	99.6	-1.6	217121.1	960150.1	98.0	5/25/2006

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Table 2.5-27—{Summary of As-Conducted Boring Information}

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Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
			North	East		
B-339	100.0	-8.0	217095.2	960212.0	92.0	9/15/2006
B-340	100.0	-15.4	217171.3	961225.2	84.6	9/15/2006
B-341	100.5	-2.3	217036.4	961104.5	98.2	9/15/2006
B-342	250.0	-174.3	217217.6	960272.9	75.7	11/21/2008
B-343	250.0	-166.9	217037.8	960306.8	83.1	11/21/2008
B-344	250.0	-177.7	216976.8	960358.0	72.3	5/14/2008
B-345	250.0	-180.4	217097.3	960392.9	69.6	11/21/2008
B-346	100.0	-38.2	217206.4	960400.4	61.8	5/14/2008
B-347	200.0	-139.8	217214.2	960531.8	60.2	5/14/2008
B-348	200.0	-131.6	217148.9	960567.4	68.4	11/21/2008
B-349	100.0	-45.6	217396.4	960537.5	54.4	5/15/2008
B-350	100.0	-53.4	217516.2	960789.0	46.6	5/14/2008
B-351	100.0	-29.9	217072.1	960538.3	70.1	11/21/2008
B-352	200.0	-90.7	216829.4	960893.9	109.3	11/21/2008
B-353	200.0	-89.1	216772.7	960972.2	110.9	5/13/2008
B-354	251.5	-159.1	217131.1	961098.9	92.4	11/20/2008
B-355	250.0	-161.8	217052.6	960993.5	88.2	5/13/2008
B-356	250.0	-129.0	216965.3	961264.9	121.0	11/20/2008
B-357	105.0	-1.9	216923.1	961175.4	103.1	11/20/2008
B-357A	250.0	-147.0	216928.8	961167.0	103.0	11/20/2008
B-401	401.5	-329.4	216344.1	961516.8	72.1	9/15/2006
B-402	200.0	-117.8	216405.1	961463.5	82.2	5/15/2006
B-403	200.0	-136.6	216305.8	961562.9	63.4	5/15/2006
B-404	200.0	-132.1	216441.3	961596.5	67.9	9/21/2006
B-405	150.0	-28.0	216487.4	961408.7	122.0	9/15/2006
B-406	150.0	-31.6	216315.6	961352.0	118.4	9/15/2006
B-407	200.0	-118.4	216239.0	961412.5	81.6	9/15/2006
B-408	150.0	-81.6	216261.7	961482.0	68.4	9/15/2006
B-409	150.0	-88.5	216253.8	961614.8	61.6	4/20/2006
B-410	55.0	64.1	216374.3	961323.7	119.1	4/20/2006
B-410A*	98.7	20.4	216381.3	961323.7	119.1	4/20/2006
B-411	150.0	-68.6	216556.3	961517.2	81.5	9/15/2006
B-412	98.9	-6.7	216589.2	961495.4	92.2	9/15/2006
B-413	150.0	-27.1	216694.9	961413.3	122.9	9/15/2006
B-414	100.0	21.2	216630.2	961354.5	121.2	9/15/2006
B-415	98.7	20.6	216480.9	961264.2	119.3	4/20/2006
B-416	100.0	-13.8	216084.5	961596.3	86.2	9/15/2006
B-417	101.5	-52.3	216435.8	961901.1	49.2	9/15/2006
B-418	200.0	-156.3	216340.3	961976.7	43.7	9/22/2006
B-419	100.0	-44.7	216267.8	961895.6	55.3	9/21/2006

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Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
			North	East		
B-420	150.0	-87.4	216213.5	961670.4	62.6	9/15/2006
B-421	150.0	-34.4	216497.6	961019.8	115.6	9/15/2006
B-422	100.0	4.0	216478.2	960915.0	104.0	9/15/2006
B-423	201.5	-91.4	216331.8	960850.2	110.1	9/15/2006
B-424	100.0	18.9	216263.3	960818.6	118.9	4/26/2006
B-425	101.5	16.9	216247.5	961274.7	118.4	4/20/2006
B-426	100.0	-16.3	216193.0	961386.6	83.7	9/21/2006
B-427	150.0	-33.7	216164.1	961272.7	116.3	9/19/2006
B-428	150.0	-35.9	216109.2	961210.1	114.1	9/19/2006
B-429	100.0	3.7	216087.9	961119.3	103.7	9/19/2006
B-430	100.0	2.5	216006.9	961193.1	102.5	9/19/2006
B-431	101.5	16.9	216271.1	961177.3	118.4	4/20/2006
B-432	100.0	18.6	216399.0	961139.1	118.6	4/20/2006
B-433	100.0	-2.5	215963.8	961107.5	97.5	4/27/2006
B-434	100.0	5.2	215827.1	961244.3	105.2	5/2/2006
B-435	100.0	7.7	216020.1	961404.7	107.7	9/15/2006
B-436	100.0	8.3	215923.9	961441.6	108.3	9/22/2006
B-437	100.5	10.1	216521.8	960968.8	110.6	9/15/2006
B-438	6.5	99.5	216414.9	960848.9	106.0	9/28/2006
B-438A	100.0	6.6	216412.0	960867.3	106.6	9/28/2006
B-439	100.0	13.8	216340.5	960948.7	113.8	9/15/2006
B-440	100.0	-43.7	216349.5	961813.7	56.3	9/21/2006
B-701	75.0	-66.3	219485.5	960507.6	8.7	9/21/2006
B-702	50.0	-39.7	218980.6	961183.2	10.3	9/21/2006
B-703	100.0	-54.6	218171.0	960957.0	45.4	9/21/2006
B-704	50.0	-10.4	217991.1	960926.1	39.6	9/21/2006
B-705	50.0	-3.3	217581.3	960917.9	46.8	4/19/2006
B-706	50.0	27.4	217140.1	961339.7	77.4	9/21/2006
B-707	50.0	17.4	217397.0	961481.8	67.4	9/21/2006
B-708	100.0	-62.7	217585.8	961810.6	37.4	9/28/2006
B-709	50.0	-18.8	217642.8	961978.2	31.3	9/28/2006
B-710	75.0	-27.0	217542.5	962136.9	48.0	9/28/2006
B-711	50.0	3.0	216755.7	961743.5	53.0	4/19/2006
B-712	50.0	-7.6	216506.2	961997.6	42.4	9/22/2006
B-713	50.0	8.0	216117.7	962283.2	58.0	9/28/2006
B-714	50.0	66.0	215705.7	962034.4	116.0	10/16/2006
B-715	50.0	36.3	214951.8	962639.6	86.3	10/17/2006
B-716	49.5	32.9	215003.2	961364.6	82.4	10/16/2006
B-717	50.0	40.7	214302.5	962349.3	90.7	10/17/2006
B-718	50.0	67.5	214130.5	961929.1	117.5	10/18/2006

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Table 2.5-27—{Summary of As-Conducted Boring Information}

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Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
			North	East		
B-719	49.4	25.8	213978.7	961500.2	75.2	10/18/2006
B-720	75.0	-1.5	215674.5	962378.5	73.5	9/28/2006
B-721	100.0	1.3	215545.8	962462.1	101.3	5/4/2006
B-722	73.9	25.9	215386.1	962467.0	99.8	5/4/2006
B-723	75.0	15.0	215108.0	963000.8	90.0	4/28/2006
B-724	100.0	-3.0	214780.0	963106.2	97.0	4/28/2006
B-725	75.0	-16.0	214664.3	963219.4	59.0	4/28/2006
B-726	75.0	3.3	215564.7	961709.6	78.3	10/16/2006
B-727	100.0	4.9	215300.9	961885.0	104.9	10/16/2006
B-728	75.0	37.3	215163.6	961910.1	112.3	10/16/2006
B-729	75.0	42.3	214861.9	962454.6	117.3	10/17/2006
B-730	75.0	40.4	214728.5	962523.8	115.4	10/17/2006
B-731	99.3	16.4	214546.5	962547.9	115.7	10/17/2006
B-732	75.0	15.7	215034.1	961594.7	90.7	5/11/2006
B-733	100.0	-12.1	214866.8	961697.7	87.9	5/11/2006
B-734	75.0	30.7	214589.6	961812.5	105.7	5/9/2006
B-735	75.0	16.2	214805.5	961021.8	91.2	10/16/2006
B-736	75.0	23.3	214681.7	961154.3	98.3	10/16/2006
B-737	100.0	-36.5	214511.9	961147.4	63.5	10/16/2006
B-738	75.0	12.3	213826.3	961679.6	87.3	10/19/2006
B-739	99.8	0.5	213719.6	961793.3	100.4	10/19/2006
B-740	75.0	-0.7	213605.1	961781.1	74.3	10/19/2006
B-741	75.0	6.4	213760.5	961029.8	81.4	10/18/2006
B-742	100.0	2.4	213472.8	961217.2	102.4	10/18/2006
B-743	75.0	28.6	213315.7	961232.0	103.6	5/9/2006
B-744	100.0	13.3	216377.3	959963.4	113.3	9/29/2006
B-745	75.0	36.7	215971.2	960529.0	111.7	9/29/2006
B-746	75.0	7.8	215743.4	960721.4	82.8	9/29/2006
B-747	75.0	15.3	216176.3	959945.0	90.3	9/29/2006
B-748	100.0	-17.6	216039.7	960288.7	82.4	9/29/2006
B-749	75.0	27.5	215775.1	960332.2	102.5	9/29/2006
B-750	73.9	-1.6	215849.2	959930.1	72.4	9/29/2006
B-751	73.9	18.3	215588.9	960146.2	92.2	9/29/2006
B-752	100.0	-4.2	215489.2	960257.6	95.8	9/29/2006
B-753	40.0	8.8	217831.2	960648.9	48.8	9/21/2006
B-754	50.0	17.0	217369.8	960290.4	67.0	9/21/2006
B-755	40.0	55.0	215923.7	961637.9	95.0	9/22/2006
B-756	50.0	56.9	215504.6	961215.1	106.9	4/21/2006
B-757	40.0	66.9	215135.1	960760.6	106.9	10/16/2006
B-758	40.0	42.6	215133.3	960332.7	82.6	10/16/2006

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-27—{Summary of As-Conducted Boring Information}

(Page 5 of 5)

Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927) North	East	Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
B-759	100.0	-1.7	214526.3	960025.3	98.4	10/19/2006
B-765	102.0	-4.6	216424.5	959701.2	97.4	9/29/2006
B-766	50.0	58.9	216932.9	959791.5	108.9	9/19/2006
B-768	100.0	-51.6	217116.0	962243.0	48.4	9/28/2006
B-769	50.0	4.2	216589.8	962559.5	54.2	9/28/2006
B-770	50.0	71.6	215466.6	962827.0	121.6	10/18/2006
B-771	100.0	-89.4	219268.2	960931.9	10.6	7/1/2008
B-772	100.0	-89.4	219323.9	960876.1	10.6	7/1/2008
B-773	165.0	-157.1	219241.3	961045.9	7.9	7/1/2008
B-773A	150.0	-141.7	219233.1	961052.9	8.3	11/25/2008
B-773B	150.0	-142.0	219248.1	961039.9	8.0	11/25/2008
B-774	150.0	-139.9	219196.0	961000.5	10.1	7/1/2008
B-775	100.0	-90.3	219105.3	961091.5	9.7	7/1/2008
B-776	51.5	-41.9	219143.0	961053.7	9.6	7/14/2008
B-778	121.5	-7.9	219075.0	960739.6	113.6	11/25/2008
B-779	102.0	-1.2	218941.1	960604.8	100.8	7/2/2008
B-780	6.0	3.7	219546.2	960610.0	9.7	11/25/2008
B-780A	8.0	1.2	219542.4	960604.1	9.2	11/25/2008
B-780B	50.0	-40.8	219532.9	960625.2	9.2	11/25/2008
B-781	50.0	-39.6	219400.9	960780.8	10.4	7/14/2008
B-782	51.5	-41.6	218936.5	961232.1	9.9	7/1/2008
B-785	70.0	28.1	218155.9	960637.4	98.1	11/25/2008
B-786	11.5	50.5	217943.5	960500.5	62.0	11/25/2008
B-786A	80.0	-17.9	217943.2	960496.4	62.1	11/25/2008
B-786B	115.0	-60.8	217914.6	960460.7	54.2	11/25/2008
B-787	100.0	-50.6	217780.9	960598.1	49.4	11/25/2008
B-788	50.0	2.1	217495.9	960896.1	52.1	11/21/2008
B-789	100.0	-42.7	217401.7	960986.9	57.3	11/21/2008
B-790	49.7	23.0	217278.1	961110.5	72.7	5/13/2008
B-791	100.0	-12.5	217143.5	961245.1	87.5	5/13/2008
B-821	50.0	-41.1	218736.3	961124.6	8.9	7/1/2008
B-821A	115.0	-89.6	218571.3	960962.8	25.4	11/25/2008
B-821B	7.6	-1.3	218727.2	961275.2	6.3	11/25/2008
B-821C	30.0	-22.6	218739.5	961258.1	7.4	11/25/2008
B-822	50.0	-11.2	218440.2	960840.8	38.8	7/2/2008

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Table 2.5-28—{Summary of Standard Penetration Test Data}

ENTIRE SITE		SPT N VALUE [Blows /ft]			SPT N CORRECTED [Blows /ft]		
		Min	Max	Avg	Min	Max	Avg
Stratum I - Terrace Sand		0	70	11	0	91	16
Stratum IIa - Chesapeake Clay/Silt		3	100	10	4	100	14
Stratum IIb - Chesapeake Cemented Sand	Layer 1	4	100	59	6	100	82
	Layer 2	0	100	16	0	100	22
	Layer 3	10	100	43	14	100	60
Stratum IIc - Chesapeake Clay/Silt		5	100	20	7	100	28
Stratum III - Nanjemoy Sand		28	100	56	36	100	72
POWERBLOCK AREA		SPT N VALUE [Blows /ft]			SPT N CORRECTED [Blows /ft]		
		Min	Max	Avg	Min	Max	Avg
Stratum I - Terrace Sand		0	70	10	0	91	14
Stratum IIa - Chesapeake Clay/Silt		3	50	11	4	70	15
Stratum IIb - Chesapeake Cemented Sand	Layer 1	6	100	63	9	100	89
	Layer 2	1	100	17	1	100	24
	Layer 3	12	100	45	16	100	63
Stratum IIc - Chesapeake Clay/Silt		9	100	21	14	100	30
Stratum III - Nanjemoy Sand		34	100	58	44	100	75
INTAKE AREA		SPT N VALUE [Blows /ft]			SPT N CORRECTED [Blows /ft]		
		Min	Max	Avg	Min	Max	Avg
Stratum I - Terrace Sand		-	-	-	-	-	-
Stratum IIa - Chesapeake Clay/Silt		-	-	-	-	-	-
Stratum IIb - Chesapeake Cemented Sand	Layer 1	26	26	26	35	35	35
	Layer 2	1	100	12	1	100	17
	Layer 3	12	100	39	16	100	54
Stratum IIc - Chesapeake Clay/Silt		5	44	16	7	59	22
Stratum III - Nanjemoy Sand		-	-	-	-	-	-

Notes:
- A cut-off value of 100 blows/ft is used

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-29—{Summary Undisturbed Tube Samples}

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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-301A	U. TRUCK	8/18/2008	UD-1	58.0 - 58.8	9	SP
			UD-2	60.0 - 61.9	23	SC
			UD-3	68.0 - 69.8	22	SM
			UD-4	198.0 - 199.9	23	MH
			UD-5	218.0 - 219.9	23	SM
			UD-6	238.0 - 239.9	23	MH
			UD-7	258.0 - 260.0	24	MH
			UD-8	268.0 - 269.8	22	MH
			UD-9	278.0 - 279.9	23	MH
			UD-10	288.0 - 290.0	24	MH
			UD-11	298.0 - 300.0	24	MH
			UD-12	308.0 - 309.9	23	SC
			UD-13	318.0 - 319.9	23	SC
			UD-14	328.0 - 330.0	24	SC
			UD-15	338.0 - 339.8	22	SC
			UD-16	348.0 -350.0	24	SM
B-301B	U. TRUCK	8/25/2008	UD-1	78.0 - 80.0	24	SM
			UD-2	88.0 - 89.9	23	SM
			UD-3	98.0 - 100.0	24	SM
			UD-4	108.0 -110.0	24	SM
			UD-5	118.0 -120.0	24	SM
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

Table 2.5-29—{Summary Undisturbed Tube Samples}
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Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
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
			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
B-314	U. ATV	5/22/2006	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

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-29—{Summary Undisturbed Tube Samples}

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Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
B-324	U. ATV	6/7/2006	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	sandy lean clay, bottom 2" bent
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
			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
			UD-2	94.5 - 95.0	?	not on boring log
			UD-3	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	C.TRACK	8/4/2006	UD-1	88.5 - 90.5	24	SM
			UD-2	98.5 - 100.5	24	SP-SM

Table 2.5-29—{Summary Undisturbed Tube Samples}

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Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
B-344	C. ATV	7/24/2008	UD-1	181.5 - 182.8	16	SM
			UD-2	191.5 - 193.4	23	SM
			UD-3	201.5 - 202.5	12	SM
			UD-4	204.0 - 206.0	24	SM
			UD-5	211.5 - 213.5	24	SM
			UD-6	221.5 - 223.5	24	ML
			UD-7	231.5 - 233.5	24	ML
			UD-8	241.5 - 243.5	24	ML
B-354	C. ATV	7/3/2008	UD-1	196.5 - 197.3	10	SM
			UD-2	197.3 - 199.3	24	SM
			UD-3	206.5 - 208.5	24	SM
			UD-4	216.5 - 218.5	24	SP-SM
			UD-5	226.5 - 228.5	24	SM
			UD-6	236.5 - 238.5	24	SM
			UD-7	246.5 - 248.1	19	SM
B-355	C. ATV	7/15/2008	UD-1	191.5 - 193.4	23	ML
			UD-2	201.5 - 203.4	23	SM
B-356	C-TRUCK	7/16/2008	UD-1	221.5 - 222.6	13	ML
			UD-2	223.0 - 224.5	18	ML
			UD-3	231.5 - 233.5	24	SM
			UD-4	241.5 - 243.5	24	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 2.5-29—{Summary Undisturbed Tube Samples}

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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-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	C. ATV	5/4/2006	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 2.5-29—{Summary Undisturbed Tube Samples}
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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	U.TRUCK	7/10/2006	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	C.TRUCK	6/28/2006	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	U. ATV	5/9/2006	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

Table 2.5-29—{Summary Undisturbed Tube Samples}
(Page 7 of 10)

Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
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
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	C. TRACK	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	C. TRACK	6/15/2006	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	C.TRACK	7/5/2006	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-1	70 - 72	8	cemented fine sandy silt, trace clay, trace shells
			P-2	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
B-771	C. TRACK	7/24/2008	UD-1	31.5 -33.5	24	SM
			UD-2	41.5 - 43.5	24	SM
			UD-3	51.5 - 53.5	24	SP-SM
			UD-4	61.5 - 63.5	24	SM
			UD-5	71.5 - 73.5	24	ML
			UD-6	81.5 - 83.5	24	ML
			UD-7	91.5 - 93.5	24	ML
B-772	C. TRACK	7/29/2008	UD-1	41.5 - 43.5	24	SM
			UD-2	51.5 - 52.6	13.5	SM
			UD-3	56.5 - 58.5	24	ML

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Table 2.5-29—{Summary Undisturbed Tube Samples}
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Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
B-773A	C. TRUCK	8/7/2008	UD-1	13.0 - 15.0	24	SM
			UD-2	23.0 - 24.3	15.5	SC
			UD-3	33.0 - 34.6	9	ML
			UD-4	43.0 - 45.0	24	SM
			UD-5	53.0 - 55.0	24	SM
			UD-6	63.0- 65.0	24	SM
			UD-7	73.0 - 75.0	24	MH
			UD-8	83.0 - 84.6	19	MH
			UD-9	93.0 - 94.8	21	SC
			UD-10	103.0 - 105.0	24	MH
			UD-11	113.0 - 114.8	22	SM
			UD-12	123.0 - 124.9	23	SM
			UD-13	136.0 - 137.8	22	SM
			UD-14	148.0 - 150.0	24	MH
B-773B	U. TRUCK	10/16/2008	UD-1	5.0 - 7.0	24	SM
			UD-2	15.0 - 16.8	22	SM
			UD-3	25.0 - 26.8	21	SC
			UD-4	35.0 - 37.0	24	ML
			UD-5	45.0 - 46.9	23	SM
			UD-6	55.0 - 57.0	24	SM
			UD-7	65.0 - 67.0	24	SM
			UD-8	75.0 - 77.0	24	MH
			UD-9	85.0 - 87.0	24	MH
			UD-10	95.0 - 97.0	24	SC
			UD-11	105.0 - 107.0	24	MH
			UD-12	115.0 - 116.5	18	SM
			UD-13	125.0 - 127.0	24	SM
			UD-14	135.0 - 137.0	24	SM
			UD-15	145.0 - 147.0	24	MH
B-774	U.ATV	7/30/2008	UD-1	11.5 - 13.1	19	SP-SM
			UD-2	16.5 -17.9	16.5	SM
			UD-3	21.5 - 23.4	23	SM
			UD-4	31.5 - 33.4	23	SM
			UD-5	41.5 - 43.5	24	SM
			UD-6	51.5 - 53.5	24	SM
			UD-7	81.5 - 83.3	22	MH
			UD-8	101.5 - 103.5	24	SM
			UD-9	111.5 - 113.4	23	SM
			UD-10	121.5 - 123.0	18	SM
			UD-11	131.5 - 133.4	22.5	SM
			UD-12	141.5 - 143.2	20	MH

Table 2.5-29—{Summary Undisturbed Tube Samples}
(Page 9 of 10)

Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
B-776	C. TRACK	7/22/2008	UD-1	36.5 - 38.2	20	ML
			UD-2	46.5 - 47.8	16	SM
B-778	C. TRACK	8/18/2008	UD-1	6.5 - 8.5	24	SM
			UD-2	11.5 - 13.5	24	SM
			UD-3	21.5 - 22.4	11	SM
			UD-4	23.5 - 24.5	12	SP-SM
			UD-5	31.5 - 32.5	12	SP-SM
			UD-6	33.5 - 34.4	10.5	SP-SM
			UD-7	41.5 - 43.1	19	CL
			UD-8	51.5 - 53.5	24	ML
			UD-9	61.5 - 63.5	24	GP
			UD-10	71.5 - 73.5	24	ML
			UD-11	81.5 - 83.5	24	ML
			UD-12	91.5 - 92.5	12	GP
			UD-13	93.5 - 94.7	14	SP
			UD-14	101.5 - 103.2	20	SP
			UD-15	111.5 - 113.5	24	SM
B-779	C. TRACK	8/13/2008	UD-1	6.5 - 8.3	21	SP
			UD-2	11.5 -13.5	24	SM
			UD-3	21.5 - 23.5	24	CL
			UD-4	31.5 - 33.5	24	SP
			UD-5	41.5 - 43.5	24	SM
			UD-6	51.5 - 52.5	12	ML
			UD-7	53.5 - 55.5	24	ML
			UD-8	61.5 - 63.5	24	ML
			UD-9	71.5 - 73.3	22	SM
			UD-10	81.5 - 82.8	16	SP-SM
			UD-11	96.5 - 97.7	14	SP-SM
			UD-12	100.0 - 102.0	24	SP-SM
B-782	C. TRACK	7/23/2008	UD-2	46.5 - 47.3	9	SM
B-786B	C. TRACK	11/6/2008	UD-1	5.0 - 7.0	24	SP-SM
			UD-2	15.0 -16.5	18	SM
			UD-3	25.0 - 26.0	12	SP
			UD-4	27.0 - 28.8	21	CH
			UD-5	35.0 - 36.7	20	CL
			UD-6	45.0 - 46.5	18	SM
			UD-7	55.0 - 57.0	24	SP-SM
			UD-8	65.0 - 66.8	22	SP-SM
			UD-9	75.0 - 76.8	21	SP-SM
			UD-10	85.0 -87.0	24	ML
			UD-11	95.0 - 97.0	24	SM

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Table 2.5-29—{Summary Undisturbed Tube Samples}

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Boring	Drill Rig	Date	Sample No.	Depth [ft]	Rec [in]	Field Remarks
B-821A	C. TRACK	11/11/2008	UD-1	10.0 - 11.2	14	SP-SM
			UD-2	12.0 - 13.0	12	SP-SM
			UD-3	20.0 - 22.0	24	SP-SM
			UD-4	30.0 - 32.0	24	SM
			UD-5	40.0 - 41.5	18	SM
			UD-6	50.0 - 52.0	24	ML
			UD-7	60.0 - 62.0	24	ML
			UD-8	70.0 - 71.0	12	SM
			UD-9	72.0 - 73.0	12	SM
			UD-10	80.0 - 82.0	24	ML
			UD-11	90.0 - 90.9	11	ML
			UD-12	92.0 - 93.6	19	SM
			UD-13	100.0 - 101.8	21	ML

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Table 2.5-30—{Summary of Hammer Rod Energy Measurements}

Drill Rig	Boring	ETR Range [%]	Average ETR [%]	Adjustment [ETR%/60%]
Failing 1500 Truck	B-401	67-88	78	1.3
CME 550X ATV	B-403	73-92	84	1.4
CME 750 ATV	B-404	78-90	87	1.45
CME 75 Truck	B-409	69-90	84	1.4
Diedrich D50 ATV	B-744	73-84	81	1.35
CME 75 ATV (Phase II)	B-348 & B-357	77-95	90	1.5
CME 550X ATV (Phase II)	B-354	79-90	83	1.38
Diedrich D50 ATV (Phase II)	B-791	74-85	81	1.35
CME 75 Truck (Phase II)	B-356	86-92	90	1.5

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Table 2.5-31—{Summary As-Conducted CPT Information}

(Page 1 of 2)

Location	Depth [ft]	Bottom Elevation [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Ground Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey	Remarks PD: Pre-Drill S: Seismic D: Dissipation		
			North	East			PD	S	D
C-301	52.3	42.5	217041.78	960820.13	94.84	9/15/2006		✓	
C-302	61.7	29.2	217088.9	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	-43.5	217026.56	960817.55	94.51	7/26/2006	✓85 ft		✓
C-303	25.4	36.2	217230.6	960804	61.58	4/24/2006			
C-303a*	47.1	14.5	217230.6	960804	61.58	7/25/2006	✓45 ft		
C-303a-1*	71.4	-9.8	217230.6	960804	61.58	7/25/2006	✓50 ft		
C-303b*	123.4	-61.8	217230.6	960804	61.58	7/25/2006	✓80 ft		✓
C-304	26.7	34.3	217235.29	960606.73	60.95	9/15/2006		✓	✓
C-305	74.3	41.6	216876.5	960961.5	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	42.3	216853.68	961079.64	117.64	9/15/2006		✓	
C-308	48.2	36.1	217129.9	960263.7	84.33	5/1/2006		✓	
C-309	70.1	35.9	217045.62	960110.76	106.04	9/15/2006			✓
C-311	34.9	39.1	216869.75	960488.16	73.97	9/15/2006			
C-312	56.4	43.4	216799.2	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	216531.4	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	-63.7	216381.06	961570.89	67.46	7/27/2006	✓85 ft	✓	✓
C-402	34.5	38.6	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	39.1	216524.3	961308.9	119.21	4/20/2006		✓	✓
C-405	40	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	216159.2	961732.2	63.23	6/22/2006		✓	✓
C-407-2a*	96.3	-33.1	216161.5	961726.7	63.23	7/28/2006	✓50 ft		✓
C-407-b*	142.4	-79.2	216161.5	961726.7	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	116.6	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	99.9	9/28/2006			
C-414	62.5	39.9	215893.42	961201.1	102.36	9/28/2006			✓

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Table 2.5-31—{Summary As-Conducted CPT Information}

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Location	Depth [ft]	Bottom Elevation [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Ground Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey	Remarks PD: Pre-Drill S: Seismic D: Dissipation		
			North	East			PD	S	D
C-415	20	36.6	216305.7	961857.4	56.63	5/26/2006			
C-701	29.5	-18.6	219262.19	960933.61	10.95	9/21/2006			✓
C-701a*	28.1	-17.2	219265.39	960936.81	10.95	7/21/2006			
C-702	20.3	-9	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	-2.8	217500.74	961710.02	45.36	9/28/2006			
C-705	34	-2.9	217637.26	961983.1	31.08	9/28/2006			
C-706	50	55.3	216958.95	961494.86	105.28	9/21/2006			
C-707	19.5	20.9	216308.12	962079.42	40.35	9/22/2006			
C-708	50	63	215658.28	961962.86	112.97	10/16/2006			
C-709	50	61.7	215027.59	962824.89	111.73	10/18/2006			
C-710	21.2	85	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	214920.3	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	78.2	214025.3	961636.9	90.21	10/18/2006			
C-720	70.7	28	213593.77	961134.09	98.66	10/18/2006			✓
C-721	52	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	97.6	9/29/2006			✓
C-724	152.2	-144.3	219309.8	960973.5	7.9	8/6/2008	✓	✓	
C-724A	13.3	-5.4	219309.3	960973.9	7.9	8/6/2008		✓	
C-725	152.4	-144.2	219157.7	961143.9	8.2	8/7/2008	✓	✓	
C-726	52.5	-43.3	219479.9	960691.8	9.2	8/6/2008			
C-727	101.1	-92.9	219368.3	960914.9	8.2	8/6/2008	✓		✓
C-728	52.8	-42.8	218975.5	961193	10	8/5/2008			
C-747	52.8	-43.7	218860.2	961248.5	9.1	8/4/2008	✓		
C-748	41.3	-8.9	218521.4	960909.8	32.4	8/20/2008			
C-748A	52	-19.7	218518.9	960908.7	32.3	8/21/2008			
C-749	18.4	43.9	218344.5	960737.8	62.3	8/20/2008			
C-749A	41.2	21.1	218346.4	960740	62.3	8/21/2008	✓		

Notes:
- (*) Location and elevation approximated based on offset observed in the field and recorded on Field Checklist

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Table 2.5-32—{Summary of As-Conducted Observation Well Information}

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Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Elevation (Top of Concrete at Base of Well Head Protector) [ft]	Elevation GW Level Measuring Point (V-Notch) [ft]	Date of As Built Survey
			North	East				
OW-301	80	14.5	217048.02	960814.47	94.51	94.78	96.27	9/15/2006
OW-313A	57.5	-6.5	217367.31	960705.3	51.03	51.31	53.2	9/15/2006
OW-313B	110	-59.3	217372.34	960713.67	50.73	51.16	53.54	9/15/2006
OW-319A	35	68.1	216962.56	961116.12	103.13	103.31	104.91	9/15/2006
OW-319B	85	18.5	216957.32	961125.02	103.53	103.85	105.35	9/19/2006
OW-323A	43.5	63.5	217034.46	960057.07	106.96	107.55	109.69	9/19/2006
OW-328	72	4.3	216828.86	960493.21	76.29	76.55	77.85	9/19/2006
OW-336	74	23.1	216643.18	960746.61	97.11	97.5	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	73.2	216703.14	961418.81	123.15	123.51	125.04	9/15/2006
OW-413B	125	-2.1	216694.88	961413.25	122.9	123.25	124.85	9/15/2006
OW-418A	40	3.7	216340.41	961966.46	43.66	44.31	45.83	9/22/2006
OW-418B	92	-48.3	216340.25	961976.71	43.67	44.13	45.77	9/22/2006
OW-423	43	68.1	216339.99	960882.24	111.12	111.67	113.16	9/15/2006
OW-428	50	63.9	216105.21	961212.38	113.92	114.32	115.92	9/19/2006
OW-436	50	58.1	215922.47	961446.87	108.13	108.53	110.39	9/22/2006
OW-703A	49	-5	218171.23	960967.72	44.02	44.44	45.65	9/21/2006
OW-703B	80	-34.4	218171.67	960958.91	45.57	45.97	47.53	9/21/2006
OW-705	52	-4.3	217566.62	960917.18	47.71	47.77	50.22	9/15/2006
OW-708A	34	3.4	217586.23	961803.52	37.44	37.82	39.61	9/28/2006
OW-711	50	2.9	216748.48	961741.61	52.92	53.26	55.31	9/22/2006
OW-714	50	66	215705.73	962034.37	116.02	116.32	117.98	10/16/2006
OW-718	43	75.5	214133.58	961924.87	118.53	118.96	120.41	10/18/2006
OW-725	60	-2	214649.3	963212.73	58.04	58.38	59.94	10/18/2006
OW-729	42	76.9	214872.58	962445.93	118.88	119.44	121.11	10/17/2006
OW-735	72	19.2	214805.48	961021.83	91.2	91.81	93.44	10/16/2006
OW-743	55	48.7	213320.62	961234.01	103.65	104.05	105.89	10/18/2006
OW-744	50	47.5	216405.37	960089.41	97.5	97.96	99.81	9/29/2006
OW-752A	37	58.3	215482.18	960250.12	95.3	95.73	97	9/29/2006
OW-752B	97	-1.2	215489.21	960257.57	95.79	96.09	97.41	9/29/2006
OW-754	44	23	217369.78	960290.37	67	67.21	68.85	9/15/2006
OW-756	42	64.6	215497.07	961212.39	106.56	107.07	108.77	10/16/2006
OW-759A	35	62.8	214536.47	960055.02	97.78	98.05	99.69	10/19/2006
OW-759B	90	8.4	214526.25	960056.32	98.35	98.72	100.14	10/19/2006
OW-765A	29	68.4	216424.51	959701.22	97.37	97.92	99.6	9/29/2006
OW-765B	102	-5.8	216420.42	959693.64	96.82	97.19	98.47	9/29/2006
OW-766	37	71.9	216932.89	959791.5	108.89	109.32	110.72	9/19/2006
OW-768A	42	6.5	217106.06	962238.98	48.48	48.96	49.84	9/28/2006

Table 2.5-32—{Summary of As-Conducted Observation Well Information}

(Page 2 of 2)

Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Elevation (Top of Concrete at Base of Well Head Protector) [ft]	Elevation GW Level Measuring Point (V-Notch) [ft]	Date of As Built Survey
			North	East				
OW-769	42	12.2	216589.75	962559.47	54.23	54.39	56.43	9/28/2006
OW-770	42	79.6	215466.6	962826.95	121.59	121.79	123.08	10/18/2006
OW-304	72.8	-4	217158.1	960920.8	68.8	69.28	71.01	7/17/2008
OW-308	103	8.4	216928	960750	111.4	111.95	113.62	7/17/2008
OW-774A	23	-13.3	219187.3	961030.5	9.7	10.2	12.2	7/31/2008
OW-774B	52.8	-42.7	219176.7	961020.2	10.1	10.5	12.55	7/31/2008
OW-778	52	61.3	219100.6	960728.6	113.3	113.7	115.45	8/27/2008
OW-779	52.5	48.4	218958.7	960587.3	100.9	101.3	102.94	8/27/2008
OW-781	53	-42.7	219421.3	960764.4	10.3	10.8	12.87	7/29/2008

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-33—{In-Situ Hydraulic Conductivity (Slug) Test Results}
(Page 1 of 2)

Location	Screened Interval Depth [ft]	USCS Soil Classification	Hydraulic Conductivity [fps]
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-323A	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-708A	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-768A	30 – 40	SM	$5.29X10^{-6}$
OW-769	32 – 42	SM, SC	$1.74X10^{-6}$
OW-304	60 - 70	SM	$4.31X10^{-6}$
OW-308	90 - 100	SP-SM	$1.87X10^{-5}$
OW-774A	20-Oct	SM	$2.72X10^{-5}$

Table 2.5-33—{In-Situ Hydraulic Conductivity (Slug) Test Results}
(Page 2 of 2)

Location	Screened Interval Depth [ft]	USCS Soil Classification	Hydraulic Conductivity [fps]
OW-774B	40 - 50	SC	$1.44X10^{-7}$
OW-778	40 - 50	ML,CH	Dry
OW-779	40 - 50	CH	Dry
OW-781	40 - 50	SM,ML	$4.01X10^{-7}$

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-34—{Summary As-Conducted Test Pit Information}

Location	Depth [ft]	Termination Elevation (Bottom) [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
			North	East		
TP-B307	6.7	112.7	216957.53	960690.62	119.35	9/19/2006
TP-B314	9	43.8	217320.35	960658.25	52.78	9/15/2006
TP-B315	8.5	57.3	217182.5	960563.12	65.8	9/15/2006
TP-B334	10	77	216515.64	960560.94	87.03	9/19/2006
TP-B335	8	91.6	216730.79	960706.97	99.64	9/19/2006
TP-B407	7	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	97.9	216414.95	960849.03	105.86	9/19/2006
TP-B434	8.5	96.7	215825.9	961244.18	105.24	9/22/2006
TP-B435	10	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	82.5	214297.68	962346.36	90.53	10/17/2006
TP-B719	8	64.3	213966.93	961493.94	72.28	10/18/2006
TP-B727	7	97.3	215299.14	961883.13	104.33	10/16/2006
TP-B744	6.5	106.8	316377.3	959963.38	113.28	9/29/2006
TP-B758	9	73.6	215133.29	960332.67	82.63	10/16/2006
TP-C309	8	100.5	217020.05	960105.24	108.45	9/19/2006
TP-C723	7	89.8	215989.07	959754.78	96.75	9/29/2006

Table 2.5-35—{Summary of Field Electrical Resistivity Information}

Location	Depth [ft]	Coordinates [ft], Maryland State Plane (NAD 1927)		Surface Elevation [ft] (NGVD 1929)	Date of As Built Survey
		North	East		
R-1	6.7	215837.3	960255.8	85.45	5/3/2006
R-2	9	215837.3	960255.8	85.45	5/3/2006
R-3	8.5	216622.5	960406.8	89.12	5/2/2006
R-4	10	215915.4	961114	99.4	4/27/2006

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-36—{Field Electrical Resistivity}

Spacing [ft]	Location R1 El. 85.5	R2 El. 85.5	R3 El. 89.1	R4 El. 99.4	Min	Max	Avg
	Measured Resistivity [ohm -m]				[ohm -m]	[ohm -m]	[ohm -m]
1.5	1210	1520	3070	471	471	3070	1568
3.0	2480	2410	3750	640	640	3750	2320
5.0	3220	2780	4550	660	660	4550	2803
7.5	3110	2890	5440	806	806	5440	3062
10.0	2490	2700	6240	1130	1130	6240	3140
15.0	1870	2780	5370	1340	1340	5370	2840
20.0	1570	1960	4100	1790	1570	4100	2355
30.0	1310	2060	1960	1640	1310	2060	1743
40.0	739	1590	1010	1280	739	1590	1155
50.0	314	1080	415	975	314	1080	696
100.0	45	487	69	463	45	487	266
200.0	37	116	38	57	37	116	62
300.0	48	76	31	41	31	76	49

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-37—{Geophysical Data from CCNPP Units 1 and 2 UFSAR}

STATION	Compressional Velocities							
	Surficial Sediments (Pleistocene)		Unconsolidated Sediments (Tertiary)		Intermediate Sediments (Cretaceous)		Basement Rock	
	Wave Velocity [fps]	Thickness [ft]	Wave Velocity [fps]	Thickness [ft]	Wave Velocity [fps]	Thickness [ft]	Wave Velocity [fps]	Thickness [ft]
Solomons Shoal	–	–	5900	3080	–	–	15,170	3130
Solomons Deed	–	–	6080	1070	6980	1900	18,100	3080
Site	2200	40	5500	–	–	–	–	–
Site	–	–	5900	–	–	–	–	–

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-38—{Pressuremeter Test Results, PM-301}

(Page 1 of 2)

Test	Depth [ft]	El. [ft]	Layer/Material		G [ksf]	$G_{u/r}$ [ksf]	ν	E [ksf]	$E_{u/r}$ [ksf]	$E_{u/r}$ / E
CC30	9.0	85.5	I	sand, some gravels	38	63	0.30	99	164	1.6
CC31	18.0	76.5	I	sand, some gravels	148	993	0.30	386	2581	6.7
CC32	29.5	65.0	I	clayey sand, silt	80	787	0.30	209	2045	9.8
CC33	28.0	66.5	I	clayey sand, silt	104	993	0.30	271	2581	9.5
CC34	41.0	53.5	IIa	sandy clay, lean clay	219	619	0.45	635	1795	2.8
CC35	39.5	55.0	IIa	sandy clay, lean clay	230	583	0.45	668	1690	2.5
CC36	51.0	43.5	IIb-1	interbedded fat clay with silty sand	508	1758	0.30	1322	4571	3.5
CC37	49.5	45.0	IIb-1	interbedded fat clay with silty sand	454	999	0.30	1179	2598	2.2
CC38	60.8	33.7	IIb-1	clayey sand, some cementation	859	3517	0.30	2232	9143	4.1
CC39	59.3	35.2	IIb-1	clayey sand to sand	572	1985	0.30	1488	5162	3.5
CC40	70.4	24.1	IIb-1	interbedded cemented sand, silt	963	9600	0.30	2504	24960	10.0
CC41	68.9	25.6	IIb-1	interbedded cemented sand, silt	637	6600	0.30	1656	17160	10.4
CC42	80.9	13.6	IIb-2	interbedded sand and clay	644	4705	0.30	1674	12232	7.3
CC43	79.4	15.1	IIb-2	interbedded sand and clay	255	3136	0.30	663	8155	12.3
CC44	91.0	3.5	IIb-2	interbedded cemented sand, silt	625	5280	0.30	1625	13728	8.4
CC45	89.5	5.0	IIb-2	interbedded cemented sand, silt	731	9827	0.30	1900	25551	13.4
CC46	101.0	-6.5	IIb-2	silty sand, some cementation	510	3517	0.30	1326	9143	6.9
CC47	99.5	-5.0	IIb-2	silty sand	508	2271	0.30	1322	5904	4.5
CC48	111.0	-16.5	IIb-3	silty sand, trace clay, cementation	1017	6273	0.30	2643	16310	6.2
CC49	109.5	-15.0	IIb-3	silty sand, trace clay	731	2839	0.30	1900	7382	3.9
CC50	120.8	-26.3	IIb-3	interbedded silty sand, sandy clay	731	3517	0.30	1900	9143	4.8
CC51	119.3	-24.8	IIb-3	interbedded cemented sand, silt	907	6273	0.30	2359	16310	6.9
CC52	131.0	-36.5	IIc	sandy clay, clayey sand, silt	510	2358	0.45	1479	6839	4.6
CC53	129.5	-35.0	IIc	sandy clay, clayey sand, silt	454	1985	0.45	1315	5757	4.4
CC54	141.0	-46.5	IIc	clayey sand, sandy clay	417	2580	0.45	1208	7482	6.2
CC55	139.5	-45.0	IIc	clayey sand, sandy clay	510	1999	0.45	1479	5797	3.9
CC56	151.0	-56.5	IIc	sandy clay	564	2580	0.45	1637	7482	4.6
CC57	149.5	-55.0	IIc	sandy clay	461	1999	0.45	1337	5797	4.3
CC58	161.0	-66.5	IIc	interbedded silty sand, sandy clay	740	2271	0.45	2145	6585	3.1
CC59	159.5	-65.0	IIc	interbedded silty sand, sandy clay	740	1758	0.45	2145	5099	2.4
CC60	171.0	-76.5	IIc	clayey sand, sandy clay	510	2358	0.45	1479	6839	4.6
CC61	169.5	-75.0	IIc	clayey sand, sandy clay	625	2166	0.45	1813	6283	3.5
CC62	181.0	-86.5	IIc	sandy elastic silt, trace clay	770	2358	0.45	2234	6839	3.1
CC63	179.5	-85.0	IIc	sandy elastic silt, trace clay	693	2278	0.45	2010	6607	3.3
CC64	191.0	-96.5	IIc	sandy elastic silt	907	2580	0.45	2631	7482	2.8
CC65	189.5	-95.0	IIc	sandy elastic silt	693	1999	0.45	2010	5797	2.9
CC66	201.0	-106.5	IIc	sandy elastic silt, clay	693	1999	0.45	2010	5797	2.9
CC67	199.5	-105.0	IIc	sandy elastic silt, clay	731	2166	0.45	2119	6283	3.0
CC68	210.9	-116.4	IIc	interbedded clayey sand, silty sand	731	1851	0.45	2119	5369	2.5
CC69	209.4	-114.9	IIc	interbedded clayey sand, silty sand	770	1999	0.45	2234	5797	2.6
CC70	221.0	-126.5	IIc	clayey sand to sandy clay	417	3147	0.45	1208	9125	7.6

Table 2.5-38—{Pressuremeter Test Results, PM-301}

(Page 2 of 2)

Test	Depth [ft]	El. [ft]	Layer/Material		G [ksf]	$G_{u/r}$ [ksf]	ν	E [ksf]	$E_{u/r}$ [ksf]	$E_{u/r}$ / E
CC71	219.5	-125.0	IIc	clayey sand to sandy clay	376	2166	0.45	1091	6283	5.8
CC72	231.0	-136.5	IIc	clayey sand to sandy clay	357	1851	0.45	1036	5369	5.2
CC73	229.5	-135.0	IIc	clayey sand to sandy clay	357	1999	0.45	1036	5797	5.6
CC74	241.0	-146.5	IIc	clayey sand	461	1851	0.45	1337	5369	4.0
CC75	239.5	-145.0	IIc	clayey sand	417	1720	0.45	1208	4989	4.1
CC76	251.0	-156.5	IIc	clay to sandy clay	510	1851	0.45	1479	5369	3.6
CC77	249.5	-155.0	IIc	clay to sandy clay	693	1999	0.45	2010	5797	2.9
CC78	261.0	-166.5	IIc	interbedded clay and sandy silt	396	2166	0.45	1148	6283	5.5
CC79	259.5	-165.0	IIc	interbedded clay and sandy silt	396	1720	0.45	1148	4989	4.3
CC80	271.0	-176.5	IIc	interbedded clay and sandy silt	417	1603	0.45	1208	4650	3.8
CC81	269.5	-175.0	IIc	interbedded clay and sandy silt	693	2358	0.45	2010	6839	3.4
CC82	281.0	-186.5	IIc	elastic silt, trace sand	510	1720	0.45	1479	4989	3.4
CC83	279.5	-185.0	IIc	elastic silt, trace sand	625	1851	0.45	1813	5369	3.0
CC84	291.0	-196.5	IIc	interbedded elastic silt and clay	461	1720	0.45	1337	4989	3.7
CC85	289.5	-195.0	IIc	interbedded elastic silt and clay	536	1498	0.45	1556	4345	2.8
CC86	301.0	-206.5	IIc	interbedded elastic silt and clay	594	2580	0.45	1722	7482	4.3
CC87	299.5	-205.0	IIc	interbedded elastic silt and clay	461	2358	0.45	1337	6839	5.1
CC88	310.7	-216.2	III	cemented sand, behaved like rock	Unsuccessful Test					
CC89	321.0	-226.5	III	interbedded clayey sand, clay	1096	3870	0.30	2850	10062	3.5
CC90	319.5	-225.0	III	interbedded clayey sand, clay	1220	4720	0.30	3171	12272	3.9
CC91	328.5	-234.0	III	cemented sand, behaved like rock	Unsuccessful Test					
CC92	338.5	-244.0	III	clayey sand	1156	3537	0.30	3005	9197	3.1
CC93	350.0	-255.5	III	clayey sand	807	3568	0.30	2098	9278	4.4
CC94	348.5	-254.0	III	clayey sand	768	3969	0.30	1996	10320	5.2
CC95	361.0	-266.5	III	clayey sand	990	3232	0.30	2573	8404	3.3
CC96	359.5	-265.0	III	clayey sand	695	3568	0.30	1808	9278	5.1

Notes:
- *G – Shear Modulus; $G_{u/r}$ – Unload/Reload Shear Modulus*
- *ν – Poisson Ratio*
- *E – Elastic Modulus; $E_{u/r}$ – Unload/Reload Elastic Modulus*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-39—{Pressuremeter Test Results, PM-701}

Test	Depth [ft]	El. [ft]	Layer/Material		G [ksf]	$G_{u/r}$ [ksf]	ν	E [ksf]	$E_{u/r}$ [ksf]	$E_{u/r}$ / E
CC03	23.5	-14.8	IIc	interbedded silts, sand, some gravel	578	3960	0.45	1676	11484	6.9
CC04	30.7	-22.0	IIc	silty sand, trace clay, shell fragments	494	3960	0.45	1432	11484	8.0
CC05	29.2	-20.5	IIc	silty sand, trace clay, shell fragments	382	2360	0.45	1109	6844	6.2
CC06	40.9	-32.2	IIc	sandy clay, silt	382	1625	0.45	1109	4712	4.2
CC07	39.3	-30.6	IIc	sandy clay, silt	610	2638	0.45	1768	7649	4.3
CC08	51.0	-42.3	IIc	silty sand, shell fragments	346	1935	0.45	1002	5611	5.6
CC09	49.5	-40.8	IIc	silty sand, shell fragments	346	3406	0.45	1002	9877	9.9
CC10	60.5	-51.8	IIc	elastic silt, clay + sand	762	2129	0.45	2211	6175	2.8
CC11	59.0	-50.3	IIc	elastic silt, clay + sand	913	3406	0.45	2647	9877	3.7
CC12	70.5	-61.8	IIc	silty sand, trace clay, shell fragments	329	1832	0.45	953	5313	5.6
CC13	69.0	-60.3	IIc	silty sand, trace clay, shell fragments	364	2360	0.45	1054	6844	6.5
CC14	80.7	-72.0	IIc	sandy silt, some clay	402	1625	0.45	1167	4712	4.0
CC15	79.2	-70.5	IIc	sandy silt, some clay	762	1769	0.45	2211	5129	2.3
CC16	90.6	-81.9	IIc	elastic silt, clay + sand	382	1290	0.45	1109	3742	3.4
CC17	89.1	-80.4	IIc	elastic silt, clay + sand	808	1625	0.45	2344	4712	2.0
CC18	100.5	-91.8	IIc	silty sand, trace clay, shell fragments	282	1935	0.45	818	5611	6.9
CC19	99.0	-90.3	IIc	silty sand, trace clay, shell fragments	913	2638	0.45	2647	7649	2.9
CC20	110.7	-102.0	IIc	sandy elastic silt, clay	644	1935	0.45	1867	5611	3.0
CC21	109.2	-100.5	IIc	sandy elastic silt, clay	469	1625	0.45	1359	4712	3.5
CC22	120.1	-111.4	IIc	silty sand, some clay	610	1499	0.45	1768	4348	2.5
CC23	118.6	-109.9	IIc	silty sand, some clay	382	1769	0.45	1109	5129	4.6
CC24	130.8	-122.1	IIc	silty sand	297	2129	0.45	861	6175	7.2
CC25	129.3	-120.6	IIc	silty sand	329	2129	0.45	953	6175	6.5
CC26	140.8	-132.1	IIc	silty sand to sandy silt, some clay	297	1499	0.45	861	4348	5.0
CC27	139.3	-130.6	IIc	silty sand to sandy silt, some clay	578	1935	0.45	1676	5611	3.3
CC28	150.9	-142.2	IIc	sandy elastic silt	494	2048	0.45	1432	5939	4.1
CC29	149.4	-140.7	IIc	sandy elastic silt	423	2129	0.45	1227	6175	5.0

Notes:
- *G – Shear Modulus; $G_{u/r}$ – Unload/Reload Shear Modulus*
- *ν – Poisson Ratio*
- *E – Elastic Modulus; $E_{u/r}$ – Unload/Reload Elastic Modulus*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-40—{Summary of Laboratory Tests and Quantities}

Test		Standard/Method	Number of Tests (1)		
			PB	IA	BF
Index	Unified Soil Classification System (USCS)	ASTM D2488	591	10	3
Index	Natural moisture content	ASTM D2216	1048	10	18
Index	Grain size analysis (sieve)	ASTM D422	546	10	9
Index	Grain size analysis (hydrometer)	ASTM D6913	546	10	6
Index	Atterberg limits	ASTM D4318	423	10	3
Index	Organic content	ASTM D2974	79	10	3
Index	Specific gravity	ASTM D854	126	10	3
Index	Unit Weight	Not Specified	126	10	-
Chemical	pH	ASTM D4972	116	-	3
Chemical	Chloride	EPA 300.0	116	-	3
Chemical	Sulfate	EPA 300.0	116	-	3
Chemical	Resistivity	ASTM G187	-	-	14
Static Performance/Strength	Consolidation	ASTM D2435	79	-	3
Static Performance/Strength	Permeability (2)	AST 2434	-	-	3
Static Performance/Strength	Unconfined compression (UC)	ASTM D2166	25	-	-
Static Performance/Strength	Unconsolidated-Undrained Triaxial (UU)	ASTM D2850	110	-	-
Static Performance/Strength	Consolidated-Undrained Triaxial (CU-)	ASTM D4767	10	-	3
Static Performance/Strength	Consolidated-Drained Triaxial (CD)	Unspecified	-	-	3
Static Performance/Strength	Direct Shear (DS)	ASTM D3080	43	-	-
Static Performance/Strength	Modified Proctor (Moisture-Density)	ASTM D1557)	-	-	4
Static Performance/Strength	California Bearing Ratio	ASTM D1883	12	-	2
Dynamic Resonant Column Torsional Shear		Not Specified	13	10	8

Notes:
- (1) PB: Powerbolck Area (Includes Construction Laydown, Cooling and Transmission Corridor)
 IA: Intake Area
 BF: Backfill
- (2) Description of slug tests and the results are provided in Section 2.4.12

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-41—{Index Properties, Powerblock Area}

POWERBLOCK AREA		USCS	Stat	γ_{moist} [%]	w [%]	LL [%]	PL [%]	PI [%]	Fines [%]
Stratum I - Terrace Sand		SM, SP-SM	Min	120.0	4.5	NV	NP	NP	4.6
			Max	124.0	36.2	55.0	20.0	37.0	72.0
			Avg	121.3	15.8	19.7	8.0	11.7	21.8
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min	103.0	15.1	27.0	11.0	8.0	50.0
			Max	122.3	42.5	79.0	36.0	54.0	99.7
			Avg	115.4	31.2	57.4	20.7	36.6	79.5
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min	117.0	13.5	NV	NP	NP	2.1
			Max	128.4	36.2	72.0	32.0	50.0	72.7
			Avg	122.2	24.1	24.8	12.0	12.8	26.2
	Layer 2	SM SP-SM	Min	120.5	25.0	NV	NP	NP	10.6
			Max	126.0	44.2	72.0	41.0	40.0	87.0
			Avg	122.5	30.5	19.7	10.6	9.1	23.3
	Layer 3	SM	Min	123.0	16.1	NV	NP	NP	9.8
			Max	123.0	38.7	49.0	28.0	28.0	35.9
			Avg	123.0	26.0	17.3	9.5	7.8	23.7
Stratum IIc - Chesapeake Clay/Silt		MH SM	Min	86.5	27.5	39.0	20.0	9.0	19.6
			Max	117.0	109.8	199.0	119.0	133.0	99.5
			Avg	103.9	51.2	95.4	42.9	52.5	59.9
Stratum III - Nanjemoy Sand		SC SM	Min	123.5	13.4	36.0	14.0	18.0	13.9
			Max	132.0	44.5	79.0	36.0	59.0	44.6
			Avg	127.0	29.1	57.1	22.6	34.5	23.3
Backfill		GP GM	Min	136.8	7.1	NV	NP	NP	7.2
			Max	150.4	5.6	NV	NP	NP	11.4
			Avg	146.2	6.3	NV	NP	NP	9.3

Notes:
- *γ_{moist}: Moist Unit Weight*
- *w: Water Content*
- *LL: Liquid Limit*
- *PL: Plastic Limit*
- *NP: Non Plastic*
- *NV: Non Viscous*
- *NA: Not Available*

Table 2.5-42—{Index Properties, Intake Area}

INTAKE AREA		USCS	Stat	γ_{moist} [%]	w [%]	LL [%]	PL [%]	PI [%]	Fines [%]
Stratum I - Terrace Sand		SM, SP-SM	Min	NA	NA	NA	NA	NA	NA
			Max	NA	NA	NA	NA	NA	NA
			Avg	NA	NA	NA	NA	NA	NA
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min	NA	NA	NA	NA	NA	NA
			Max	NA	NA	NA	NA	NA	NA
			Avg	NA	NA	NA	NA	NA	NA
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min	NA	7.9	NA	NA	NA	16.5
			Max	NA	7.9	NA	NA	NA	16.5
			Avg	NA	7.9	NA	NA	NA	16.5
	Layer 2	SM SP-SM	Min	NA	9.4	NV	NP	NP	6.3
			Max	NA	36.0	27.0	17.0	10.0	44.2
			Avg	NA	24.4	5.4	3.4	2.0	18.9
	Layer 3	SM	Min	118.2	15.4	NV	NP	NP	8.4
			Max	123.4	37.4	42.0	23.0	22.0	37.9
			Avg	120.4	25.5	13.3	9.2	4.1	25.9
Stratum IIc - Chesapeake Clay/Silt		SM MH	Min	93.6	22.4	NV	NP	NP	11.0
			Max	118.4	94.5	143.0	79.0	110.0	98.3
			Avg	108.2	48.5	72.5	32.6	39.9	49.0
Stratum III - Nanjemoy Sand		SC SM	Min	Not Encountered					
			Max						
			Avg						
Backfill		GP GM	Min	136.8	7.1	NV	NP	NP	7.2
			Max	150.4	5.6	NV	NP	NP	11.4
			Avg	146.2	6.3	NV	NP	NP	9.3

Notes:
- *γ_{moist}: Moist Unit Weight*
- *w: Water Content*
- *LL: Liquid Limit*
- *PL: Plastic Limit*
- *NP: Non Plastic*
- *NV: Non Viscous*
- *NA: Not Available*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-43—{Summary of Soils Chemical Testing Data}

CCNPP Unit 3		USCS	Stat	pH [$CaCl_2$]	pH [H_2O]	Sulfate [(1)]	Chloride [(2)]
Stratum I - Terrace Sand		SM, SP-SM	Min	2.6	2.7	0.0	<10
			Max	6.7	7.6	2.6	48.6
			Avg	4.6	5.5	0.2	<12
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min	2.6	2.5	0.0	<10
			Max	4.9	5.8	2.6	10.7
			Avg	3.1	3.6	0.7	<10
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min	2.4	2.5	0.0	<10
			Max	7.4	8.0	3.1	145.0
			Avg	5.7	5.8	0.6	<22
	Layer 2	SM SP-SM	Min	2.4	2.5	0.0	<10
			Max	7.4	8.0	3.1	145.0
			Avg	5.7	5.8	0.6	<22
	Layer 3	SM	Min	2.4	2.5	0.0	<10
			Max	7.4	8.0	3.1	145.0
			Avg	5.7	5.8	0.6	<22
Stratum IIc - Chesapeake Clay/Silt		MH SM	Min	6.6	7.0	0.2	<10
			Max	6.6	7.0	0.2	<10
			Avg	6.6	7.0	0.2	<10
Stratum III - Nanjemoy Sand		SC SM	Min	NA	NA	NA	NA
			Max	NA	NA	NA	NA
			Avg	NA	NA	NA	NA
Backfill		GP GM	Min	8.3	-	204.0	<2.1
			Max	8.5	-	446.0	2.2
			Avg	8.4	-	325.0	2.1

Notes:
(1) Expressed as [%] for in-situ soils and as [mg/Kg] for backfill
(2) Expressed as [ppm] for in-situ soils and as [mg/Kg] for backfill
- NA: Not Available

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-44—{Consolidation Test Results, Powerblock Area}

POWERBLOCK AREA		USCS	Stat	C_r	C_c	e_o	p'_c [ksf]	OCR
Stratum I - Terrace Sand		SM, SP-SM	Min	0.009	0.37	0.85	11.40	4.26
			Max	0.009	0.37	0.85	11.40	4.26
			Avg	0.009	0.37	0.85	11.40	4.26
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min	0.013	0.46	0.82	11.20	4.91
			Max	0.043	0.68	1.15	35.00	15.40
			Avg	0.026	0.54	1.03	21.66	8.10
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min	0.006	0.04	0.63	20.20	2.82
			Max	0.012	0.32	0.92	30.00	22.61
			Avg	0.010	0.19	0.80	24.40	9.99
	Layer 2	SM SP-SM	Min	0.003	0.11	0.71	4.20	1.00
			Max	0.003	0.11	0.90	23.80	4.68
			Avg	0.003	0.11	0.80	14.00	2.84
	Layer 3	SM	Min	NA	NA	NA	NA	NA
			Max	NA	NA	NA	NA	NA
			Avg	NA	NA	NA	NA	NA
Stratum IIc - Chesapeake Clay/Silt(1)		MH SM	Min	0.007	0.35	1.01	21.40	2.14
			Max	0.169	1.73	2.41	42.30	5.66
			Avg	0.060	0.95	1.61	33.30	3.21
Stratum III - Nanjemoy Sand		SC SM	Min	0.021	0.26	0.73	29.20	1.76
			Max	0.092	0.91	1.42	32.80	1.90
			Avg	0.045	0.53	1.00	30.40	1.85
Backfill		GP GM	Min	Large preconsolidation pressure of 54 ksf reported in one instance. It was not possible to define the virgin compression slope and the preconsolidation pressure				
			Max					
			Avg					

Notes:
- *C_r: Recompression index*
- *C_c: Compression index*
- *e_o: Initial void ratio*
- *p'_c: Preconsolidation pressure*

- (1) Properties given for clay portions of layer

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-45—{Consolidation Test Results, Intake Area}

INTAKE AREA		USCS	Stat	C_r	C_c	e_o	p'_c [ksf]	OCR
Stratum I - Terrace Sand		SM, SP-SM	Min	NA				
			Max					
			Avg					
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min					
			Max					
			Avg					
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min					
			Max					
			Avg					
	Layer 2	SM SP-SM	Min					
			Max					
			Avg					
	Layer 3	SM	Min	0.006	0.135	0.635	32.5	17.7
			Max	0.006	0.135	0.635	32.5	17.7
			Avg	0.006	0.135	0.635	32.5	17.7
Stratum IIc - Chesapeake Clay/Silt		MH SM	Min	0.020	0.371	0.97	25.7	3.7
			Max	0.155	1.641	1.95	40.8	9.2
			Avg	0.085	1.036	1.47	32.4	7.1
Stratum III - Nanjemoy Sand		SC SM	Min	Not Encountered				
			Max					
			Avg					
Backfill		GP GM	Min	Large preconsolidation pressure of 54 ksf reported in one instance. It was not possible to define the virgin compression slope and the preconsolidation pressure				
			Max					
			Avg					

Notes:
- *C_r: Recompression index*
- *C_c: Compression index*
- *e_o: Initial void ratio*
- *p'_c: Preconsolidation pressure*
- *NA: Not Available*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-46—{Shear Strength Laboratory Testing Data, Powerblock Area}

POWERBLOCK AREA		USCS	Stat	Triaxial Test				Direct Shear		s_u [ksf]	
				c' [ksf]	ϕ' [°]	c [ksf]	ϕ [°]	c' [ksf]	ϕ' [°]	UC	UU
Stratum I - Terrace Sand		SM, SP-SM	Min	0.55	27.9	1.16	13.3	0.42	24.9	1.72	1.20
			Max	0.55	27.9	1.16	13.3	0.89	26.0	1.72	1.46
			Avg	0.55	27.9	1.16	13.3	0.66	25.5	1.72	1.33
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min	0.44	31.0	0.72	12.5	0.64	19.0	1.14	1.42
			Max	0.98	32.1	2.06	17.0	1.38	30.1	4.06	4.60
			Avg	0.71	31.6	1.39	14.8	1.01	22.9	2.50	2.38
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min	0.30	33.5	0.59	19.5	NA	NA	NA	0.80
			Max	0.30	33.5	0.59	19.5	NA	NA	NA	2.44
			Avg	0.30	33.5	0.59	19.5	NA	NA	NA	5.76
	Layer 2	SM SP-SM	Min	0.04	30.0	1.94	13.4	NA	NA	NA	0.90
			Max	1.00	34.6	3.36	20.0	NA	NA	NA	0.90
			Avg	0.52	32.3	2.65	16.7	NA	NA	NA	0.90
	Layer 3	SM	Min	NA	NA	NA	NA	NA	NA	NA	NA
			Max	NA	NA	NA	NA	NA	NA	NA	NA
			Avg	NA	NA	NA	NA	NA	NA	NA	NA
Stratum IIc - Chesapeake Clay/Silt		MH SM	Min	NA	NA	NA	NA	0.00	29.0	3.74	1.80
			Max	NA	NA	NA	NA	1.58	35.0	5.24	9.58
			Avg	NA	NA	NA	NA	0.79	32.0	4.49	6.37
Stratum III - Nanjemoy Sand		SC SM	Min	NA	NA	NA	NA	NA	NA	NA	4.56
			Max	NA	NA	NA	NA	NA	NA	NA	7.66
			Avg	NA	NA	NA	NA	NA	NA	NA	5.78
Backfill		GP GM	Min	0.00	42.5	-	-	NA	NA	-	-
			Max	0.00	43.5	-	-	NA	NA	-	-
			Avg	0.00	43.0	-	-	NA	NA	-	-

Notes:
- NA: Not Available
- UC: Unconfined compression
- UU: Unconsolidated undrained triaxial test

Table 2.5-47—{Shear Strength Laboratory Testing Data, Intake Area}

INTAKE AREA		USCS	Stat	Triaxial Test				Direct Shear		s_u [ksf]	
				c' [ksf]	φ' [°]	c [ksf]	φ [°]	c' [ksf]	φ' [°]	UC	UU
Stratum I - Terrace Sand		SM, SP-SM	Min	NA							
			Max								
			Avg								
Stratum IIa - Chesapeake Clay/Silt		CH MH	Min								
			Max								
			Avg								
Stratum IIb - Chesapeake Cemented Sand	Layer 1	SM SP	Min								
			Max								
			Avg								
	Layer 2	SM SP-SM	Min								
			Max								
			Avg								
	Layer 3	SM	Min	0.00	38.0	NA	NA	0.46	28.2	NA	NA
			Max	0.00	38.0	NA	NA	0.46	28.2	NA	NA
			Avg	0.00	38.0	NA	NA	0.46	28.2	NA	NA
Stratum IIc - Chesapeake Clay/Silt		MH SM	Min	0.00	19.4	2.69	0.0	0.00	24.4	NA	1.92
			Max	3.63	37.3	7.68	18.7	2.10	38.7	NA	8.32
			Avg	1.52	31.9	4.35	11.9	0.73	30.8	NA	4.83
Stratum III - Nanjemoy Sand		SC SM	Min	Not Encountered							
			Max								
			Avg								
Backfill		GP GM	Min	0.00	42.5	-	-	NA	NA	-	-
			Max	0.00	43.5	-	-	NA	NA	-	-
			Avg	0.00	43.0	-	-	NA	NA	-	-

Notes:
- NA: Not Available
- UC: Unconfined compression
- UU: Unconsolidated undrained triaxial test

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-48—{Modified Proctor Tests on Backfill Samples}

Sample	Modified Proctor						98% MP		90% MP	
	Uncorrected			Corrected						
	w [%]	γ_{dry} [pcf]	γ_{moist} [pcf]	w [%]	γ_{dry} [pcf]	γ_{moist} [pcf]	γ_{dry} [pcf]	γ_{moist} [pcf]	γ_{dry} [pcf]	γ_{moist} [pcf]
CR6 Composite FUGRO	6.9	145.2	155.2	6.0	148.0	156.9	145.0	153.7	133.2	141.2
CR6 Composite MACTEC	6.4	144.0	153.2	6.0	145.3	154.0	142.4	150.9	130.8	138.6
GAB Composite MACTEC	6.4	145.9	155.2	5.7	148.6	157.1	145.6	153.9	133.7	141.4
GAB Composite FUGRO	7.1	145.3	155.6	6.5	148.5	158.2	145.5	155.0	133.7	142.3
Min	6.4	144.0	153.2	5.7	145.3	154.0	142.4	150.9	130.8	138.6
Max	7.1	145.9	155.6	6.5	148.6	158.2	145.6	155.0	133.7	142.3
Avg	6.7	145.1	154.8	6.1	147.6	156.5	144.6	153.4	132.8	140.9

Notes:
- USCS: GP-GM

Table 2.5-49—{RCTS Testing Samples}

	Sample	Depth [ft]	USCS	Type	γ [pcf]	w [%]
POWERBLOCK AREA	B-437-6	13.5	SP-SM	UD	124.1	7.2
	B-301-10	33.5	CH	UD	117.5	31.1
	B-305-17	39.5	SC	UD	117.2	34.7
	B-404-14	52.0	SP-SM	UD	117.6	27.7
	B-401-31	138.5	CH	UD	104.1	44.1
	B-401-67	348.5	SM	UD	116.4	35.6
	B-401-48	228.5	MH	UD	98.2	58.6
	B-301-78	383.5	SM	Jar	116.4	34.4
	B-306-17	68.0	CH	UD	115.8	30.7
	B-409-15	35.0	SP-SM	UD	124.8	23.3
	B-404-22	83.5	SM	UD	115.4	32.2
	B-401-42	198.5	SM	UD	101.2	48.8
	B-409-39	95.0	SM	UD	109.3	33.1
INTAKE AREA	B-773-2	15.9	SM	UD	125.7	23.3
	B-773-3	27.0	SC	UD	111.6	35.0
	B-773-4	37.0	CH	UD	103.0	53.6
	B-773-5	47.0	SC	UD	110.9	34.1
	B-773-6	57.0	CH	UD	106.4	44.5
	B-773-7	66.1	CH	UD	110.1	33.5
	B-773-9	87.0	CH	UD	99.1	59.2
	B-773-11	107.0	CH	UD	102.5	55.1
	B-773-13	127.0	SC	UD	108.3	45.2
	B-773-15	147.0	CH	UD	101.5	52.3
BACKFILL	CR6 Composite(1)	-	GP-GM	Bulk	145.4	6.4
	GAB Composite(1)	-	GP-GM	Bulk	147.3	5.8
	CR6 Vulcan Average(1)	-	GP-GM	Bulk	143.1	5.5

Notes:
(1) Test results reported for target unit weight of 95% Modified Proctor

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-50—{Low Strain Results for Backfill Samples}

Source	γ [pcf]	Moisture Content [%]	Confining Pressure [ksf]	G_{max} [ksf]	V_s [fps]	D [%]
CR-6 Composite	145.4	6.4	1.08	2680	770	4.61
			2.16	3851	922	4.1
			4.32	5846	1133	3.41
CR-6 Vulcan Avg	143.1	5.5	1.08	3741	917	2.31
			2.16	5196	1080	1.96
			4.32	7054	1257	1.88
GAB Composite	147.3	5.8	1.08	3904	923	3.91
			2.16	5444	1089	3.33
			4.32	7427	1270	2.99

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-51—{USCS Classification and Index Properties}

STRATUM			USCS	γ_{moist} [%]	w [%]	LL [%]	PL [%]	PI [%]	Fines [%]
POWERBLOCK AREA	I - Terrace Sand		SM, SP-SM	120.0	16.0	20.0	8.0	12.0	21.8
	IIa - Chesapeake Clay/Silt		CH MH	115.0	31.0	57.0	21.0	36.0	79.5
	IIb - Chesapeake Cemented Sand	L1	SM SP	120.0	24.0	26.0	13.0	13.0	26.2
		L2	SM SP-SM	120.0	31.0	20.0	11.0	9.0	23.3
		L3	SM	120.0	26.0	17.0	9.0	8.0	23.7
	IIc - Chesapeake Clay/Silt		MH SM	105.0	51.0	95.0	42.0	53.0	59.9
	III - Nanjemoy Sand		SC SM	125.0	29.0	57.0	22.0	35.0	23.3
INTAKE AREA	I - Terrace Sand		SM, SP-SM	NA	NA	NA	NA	NA	NA
	IIa - Chesapeake Clay/Silt		CH MH	NA	NA	NA	NA	NA	NA
	IIb - Chesapeake Cemented Sand	L1	SM SP	NA	8.0	NA	NA	NA	16.5
		L2	SM SP-SM	NA	24.0	5.0	3.0	2.0	18.9
		L3	SM	120.0	26.0	13.0	9.0	4.0	25.9
	IIc - Chesapeake Clay/Silt		SM MH	110.0	49.0	73.0	33.0	40.0	49.0
	III - Nanjemoy Sand		SC SM	125.0	29.0	57.0	22.0	35.0	23.3
BACKFILL			GP GM	145.0	6.0	NV	NP	NP	9.0

Notes:
- *NP: Non Plastic*
- *NV: Non Viscous*
- *NA: Not Available*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-52—{Guidelines for Soil Chemistry Evaluation}

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

Soil Aggressiveness Recommendations for Normal Weight Concrete Subject to Sulfate Attack			
Concrete Exposure	Water Soluble Sulfate (SO_4) in Soil, Percent	Cement Type	Max W/C Ratio
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:
- (A) API, 2007
- (B) FHWA, 1990
- (C) ACI, 1994
- (1) Or a blend of Type II cement and a ground granulated blast furnace slag or a pozzolan that gives equivalent sulfate resistance

Table 2.5-53—{Performance Properties under Static Loading}

Area	STRATUM		C_r	C_c	e_o	p'_c [ksf]	OCR	c_v [ft^2/year]	k_h [ft/s]	k_v [ft/s]
POWERBLOCK AREA	I - Terrace Sand		0.009	0.37	0.85	11.40	4.26	NA	NA	NA
	IIa - Chesapeake Clay/Silt		0.026	0.54	1.03	21.66	8.10	316.0	1.62E-09	1.62E-09
	IIb - Chesapeake Cemented Sand	L1	0.010	0.19	0.80	24.40	9.99	2018.0	9.84E-06	9.84E-07
		L2	0.003	0.11	0.80	14.00	2.84	2018.0	9.84E-06	9.84E-07
		L3	0.010	0.19	0.80	24.40	9.99	2018.0	9.84E-06	9.84E-07
	IIc - Chesapeake Clay/Silt		0.06	0.95	1.61	33.30	3.21	1913.0	1.62E-09	1.62E-09
	III - Nanjemoy Sand		0.05	0.53	1.00	30.40	1.85	2018.0	9.84E-07	9.84E-08
INTAKE AREA	I - Terrace Sand/ Fill		NP	NP	NP	NP	NP	NP	NP	NP
	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP	NP	NP	NP	NP
	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP	NP	NP	NP	NP
		L2	NA	NA	NA	NA	NA	NA	NA	NA
		L3	0.01	0.14	0.64	32.50	17.69	2018.0	9.84E-06	9.84E-07
	IIc - Chesapeake Clay/Silt		0.09	1.04	1.47	32.44	7.11	1913.0	1.62E-09	1.62E-09
	III - Nanjemoy Sand		0.05	0.53	1.00	30.40	1.85	2018.0	9.84E-07	9.84E-08
BACKFILL			Consolidation in backfill material will not be significant						9.50E-03	9.50E-04

Notes:
- *NP: Not Present*
- *NA: Not Available*
- *c_v Values correspond to an applied pressure of 8 ksf for IIA, 32 ksf for IIb, and 64 ksf for IIc*
- *k_h is horizontal hydraulic conductivity; k_v is vertical hydraulic conductivity*
- *Intake area values for deeper strata are obtained from Powerblock recommendation*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-54—{Strength Properties of Soils}

STRATUM			c' [ksf]	ϕ' [°]	c [ksf]	ϕ [°]	S_u [ksf]
POWERBLOCK AREA	I - Terrace Sand		0.0	27.9	1.2	13.3	1.5
	IIa - Chesapeake Clay/Silt		0.7	31.6	1.4	14.8	2.4
	IIb - Chesapeake Cemented Sand	L1	0.0	33.5	1.2	19.5	5.8
		L2	0.0	32.3	2.7	16.7	0.9
		L3	0.0	31.7	1.2	19.5	5.8
	IIc - Chesapeake Clay/Silt		0.8	32.0	1.4	14.8	5.4
	III - Nanjemoy Sand(1)		0.0	40.0	1.2	19.5	5.8
INTAKE AREA	I - Terrace Sand		NP	NP	NP	NP	NP
	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP	NP
	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP	NP
		L2	NP	NP	NP	NP	NP
		L3	0.0	33.1	NA	NA	5.8
	IIc - Chesapeake Clay/Silt		1.5	31.1	4.3	11.9	4.8
	III - Nanjemoy Sand(1)		0.0	40.0	1.2	19.5	2.9
BACKFILL			0.0	40.0	-	-	-

Notes:
(1) Friction of 40 degrees assumed, recommendation at Intake taken from Powerblock
NA: Not Available
NP: Not Present

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-55—{Estimation of Elastic Modulus}

	STRATUM		E [ksf] from various methods						
			V_s (1)	PM	SPT		s_u		Avg
					(2)	(3)	(4)	(5)	
POWERBLOCK AREA	I - Terrace Sand		729	241	504	268	-	-	436
POWERBLOCK AREA	IIa - Chesapeake Clay/Silt		1210	652	-	-	1098	1415	1094
POWERBLOCK AREA	IIb - Chesapeake Cemented Sand	L1	4090	1575	3204	1226	-	-	2525
POWERBLOCK AREA	IIb - Chesapeake Cemented Sand	L2	1300	1573	864	375	-	-	1028
POWERBLOCK AREA	IIb - Chesapeake Cemented Sand	L3	5120	2200	2268	914	-	-	2625
POWERBLOCK AREA	IIc - Chesapeake Clay/Silt		1560	1555	-	-	2772	3573	2365
POWERBLOCK AREA	III - Nanjemoy Sand		4300	2500	2700	-	-	-	3166
INTAKE AREA	I - Terrace Sand		NP	NP	NP	NP	NP	NP	NP
INTAKE AREA	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP	NP	NP	NP
INTAKE AREA	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP	NP	NP	NP
INTAKE AREA	IIb - Chesapeake Cemented Sand	L2	941	-	612	308	-	-	620
INTAKE AREA	IIb - Chesapeake Cemented Sand	L3	1840	-	1944	752	-	-	1512
INTAKE AREA	IIc - Chesapeake Clay/Silt		1290	-	-	-	2169	1928	1796
INTAKE AREA	III - Nanjemoy Sand(6)		4300	2500	2700	-	-	-	3166
BACKFILL			1920		-	-	-	-	1920

Notes:
- *(1) Calculated from $G_{dyn}/G_{static} = 10$;*
- *(2) $E = 18N_{60}$ (Davie, 1988); [tsf]*
- *(3) $E = \beta_0$ sqrt(OCR) $+\beta_1 N_{60}$; [psf]*
- *(4) $E = 450\,s_u$ (Davie, 1988); [tsf]*
- *(5) $E = 2G(1+\nu)$; $G = 200\,s_u$ (Senapathy, 2001); [tsf]*
- *(6) Values adopted from Powerblock Area*
- *NP: Not Present*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-56—{Basis for Recommendation of $E_{u/r}/E$ Ratio}

Stratum			$E_{u/r}/E$			
			Min	Max	Avg	Rec
POWERBLOCK AREA	I - Terrace Sand		1.6	9.8	6.9	3.0
	IIa - Chesapeake Clay/Silt		2.5	2.8	2.7	3.0
	IIb - Chesapeake Cemented Sand	L1	2.2	10.4	5.6	3.0
		L2	4.5	13.4	8.8	4.5
		L3	3.9	6.9	5.4	3.9
	IIc - Chesapeake Clay/Silt		2.4	7.6	4.0	3.0
	III - Nanjemoy Sand		3.1	5.2	4.1	3.1
INTAKE AREA	I - Terrace Sand		NP	NP	NP	NP
	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP
	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP
		L2	NA	NA	NA	4.5
		L3	-	-	-	3.0
	IIc - Chesapeake Clay/Silt		2.0	9.9	4.8	3.0
	III - Nanjemoy Sand		-	-	-	3.0
BACKFILL			-	-	-	-

Notes:
- Valuesfrom pressuremeter tests at B-301 (Powerblock Area) and B-701 (Intake Area)
- NP: Not Present
- NA: Not Available

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-57—{Elastic Properties Under Static Conditions}

Area	Stratum		E [ksf]	ν[1]	G [ksf]	$E_{u/r}/E$
POWERBLOCK AREA	I - Terrace Sand		436	0.30	168	3.0
	IIa - Chesapeake Clay/Silt		1094	0.45	377	3.0
	IIb - Chesapeake Cemented Sand	L1	2525	0.30	971	3.0
		L2	1028	0.30	395	4.5
		L3	2625	0.30	1010	3.9
	IIc - Chesapeake Clay/Silt		2365	0.45	815	3.0
	III - Nanjemoy Sand		3166	0.30	1218	3.1
INTAKE AREA	I - Terrace Sand		NP	NP	NP	NP
	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP
	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP
		L2	620	0.30	239	4.5
		L3	1512	0.30	581	3.0
	IIc - Chesapeake Clay/Silt		1796	0.45	619	3.0
	III - Nanjemoy Sand[2]		3166	0.30	1218	3.0
BACKFILL			1920	0.35	711	not used

Notes:

- *(1) Adopted from typical values reported in the literature (Salgado, 2008).*
- *(2) Adopted from Powerblock Area*
- *NP: Not Present*

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-58—{Earth Pressure Coefficients}

Area	Stratum		K_a	K_p	K_0	tan δ[1]	FOS against Sliding[2]
POWERBLOCK AREA	I - Terrace Sand		0.36	2.76	0.53	0.40	2.7
	IIa - Chesapeake Clay/Silt		0.31	3.20	0.48	0.35	2.3
	IIb - Chesapeake Cemented Sand	L1	0.29	3.46	0.45	0.45	3.0
		L2	0.30	3.30	0.47	0.45	3.0
		L3	0.31	3.21	0.47	0.45	3.0
	IIc - Chesapeake Clay/Silt		0.31	3.25	0.47	0.40	2.7
	III - Nanjemoy Sand		Not Required				
INTAKE AREA	I - Terrace Sand		NP	NP	NP	NP	
	IIa - Chesapeake Clay/Silt		NP	NP	NP	NP	
	IIb - Chesapeake Cemented Sand	L1	NP	NP	NP	NP	
		L2	NA	NA	NA	NA	
		L3	0.29	3.41	0.45	0.45	3.0
	IIc - Chesapeake Clay/Silt		0.32	3.14	0.48	0.40	2.7
	III - Nanjemoy Sand		Not Required				
BACKFILL			0.22	4.60	0.36	0.40	2.7

Notes:

- (1) tan δ is sliding resistance

- values of ϕ are used to determine K coefficients

$K_a = tan^2(45-\phi'/2); K_p = tan^2(45+\phi'/2); K_0 = 1 - sin(\phi')$

- (2) Factor of Safety is tan δ divided by SSE acceleration, 0.15 g.

The *FOS does not account for passive earth pressure on the sides of the buildings.*

Table 2.5-59—{Dynamic Properties for Powerblock Area}

POWERBLOCK AREA	El [ft msl]	D [ft]	γ [pcf]	G_o [ksf]	V_s [fps]	V_p [fps]	ν	Damping [%]	
								S(1)	P(2)
Backfill 1	85.0	0.0	145.0	2810	790	1645	0.35	1.50	0.50
Backfill 2	79.0	6.0	145.0	3650	900	1915	0.36	1.50	0.50
Backfill 3	63.0	22.0	145.0	5250	1080	2260	0.35	1.50	0.50
I, Terrace Sand	85.0	0.0	120.0	2330	790	2903	0.46	1.40	0.47
IIA, Chesapeake Clay/Silt	60.0	25.0	115.0	4320	1100	4623	0.47	1.30	0.43
IIB-1, Che. Cem. Sand	45.0	40.0	120.0	7840	1450	4800	0.45	1.30	0.43
IIB-2, Che. Cem. Sand	30.0	55.0	120.0	12070	1800	5970	0.45	1.30	0.43
IIB-3, Che. Cem. Sand	15.0	70.0	120.0	4760	1130	5762	0.48	1.30	0.43
IIB-4, Che. Cem. Sand	0.0	85.0	120.0	11280	1740	5771	0.45	1.30	0.43
IIc, Chesapeake Clay/Silt	-15.0	100.0	105.0	5100	1250	5254	0.47	1.10	0.37
III, Nanjemoy Sand (NS1)	-200.0	285.0	125.0	12440	1790	5937	0.45	1.30	0.43
III, Nanjemoy Sand (NS2)	-220.0	305.0	125.0	21070	2330	6274	0.42	1.30	0.43
III, Nanjemoy Sand (NS3)	-230.0	315.0	125.0	16000	2030	5793	0.43	1.30	0.43
III, Nanjemoy Sand (NS4)	-270.0	355.0	125.0	14460	1930	5896	0.44	1.30	0.43
I1, Deep Soil	-317.0	402.0	115.0	17290	2200	5389	0.40	1.30	0.43
I2, Deep Soil	-1000.0	1085.0	115.0	19390	2330	5707	0.40	1.30	0.43
I3, Deep Soil	-1500.0	1585.0	115.0	23220	2550	6246	0.40	1.30	0.43
I4, Deep Soil	-2000.0	2085.0	115.0	28000	2800	6859	0.40	1.30	0.43
I5, Bedrock	-2446.0	2531.0	162.0	125780	5000	9354	0.30	1.30	0.43
I6, Bedrock	-2456.0	2541.0	162.0	246520	7000	13096	0.30	1.30	0.43
I7, Bedrock	-2466.0	2551.0	162.0	425830	9200	17212	0.30	1.30	0.43
Base	-3000.0	3085.0	162.0	425830	9200	17212	0.30	1.30	0.43

Notes:
- (1) Shear damping based on RCTS test results
- (2) P damping assumed as 1/3 of S damping

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-60—{Dynamic Properties for Intake Area}

INTAKE AREA	El [ft msl]	D [ft]	γ [pcf]	G_o [ksf]	V_s [fps]	V_p [fps]	ν	Damping [%]	
								S(1)	P(2)
Backfill 1	10.0	0.0	145.0	2810	790	1645	0.35	1.50	0.50
Backfill 2	4.0	6.0	145.0	3650	900	1915	0.36	1.50	0.50
Backfill 3	-12.0	22.0	145.0	5250	1080	2260	0.35	1.50	0.50
IIB-3, C. Cemented Sand	-0.3	8.2	120.0	2270	780	1610	0.35	1.30	0.43
IIB-4, C. Cemented Sand	-2.3	10.2	120.0	6890	1360	5580	0.47	1.30	0.43
IIC-1, C. Clay/Silt	-18.7	26.6	115.0	4720	1150	5250	0.47	1.30	0.43
IIC-2, C. Clay/Silt	-43.0	50.9	105.0	4310	1150	5250	0.47	1.30	0.43
IIC-3, C. Clay/Silt	-105.0	112.9	115.0	4720	1150	5250	0.47	1.30	0.43
IIC-4, C. Clay/Silt	-131.0	138.9	105.0	4310	1150	5250	0.47	1.30	0.43
III, Nanjemoy Sand (NS1)	-200.0	207.9	125.0	12440	1790	5937	0.45	1.10	0.37
III, Nanjemoy Sand (NS2)	-220.0	227.9	125.0	21070	2330	6274	0.42	1.30	0.43
III, Nanjemoy Sand (NS3)	-230.0	237.9	125.0	16000	2030	5793	0.43	1.30	0.43
III, Nanjemoy Sand (NS4)	-270.0	277.9	125.0	14460	1930	5896	0.44	1.30	0.43
I1, Deep Soil	-317.0	324.9	115.0	17290	2200	5389	0.40	1.30	0.43
I2, Deep Soil	-1000.0	1007.9	115.0	19390	2330	5707	0.40	1.30	0.43
I3, Deep Soil	-1500.0	1507.9	115.0	23220	2550	6246	0.40	1.30	0.43
I4, Deep Soil	-2000.0	2007.9	115.0	28000	2800	6859	0.40	1.30	0.43
I5, Bedrock	-2446.0	2453.9	162.0	125780	5000	9354	0.30	1.30	0.43
I6, Bedrock	-2456.0	2463.9	162.0	246520	7000	13096	0.30	1.30	0.43
I7, Bedrock	-2466.0	2473.9	162.0	425830	9200	17212	0.30	1.30	0.43
Base	-3000.0	3007.9	162.0	425830	9200	17212	0.30	1.30	0.43

Notes:
- (1) Shear damping based on RCTS test results
- (2) P damping assumed as 1/3 of S damping

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-61—{Strain Dependant Properties for Powerblock Area}

Strata	Strain	G/G_{max}	Damping [%]
I-Terrace Sand	0.0001	1.0000	1.40
	0.0003	1.0000	1.50
	0.0010	0.9800	1.80
	0.0030	0.9150	2.30
	0.0100	0.7600	3.80
	0.0300	0.5600	6.50
	0.1000	0.3400	10.50
	0.3000	0.2000	14.80
	1.0000	0.1000	-
IIC-Chesapeake Clay/Silt	0.0001	1.0000	1.10
	0.0003	1.0000	1.10
	0.0010	1.0000	1.10
	0.0030	1.0000	1.13
	0.0100	0.9900	1.20
	0.0300	0.9400	1.50
	0.1000	0.8000	2.40
	0.3000	0.6300	4.10
	0.6000	0.5000	5.80
	1.0000	0.4000	7.40
All Other Natural Soils	0.0001	1.0000	1.30
	0.0003	1.0000	1.30
	0.0010	1.0000	1.40
	0.0030	0.9900	1.60
	0.0100	0.9400	2.20
	0.0300	0.8200	3.20
	0.1000	0.6200	5.40
	0.3000	0.4200	8.40
	0.6000	0.3100	10.60
	1.0000	0.2500	12.60

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-62—{Strain Dependant Properties for Intake Area}

Strata	Strain	G/G_{max}	Damping [%]
IIB, IIC-1 & III	0.0001	1.0000	1.30
	0.0003	1.0000	1.30
	0.0010	1.0000	1.40
	0.0030	0.9900	1.60
	0.0100	0.9400	2.20
	0.0300	0.8200	3.20
	0.0548	0.7200	4.30
	0.1000	0.6200	5.40
	0.1732	0.5200	6.90
	0.3000	0.4200	8.40
	0.4243	0.3650	9.50
	0.6000	0.3100	10.60
	1.0000	0.2500	12.60
IIC-2	0.0001	1.0000	1.10
	0.0003	1.0000	1.10
	0.0010	1.0000	1.10
	0.0030	0.9900	1.13
	0.0100	0.9400	1.70
	0.0300	0.8200	3.20
	0.0548	0.7200	4.30
	0.1000	0.6200	5.40
	0.1732	0.5200	6.90
	0.3000	0.4200	8.40
	0.4243	0.3650	9.50
	0.6000	0.3100	10.60
	1.0000	0.2500	12.60

Strata	Strain	G/G_{max}	Damping [%]
IIC-3	0.0001	1.0000	1.10
	0.0003	1.0000	1.10
	0.0010	1.0000	1.10
	0.0030	0.9700	1.13
	0.0100	0.8600	1.20
	0.0300	0.7400	1.50
	0.0548	0.6500	1.95
	0.1000	0.5600	2.40
	0.1732	0.4700	3.25
	0.3000	0.3900	4.10
	0.4243	0.3400	4.95
	0.6000	0.3000	5.80
	1.0000	0.2400	7.40
IIC-4	0.0001	1.0000	0.80
	0.0003	1.0000	0.80
	0.0010	1.0000	0.80
	0.0030	0.9900	0.90
	0.0100	0.9400	1.12
	0.0300	0.8200	1.50
	0.0548	0.7200	1.95
	0.1000	0.6200	2.40
	0.1732	0.5200	3.25
	0.3000	0.4200	4.10
	0.4243	0.3650	4.95
	0.6000	0.3100	5.80
	1.0000	0.2500	7.40

Table 2.5-63—{Strain Dependant Properties for Backfill}

Strata	Strain	G/Gmax	Damping [%]
BACKFILL	0.0001	1.0000	1.49
	0.0003	0.9700	1.57
	0.0010	0.8900	1.84
	0.0032	0.7400	2.71
	0.0100	0.5300	5.02
	0.0316	0.3000	9.38
	0.1000	0.1300	15.00
	0.3160	0.0600	-
	1.0000	0.0382	-

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Table 2.5-64—{Building Elevation, Depth, Area, and Load}

	Building	El. [ft]	Depth [ft]	Area [ft^2]	Load [kips]	Pressure [ksf]	Eq. Shape [ft] (1)
Nuclear Island	Reactor Building (RB)	41.5	41.5	26268	313477	11.9	270 x 300
	Fuel Building (FB)	41.5	41.5	14545	216806	14.9	
	Safeguard Building 1 (SGB1)	41.5	41.5	9198	108064	11.7	
	Safeguard 2&3 Buildings (SGB23)	41.5	41.5	20952	200814	9.6	
	Safeguard Building 4 (SGB4)	41.5	41.5	9247	104079	11.3	
Other Buildings	Nuclear Auxiliary Building (NAB)	48.0	35.0	12559	122000	9.7	105 x 120
	Access Building (AB)	48.0	35.0	7620	49300	6.5	95 x 80
	Rad. Waste Building (RWPB)	47.0	36.0	16970	109700	6.5	130 x 130
	E. Power Gen. Buildings (EPGB)	76.0	7.0	12611	40200	3.2	84 x 150
	E. Service Water Building (ESWB)	61.0	22.0	16284	88700	5.4	105 x 155
	Turbine Building (TB)	60.5	22.5	101305	446600	4.4	270 x 380
Intake Area	UHS Makeup Water Intake Structure (UHS MWIS)	-26.5	36.5	4284	24900	5.8	63 x 68
	UHS Electrical Building (UHS EB)	-10.5	20.5	2660	5050	1.9	35 x 76

Notes:
- (1) Equivalent Rectangular shape
Depth is based on average site grade elevation of 83 ft

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-65—{Bearing Capacity}

Building	Bulding Load [ksf]	Ultimate Bearing Capacity q_{ult} [ksf]			Allowable Bearing Capacity q_a [ksf] (1)	
		VESIC		MEYERHOF	STATIC	DYNAMIC
		Case a	Case b	Case c (2)		
NI Common Mat	11.8	192.7	228.9	70.5	23.5	35.2
NAB	9.7	170.7	179.1	105.8	35.3	52.9
EPGB	3.2	113.6	102.2	115.0	34.1	51.1
ESWB	5.4	145.7	153.8	118.0	39.3	59.0
UHS MWIS (3)	5.8	NA	36.0	NA	12.0	18.0
UHS EB (3)	1.9	NA	33.0	NA	11.0	16.5

Notes:
- (1) With FS = 3.0 for static conditions and FS = 2.0 for dynamic condition (minimum q_{ult} used)
- (2) Case c, Dense sand over soft clay
- (3) Case b with Stratum II-C used for UHS, other scenarios are not applicable (NA).

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Table 2.5-66—{Heave after Excavation} Measured at NI Foundation Level

Location	Vertical Displacement after Excavation [in]	
	Immediate	1 Year
A	-2.2	-2.4
B	-3.0	-3.5
C	-4.7	-5.3
D	-3.1	-3.5

Table 2.5-67—{Foundation Loading Sequence}

Building Name		Loads [ksf]								
	Step	0	1	2	3	4	5	6	7	8
	Day	0	60	140	300	500	800	1000	1400	2000
	Month	0	2	4	10	16	26	33	46	66
	Year	0	1	1	1	2	3	3	4	6
Reactor (RB)		0.0	0.1	0.2	2.1	3.8	7.5	9.6	9.8	11.9
Fuel (FB)		0.0	0.0	1.0	1.7	2.0	5.7	9.0	14.9	14.9
Safeguard 1 (SGB1)		0.0	0.0	0.8	3.8	5.9	8.6	11.8	11.8	11.8
Safeguard 2&3 (SGB23)		0.0	0.0	1.0	1.7	2.6	5.4	8.2	9.6	9.6
Safeguard (SGB4)		0.0	0.0	0.8	3.6	5.6	8.2	11.3	11.3	11.3
Nuclear Auxiliary (NAB)		0.0	0.0	0.9	1.8	3.5	5.3	7.1	9.7	9.7
Access (AB)		0.0	0.0	0.9	1.9	2.8	4.6	5.6	6.5	6.5
Radioactive Waste (RWPB)		0.0	0.0	0.0	0.0	0.0	0.7	2.2	6.5	6.5
Emergency Power Gen. (EPGB)		0.0	0.0	0.0	0.0	0.0	0.0	0.5	3.2	3.2
Emergency Service Water (ESWB)		0.0	0.0	0.0	0.0	0.0	1.6	5.5	5.5	5.5
Turbine (TB)		0.0	0.0	0.0	0.6	1.8	3.2	4.4	4.4	4.4
Turbine Extension (TBE)		0.0	0.0	0.0	0.6	1.8	3.2	4.4	4.4	4.4

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October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Table 2.5-68—{Building Center Point Settlement Estimates}

Building Name		Settlement [in] (Medium Elevation Surface Topography)(1)							
	Step	1	2	3	4	5	6	7	8
	Day	60	140	300	500	800	1000	1400	2000
	Month	2	4	10	16	26	33	46	66
	Year	1	1	1	2	3	3	4	6
Reactor (RB)		0.3	1.1	2.0	3.1	7.1	10.2	12.1	12.7
Fuel (FB)		0.3	1.3	2.0	3.0	6.8	9.8	12.4	13.0
Safeguard 1 (SGB1)		0.3	1.4	2.3	3.3	7.1	10.1	11.4	12.0
Safeguard 2&3 (SGB23)		0.3	1.4	2.2	3.2	7.0	9.9	11.1	11.6
Safeguard (SGB4)		0.3	1.3	2.2	3.2	6.9	9.8	12.1	12.5
Nuclear Auxiliary (NAB)		0.4	1.4	2.2	3.2	6.5	9.1	12.0	12.3
Access (AB)		0.4	1.6	2.4	3.5	7.0	9.8	11.4	11.7
Radioactive Waste (RWPB)		0.5	1.3	1.9	2.7	5.1	6.9	9.4	9.6
E. Service Water 1 (ESWB1)		0.0	1.6	2.0	2.6	4.7	7.0	7.4	7.4
E. Service Water 2 (ESWB2)		0.0	1.7	2.2	2.9	5.5	8.3	8.9	9.1
E. Service Water 3 (ESWB3)		0.0	2.0	2.5	3.3	6.0	8.8	9.1	9.2
E. Service Water 4 (ESWB4)		0.0	1.9	2.3	3.0	5.3	7.9	8.1	8.2
E. Power Generating (EPBG1)		0.0	0.0	0.0	3.8	5.9	7.6	9.5	9.6
E. Power Generating (EPBG2)		0.0	0.0	0.0	3.7	5.7	7.1	8.5	8.7

Notes:

- (1) Settlement estimates correspond to Medium Elevation Surface Topography 2, Revert after 4th Step

Table 2.5-69—{Maximum Tilt at End of Construction}

Building	Section	Tilt Direction (1)	Tilt [in/50 ft]	
			Tilt Average Elevation Case(2)	Construction Baseline(3)
NI	AA	N	-0.10	-0.10
	BB	E	0.27	0.27
	CC	SW	-0.21	-0.21
	DD	SE	0.03	0.03
ESWB1	EE	NW	-0.80	-0.59
	FF	NE	-0.17	-0.18
ESWB2	GG	SW	0.29	0.23
	HH	NW	-0.88	-0.72
ESWB3	II	NE	0.13	-0.17
	JJ	SE	0.19	0.13
ESWB4	KK	SW	0.36	0.28
	LL	SE	0.53	0.42
EPBG1	MM	SW	0.35	0.16
	NN	SE	0.68	0.49
EPBG2	OO	NE	0.72	-0.42
	PP	NW	0.37	-0.14

Notes
- (1) Local Plant Coordinates
- (2) Tilt recorded with calculation for Medium Elevation Surface Topography Revert 2
Sign is positive for clockwise tilt and negative for counter-clockwise
- (3) Correction to subtract the observed tilt before the construction of the building

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Table 2.5-70—{Settlement and Tilt for UHS Facilities}

Building	Δ Center [in]	Δ Edge [in]	Maximum Tilt (1) [in/50 ft]
Ultimate Heat Sink Makeup Water Intake Structure (UHS MWIS)	3.5	3.5	0.1
Ultimate Heat Sink Electrical Building (UHS RB)	3.1	3.3	0.7
Cooling Water Makeup Intake Structure (CW MIS)	3.2	3.2	0.1

Notes

- (1) Adjustment for construction not incorporated.

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Table 2.5-71—{Material Properties for Slope Stability}

Stratum	Material Property	Powerblock[1]	Intake Area & Intake Slope	Utility Corridor
Structural Backfill	Unit Weight (pcf)	145	-	-
	c (psf)	0	-	-
	ϕ (degrees)	40	-	-
	c' (psf)	0	-	-
	ϕ' (degrees)	40	-	-
Stratum I: Terrace Sand	Unit Weight (pcf)	120	120	120
	c (psf)	1100	1100	1100
	ϕ (degrees)	13	13	13
	c' (psf)	0	0	0
	ϕ' (degrees)	32	32	32
Stratum IIa: Chesapeake Clay/Silt	Unit Weight (pcf)	115	115	115
	c (psf)	2500	3400	2500
	ϕ (degrees)	0	0	0
	c' (psf)	900	1400	900
	ϕ' (degrees)	25	28	25
Stratum IIb: Chesapeake Cemented Sand	Unit Weight (pcf)	120	120	120
	c (psf)	2800	2800	2800
	ϕ (degrees)	17	17	17
	c' (psf)	0	0	0
	ϕ' (degrees)	34	34	34
Stratum IIc: Chesapeake Clay/Silt	Unit Weight (pcf)	105	110	105
	c (psf)	5000	4800	5000
	ϕ (degrees)	0	0	0
	c' (psf)	2300	1000	2300
	ϕ' (degrees)	26	26	26

Notes:

(1) Powerblock includes the Construction Laydown Area

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Table 2.5-72—{Computed Factors of Safety for Critical Slip Surface}

Slope Section	Affected Area	Effective Stress Conditions						Total Stress Conditions[1]		
		Static Analysis			Pseudo-static (Dynamic) Analysis			Pseudo-static (Dynamic) Analysis		
		Ordinary	Bishop	M-P	Ordinary	Bishop	M-P	Ordinary	Bishop	M-P
A - Case a	Powerblock	1.92	2.19	2.18	*1.32*	*1.47*	*1.47*	1.73	1.76	1.76
A - Case b	Powerblock	1.63	1.89	1.89	*1.14*	*1.27*	*1.28*	1.61	1.68	1.68
B - Case a	Powerblock	1.95	2.22	2.22	*1.35*	*1.49*	*1.49*	1.76	1.81	1.81
B - Case b	Powerblock	1.85	2.12	2.12	*1.23*	*1.40*	*1.41*	1.74	1.78	1.79
C	Powerblock	1.96	2.02	2.02	*1.31*	*1.36*	*1.36*	3.15	3.24	3.24
D	Powerblock	1.93	1.97	1.97	*1.32*	*1.38*	*1.38*	4.09	4.14	4.14
E	Powerblock	1.98	2.05	2.05	*1.34*	*1.41*	*1.41*	3.15	3.15	3.15
F	Intake Area	2.20	2.34	2.34	*1.57*	*1.68*	*1.69*	2.73	2.81	2.82
G	Utility Corridor	1.87	2.04	2.05	*1.24*	*1.34*	*1.35*	1.86	1.92	1.93

Notes:

Ordinary = Ordinary method

Bishop = Bishop's simplified method

M-P = Morgenstern-Price method

Typical minimum acceptable values of FOS are 1.5 for static conditions and 1.0 to 1.2 for pseudo-static (e.g., earthquake) conditions (Duncan, 1996)

(1) Total stress conditions are more representative of dynamic conditions are not used in the discussion.

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Figure 2.5-103 -{Site Utilization Plan with Boring Locations}

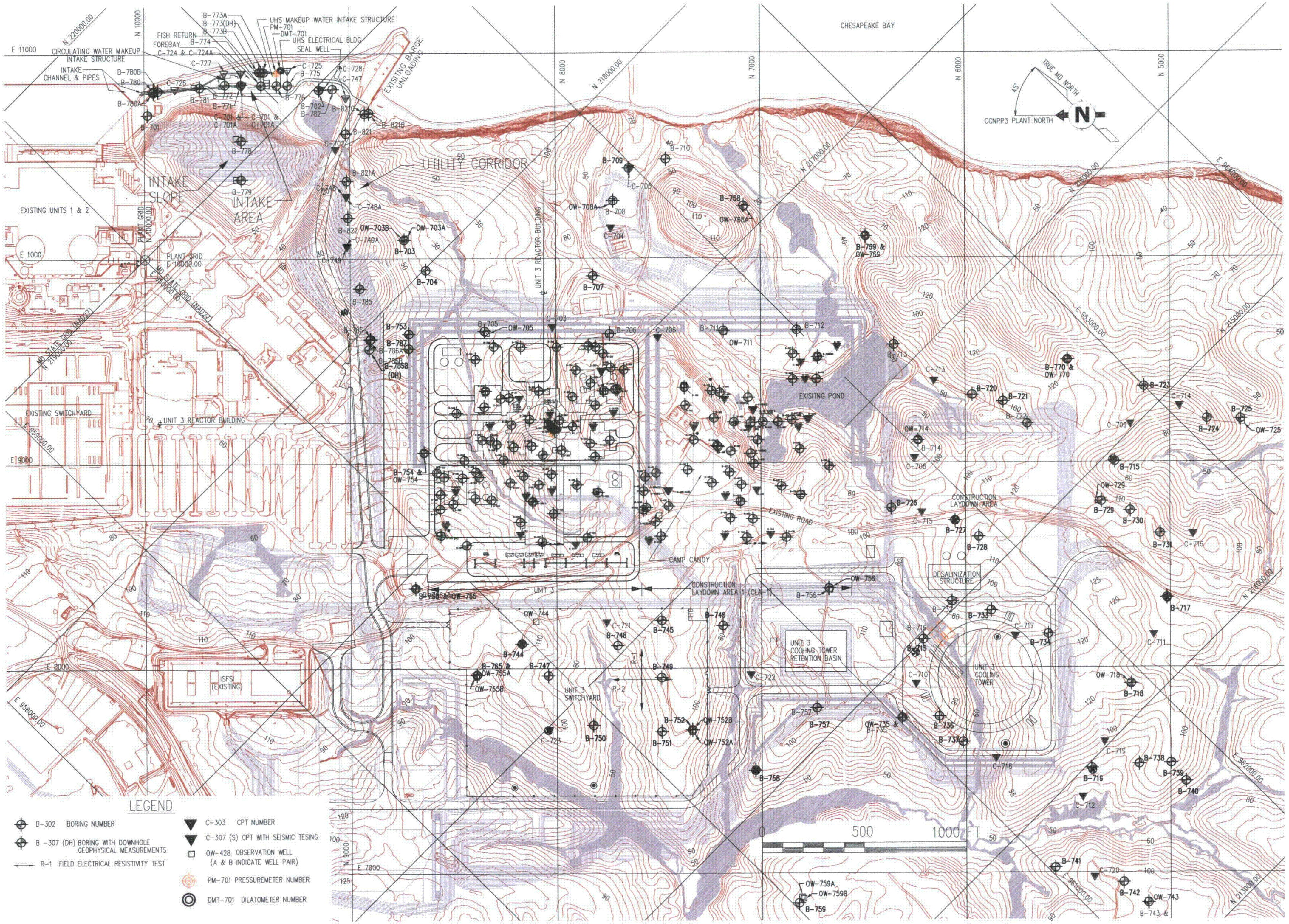
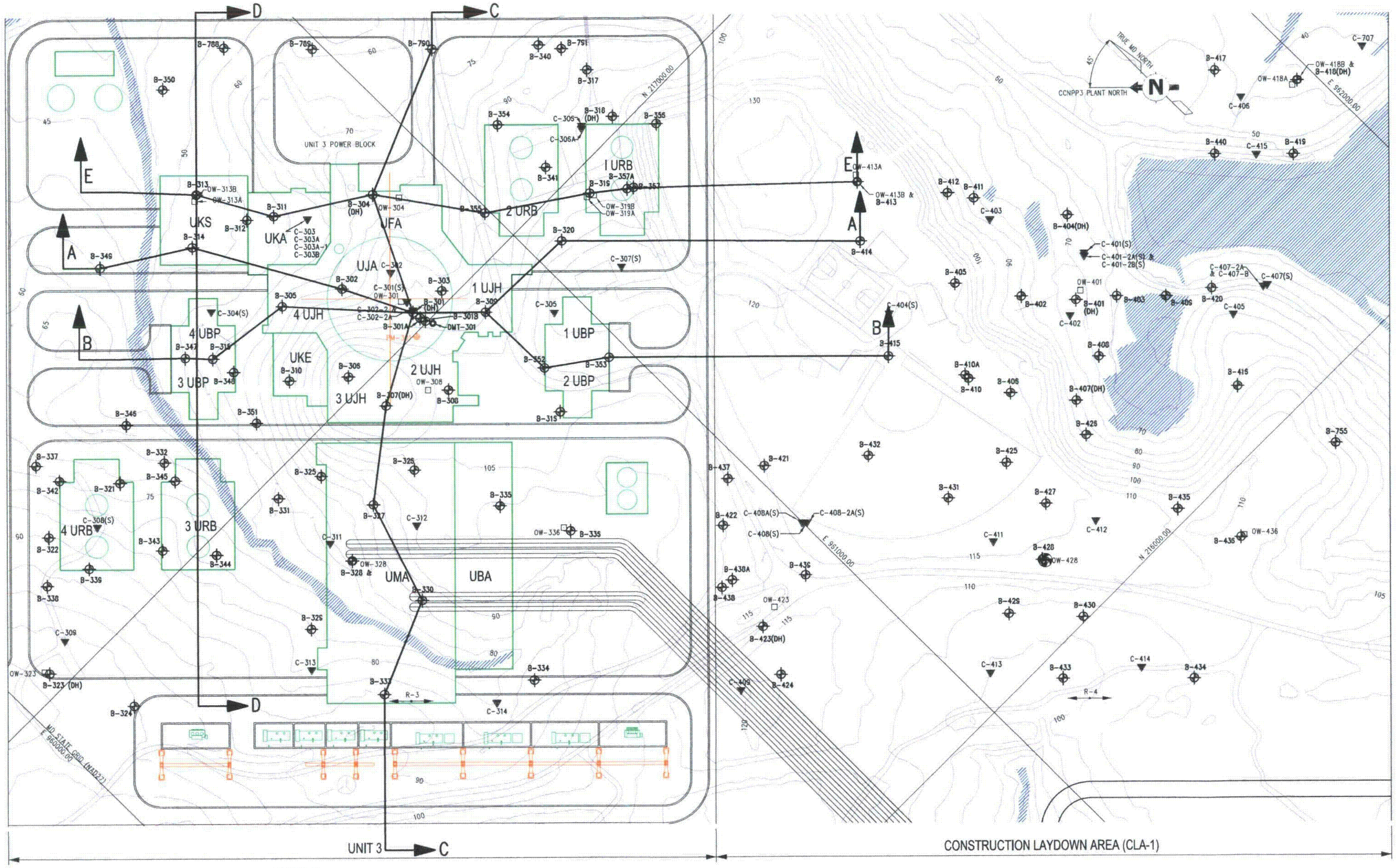


Figure 2.5-104 - {Powerblock Area Profile Locations}



0 100 200 300FT

LEGEND

1-4 UBP EMERGENCY POWER GENERATING BUILDINGS
UFA FUEL BUILDING
UJA REACTOR BUILDING
1-4 UJH SAFEGUARD BUILDINGS MECHANICAL/ELECTRICAL
UKA NUCLEAR AUXILIARY BUILDING
UKE ACCESS BUILDING
UKS RADIOACTIVE WASTE PROCESSING BUILDING
UMA TURBINE BUILDING
UBA SWITCHGEAR BUILDING
1-4 URB ESSENTIAL SERVICE WATER BUILDING

B-302 BORING NUMBER
B-307(DH) BORING WITH DOWNHOLE GEOPHYSICAL MEASUREMENTS
C-303 CPT NUMBER
C-307(S) CPT WITH SEISMIC TESTING
PM-301 PRESSUREMETER NUMBER
DMT-301 DILATOMETER NUMBER
OW-428 OBSERVATION WELL (A&B INDICATE WELL PAIR)
R-1 FIELD ELECTRICAL RESISTIVITY TEST
A PROFILE LOCATION

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Figure 2.5-105—{Intake Area Profile Location}

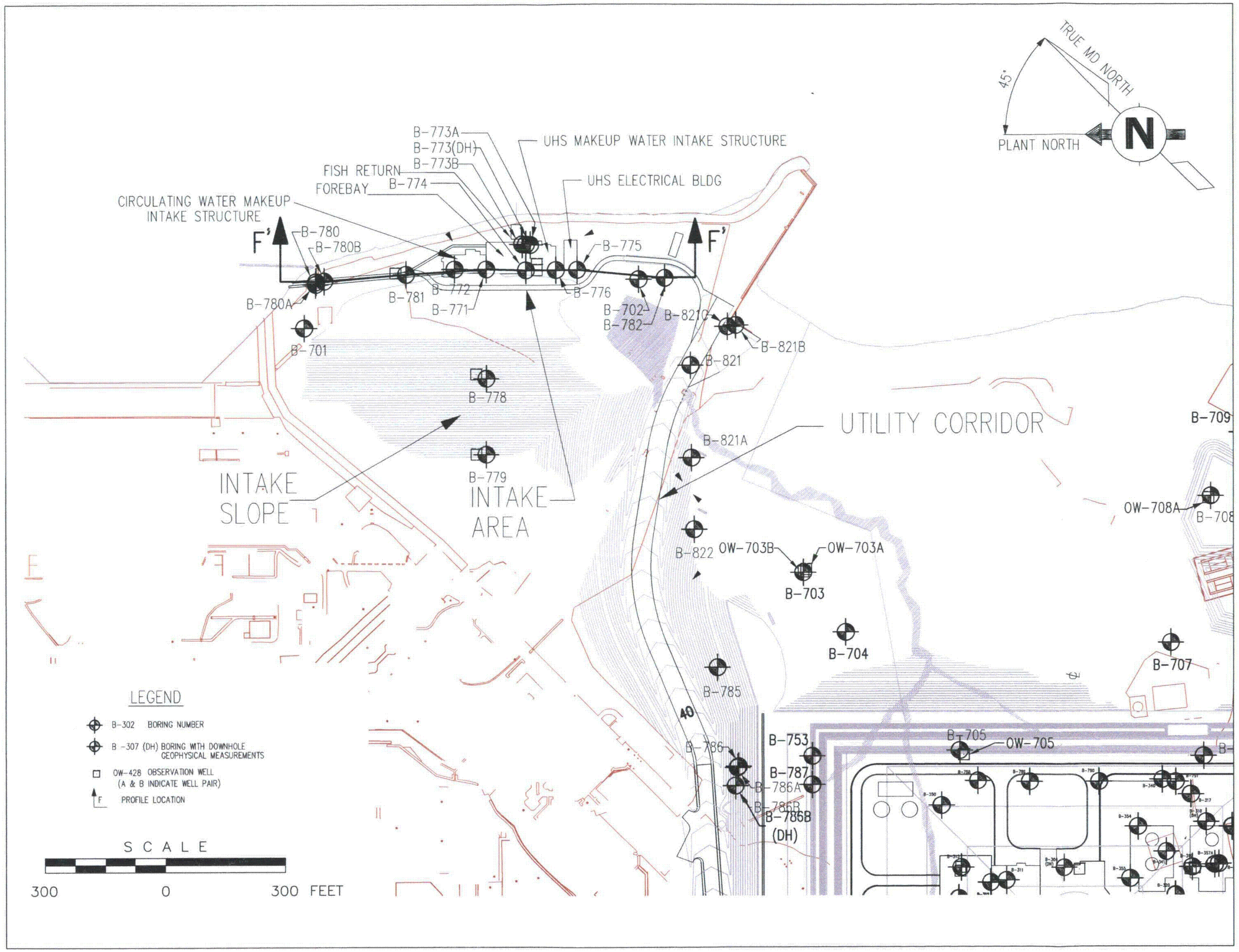

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Figure 2.5-106—{Generalized CCNPP Soil Column}

UNIT	THICKNESS [ft]		
	MIN	MAX	AVG
STRATUM I - TERRACE SAND	1	68	28
STRATUM IIa - CHESAPEAKE CLAY/SILT	4	36	19
STRATUM IIb - CHESAPEAKE CEMENTED SAND 1	3	69	24
STRATUM IIb - CHESAPEAKE CEMENTED SAND 2	3	55	23
STRATUM IIb - CHESAPEAKE CEMENTED SAND 3	4	39	16
STRATUM IIc - CHESAPEAKE CLAY/SILT (FROM B-301 AND B-401); INTERBEDDED SAND LAYERS	190	195	193
STRATUM III - NANJEMOY SAND (FROM B-301 and B-401)	>101	>115	>108

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Figure 2.5-107—{Subsurface Profile A-A'}

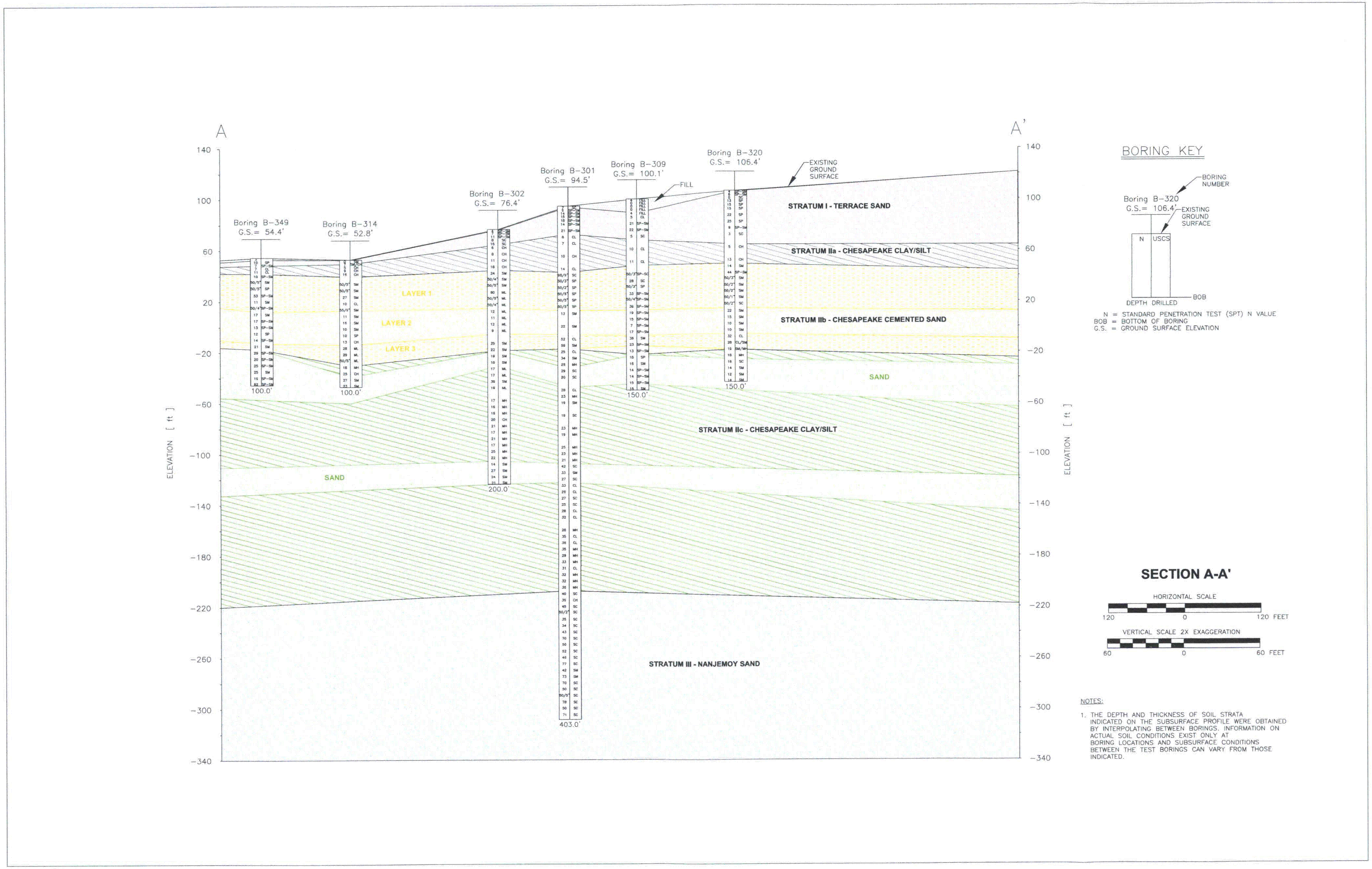

Figure 2.5-108—{Subsurface Profile B-B'}

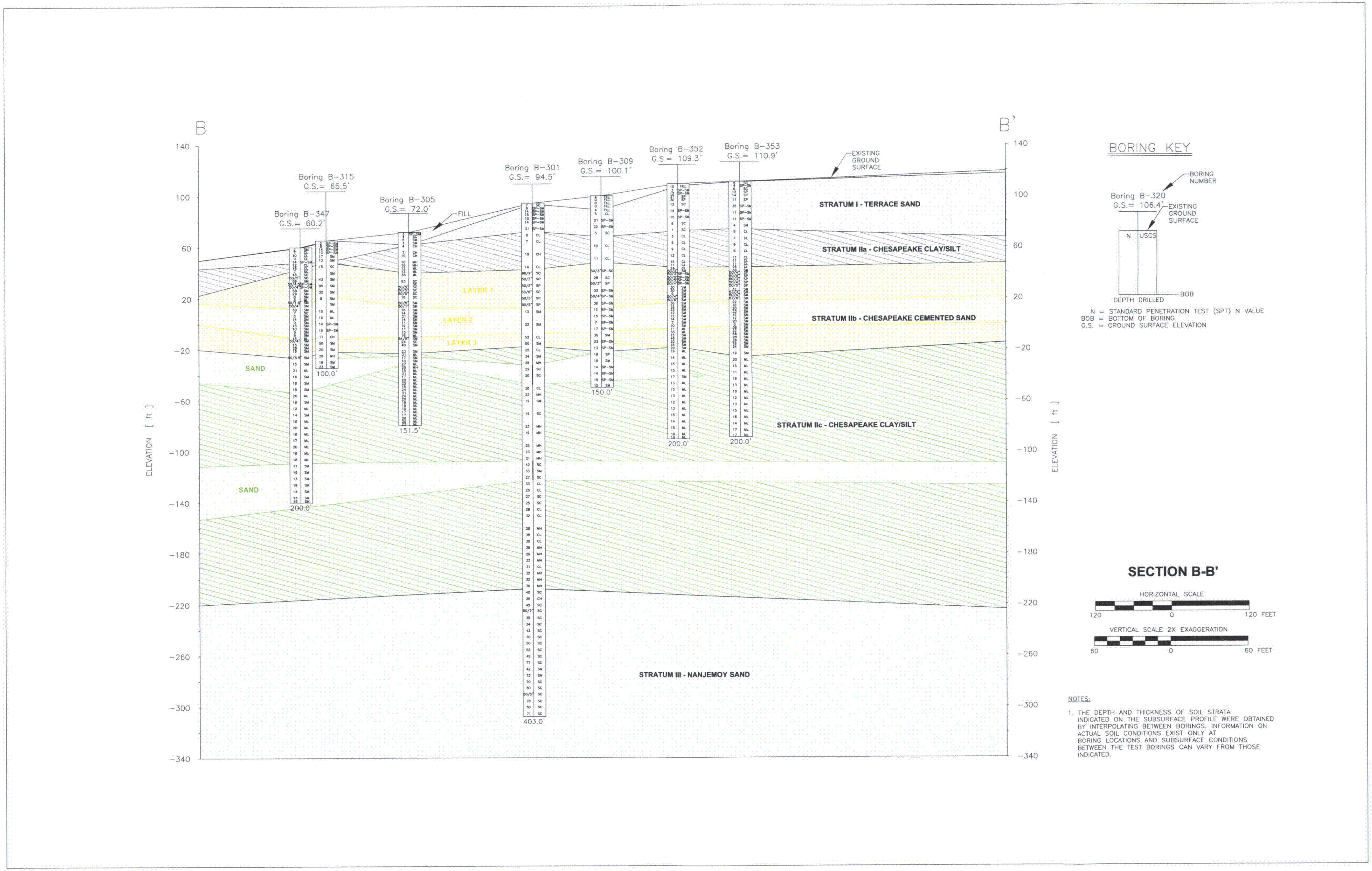

Figure 2.5-109—{Subsurface Profile C-C'}

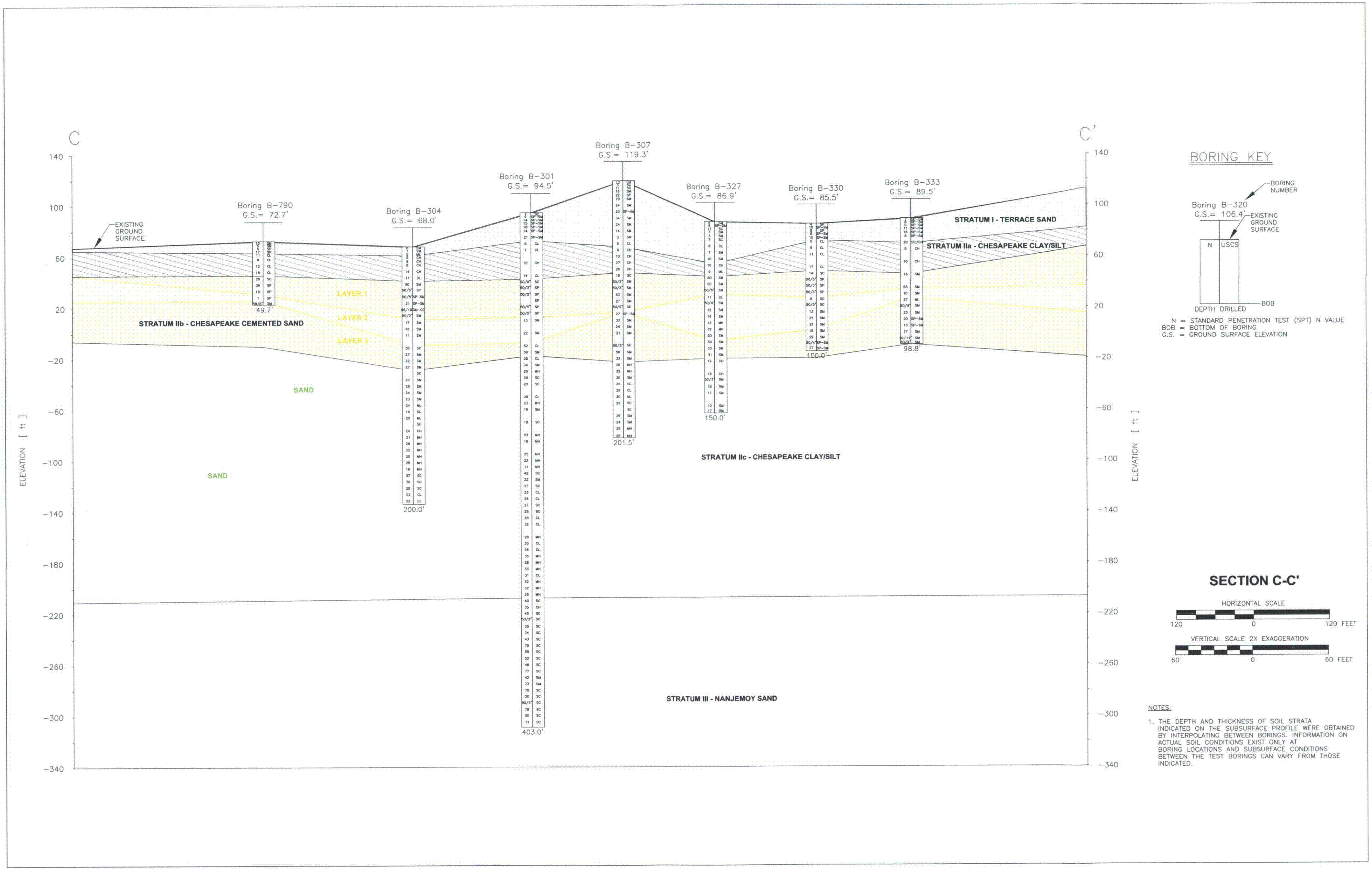

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Figure 2.5-110—{Subsurface Profile D-D'}

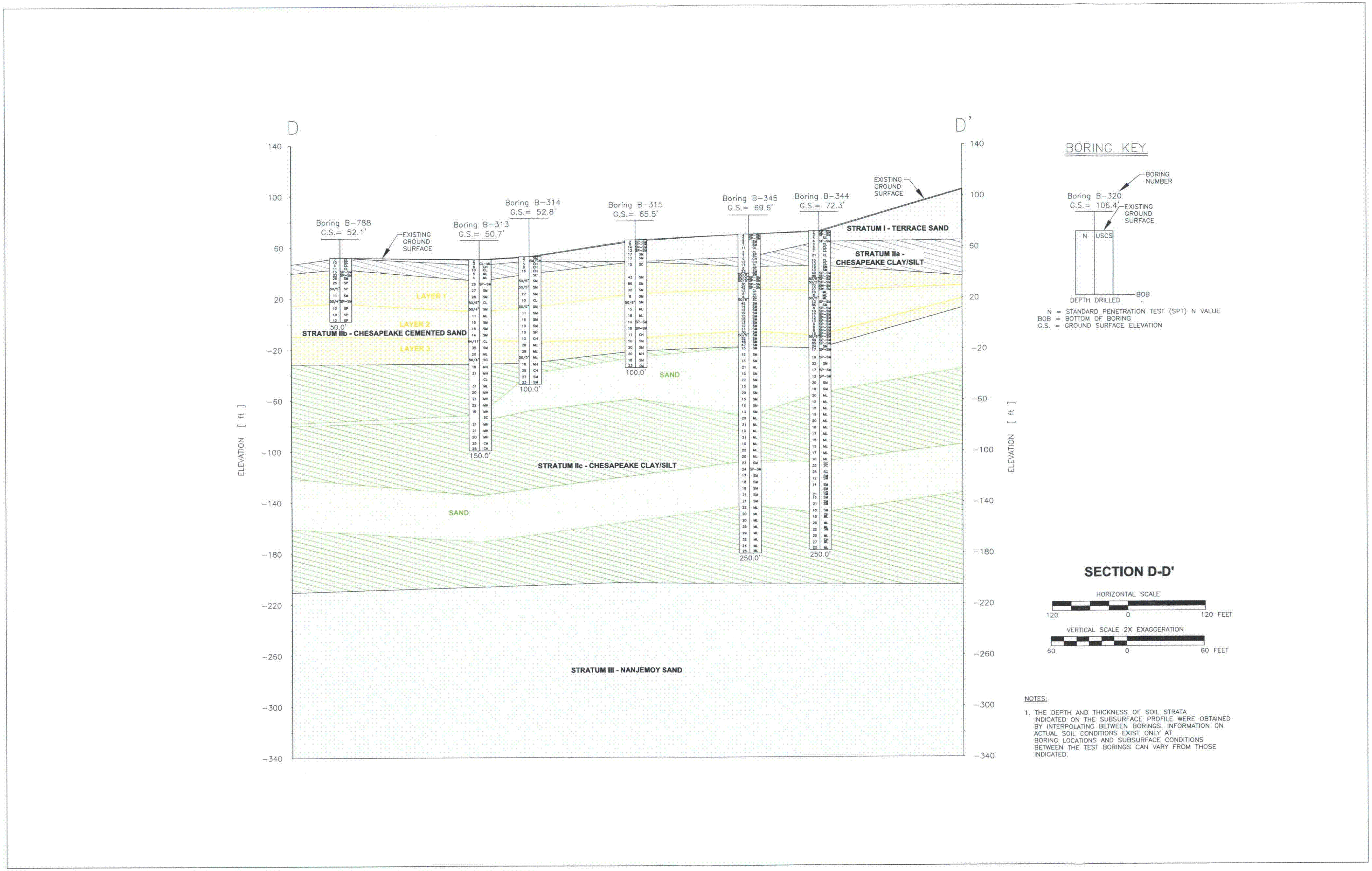
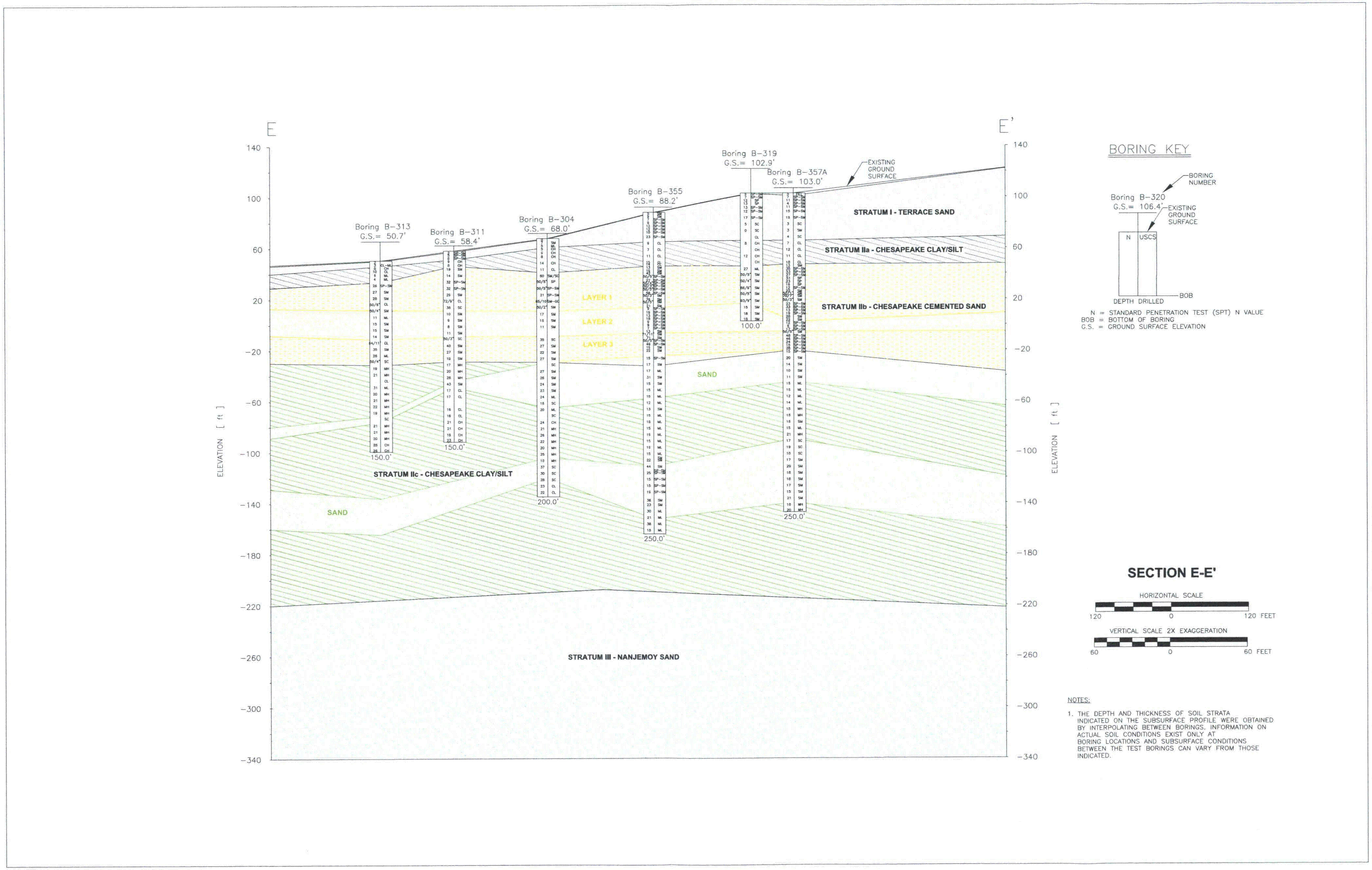
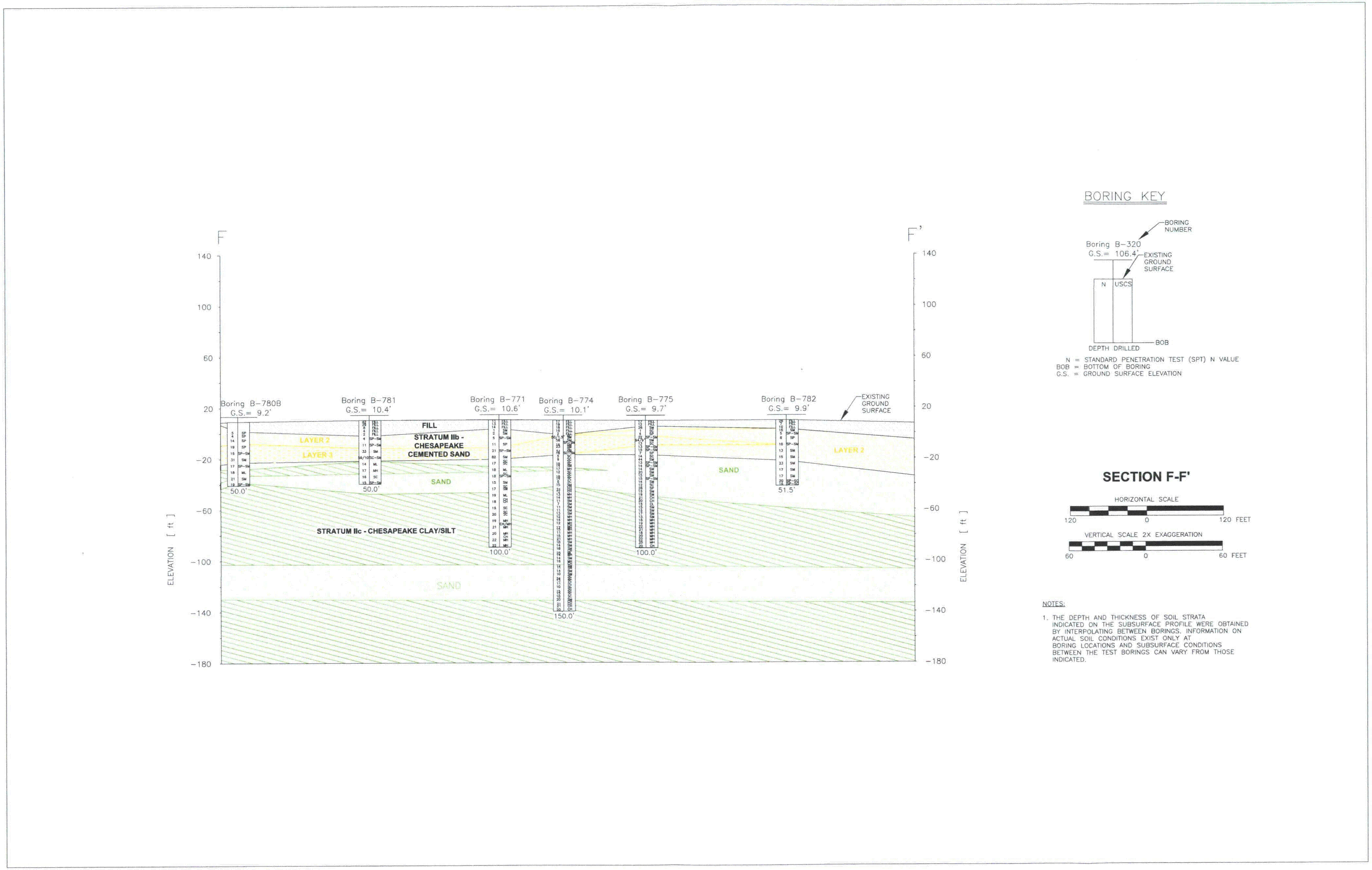


Figure 2.5-111—{Subsurface Profile E-E'}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-112—{Subsurface Profile F-F'}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-113—{SPT Data for Powerblock Area}

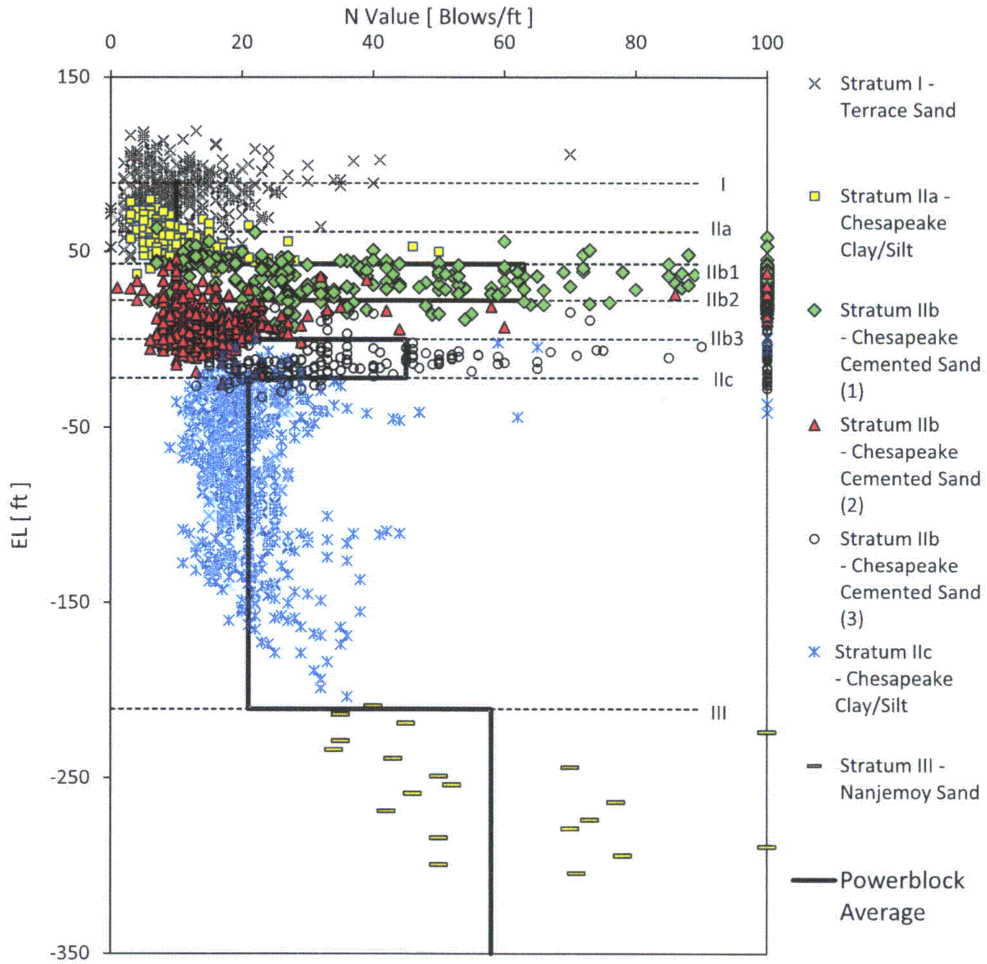


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Figure 2.5-114—{SPT Data for Intake Area}

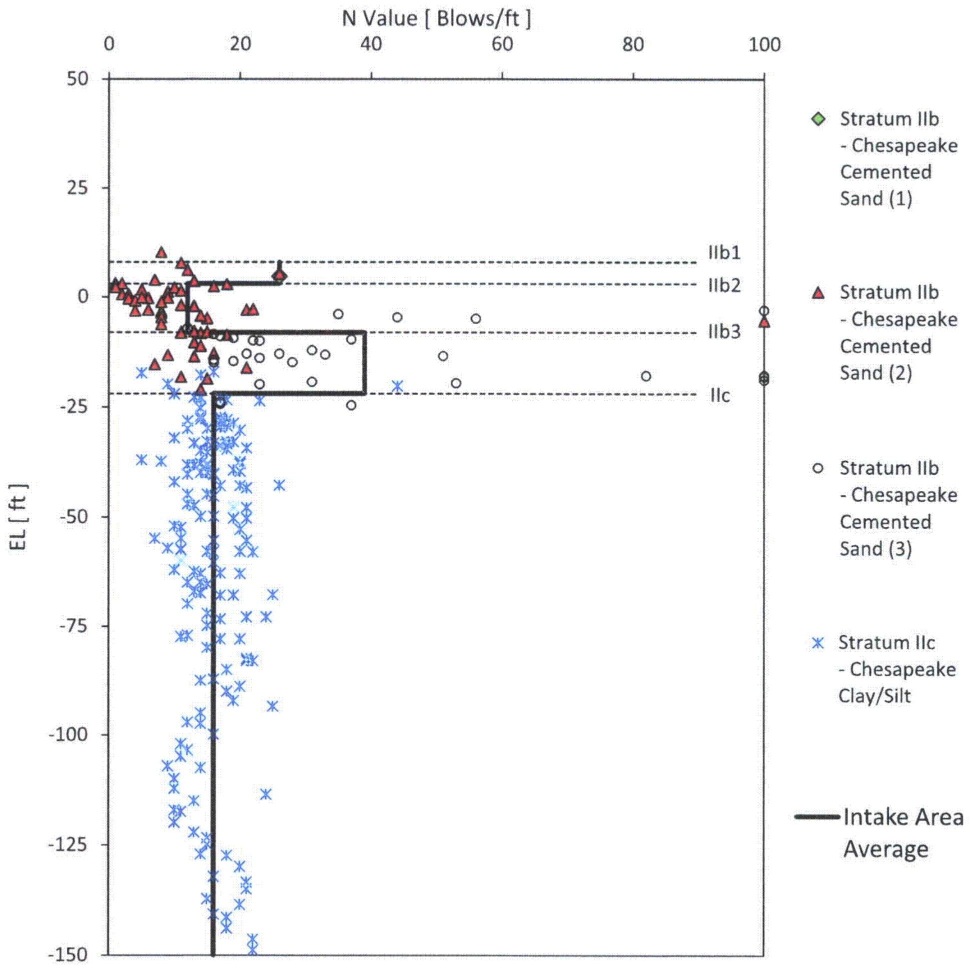
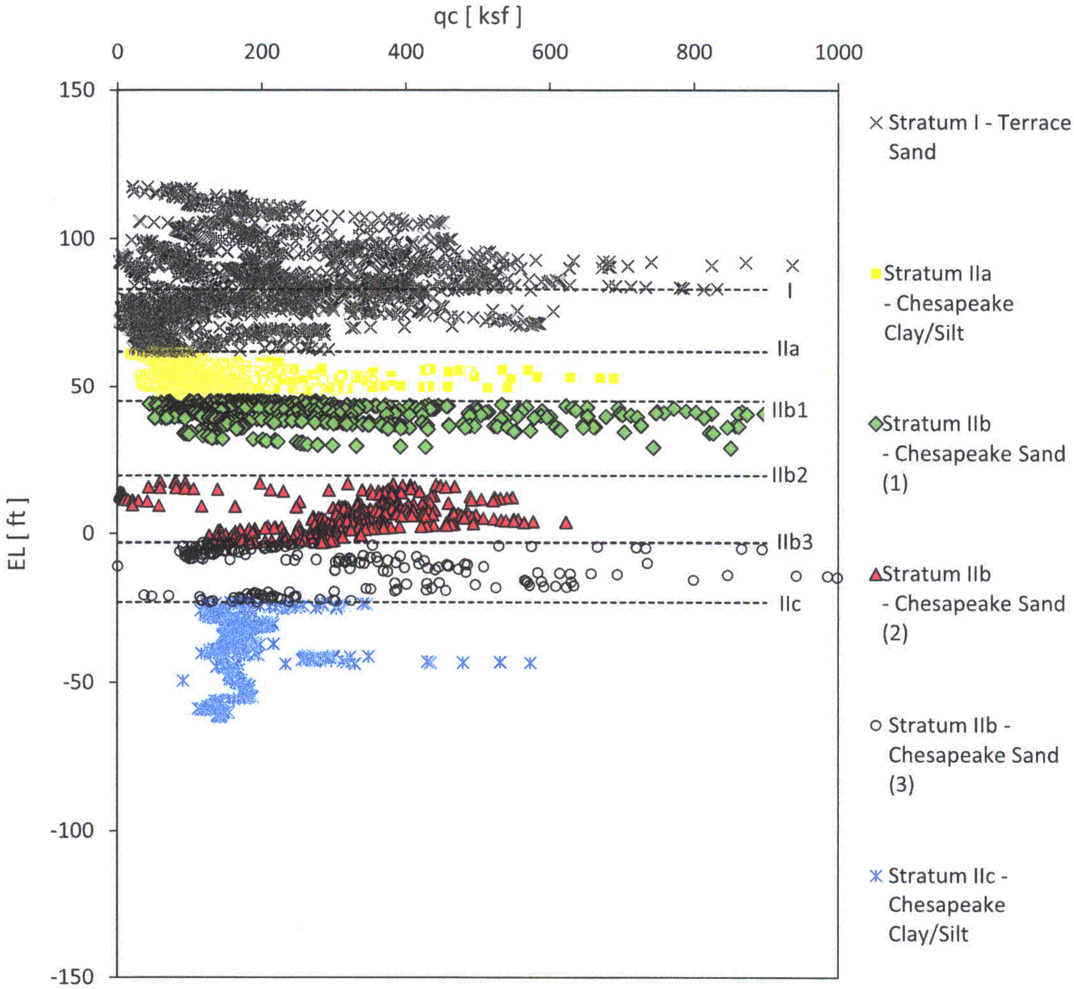
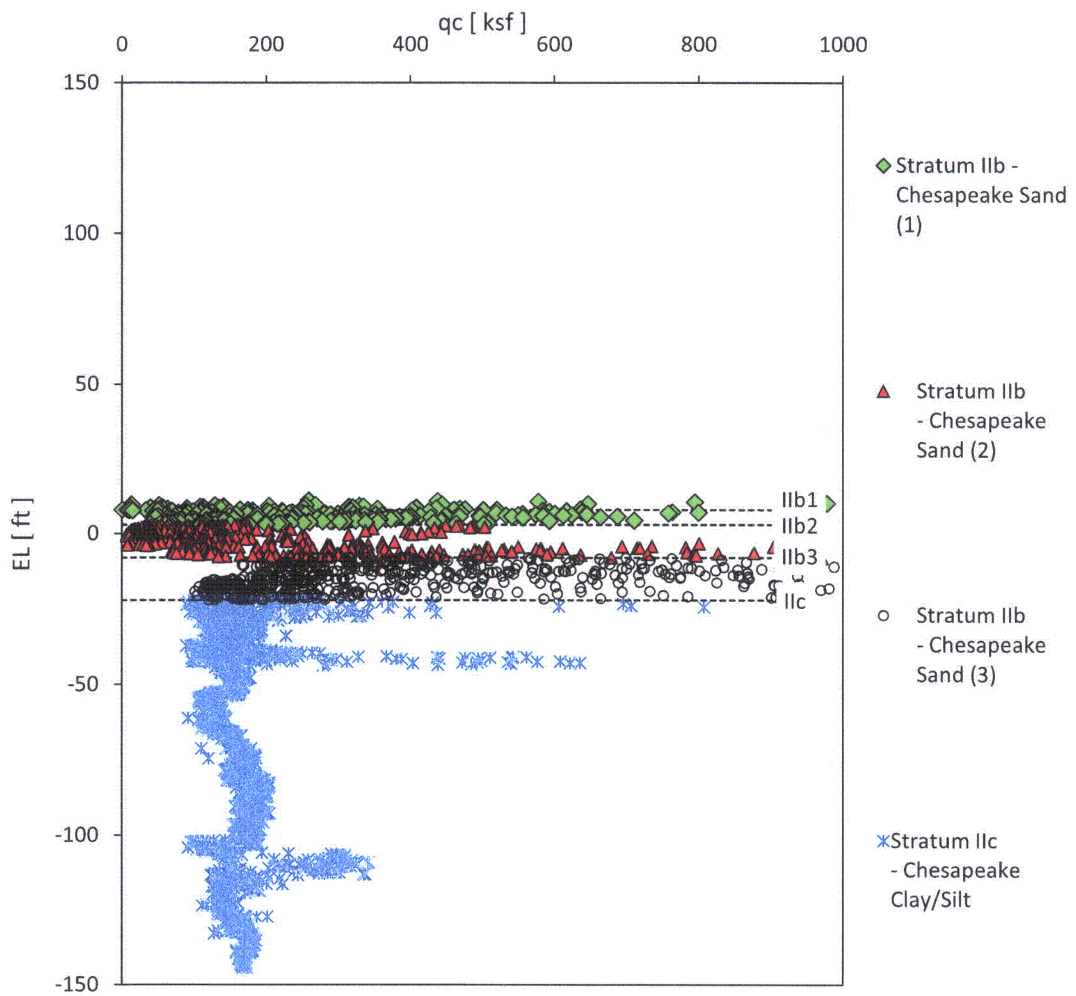


Figure 2.5-115—{CPT Tip Resistance, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

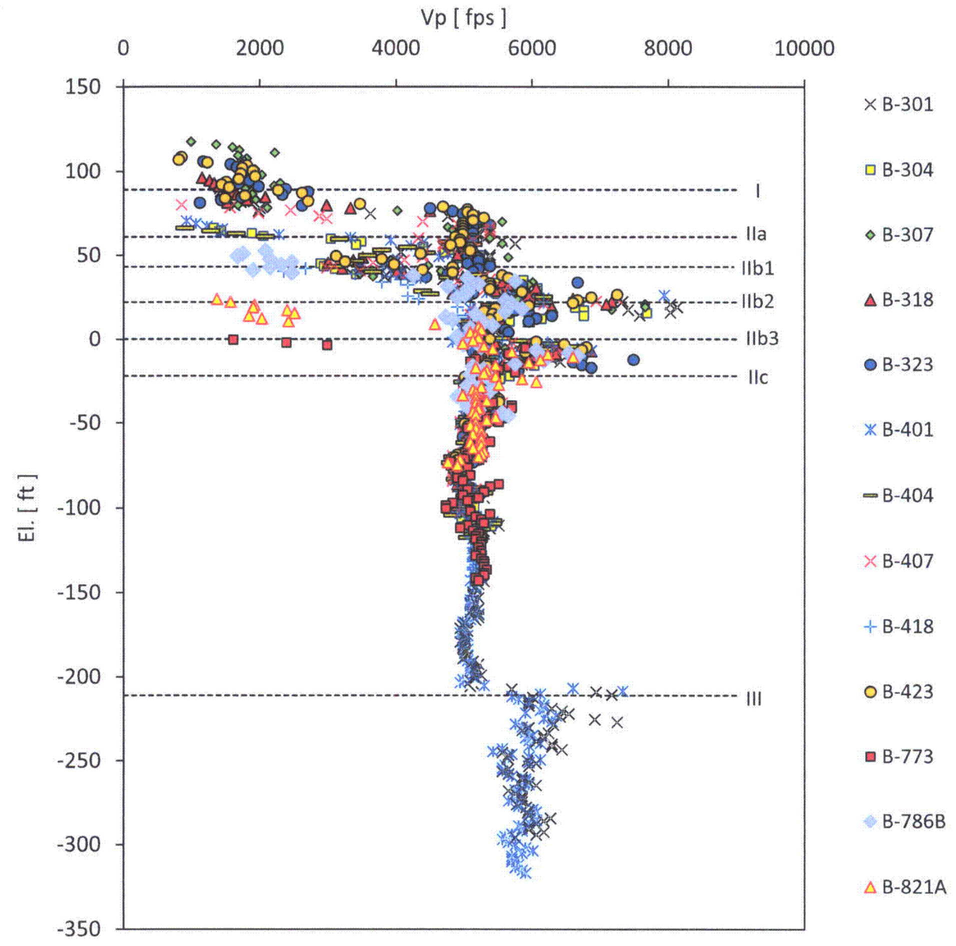
Figure 2.5-116—{CPT Tip Resistance, Intake Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-117—{V_p Measurements from Suspension P-S Velocity Logging}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-118—{V_s Measurements from Suspension P-S Velocity Logging}

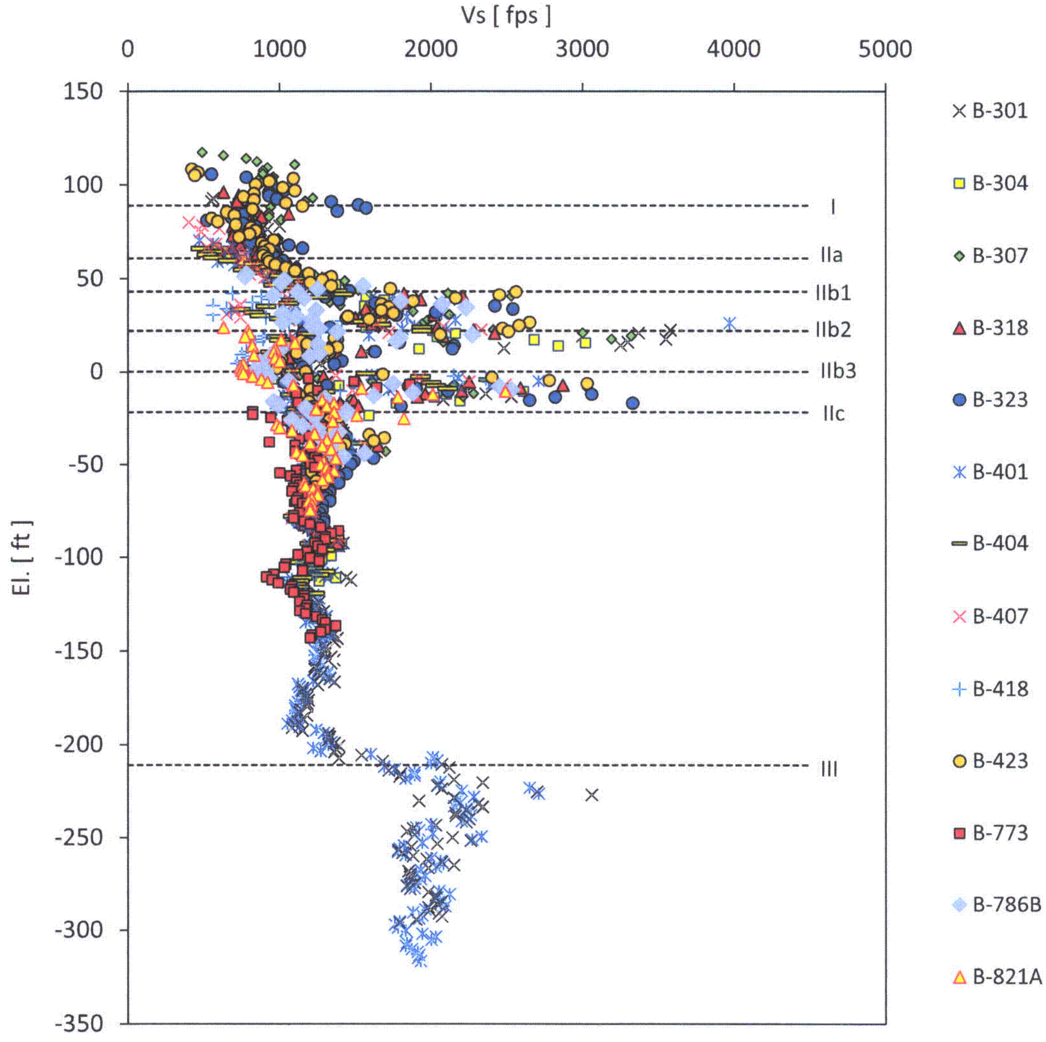
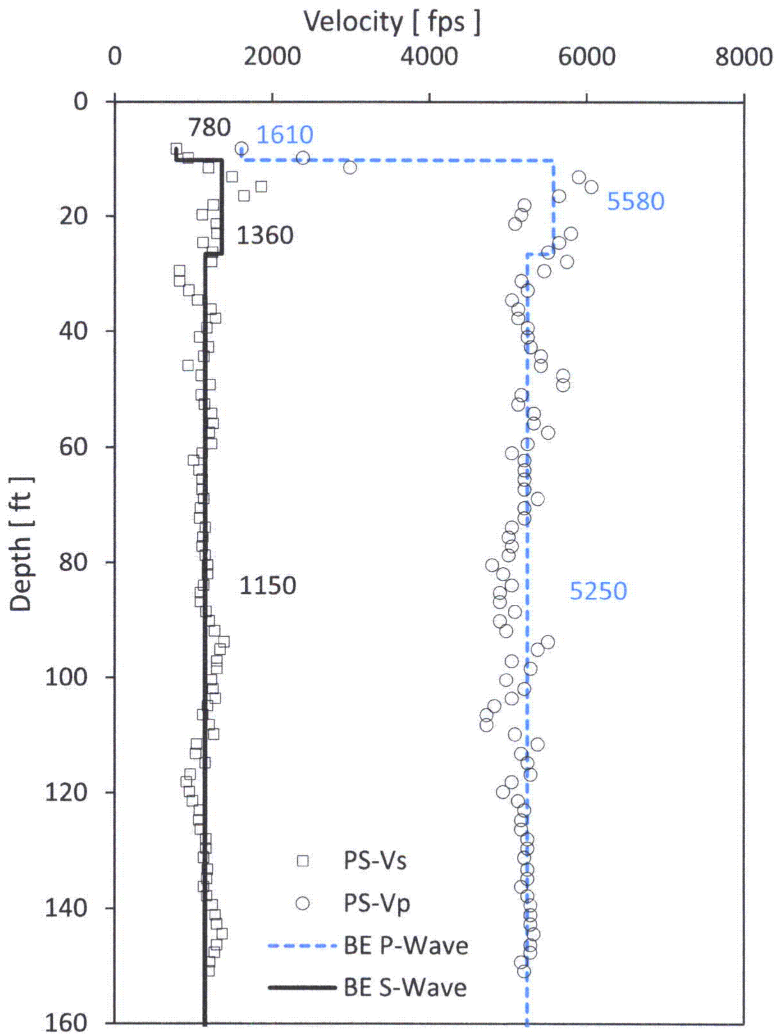


Figure 2.5-119—{PS Logging Test at Intake Area B-773}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-120—{Uphole Seismic Survey Results from CCNPP Units 1 and 2 UFSAR}

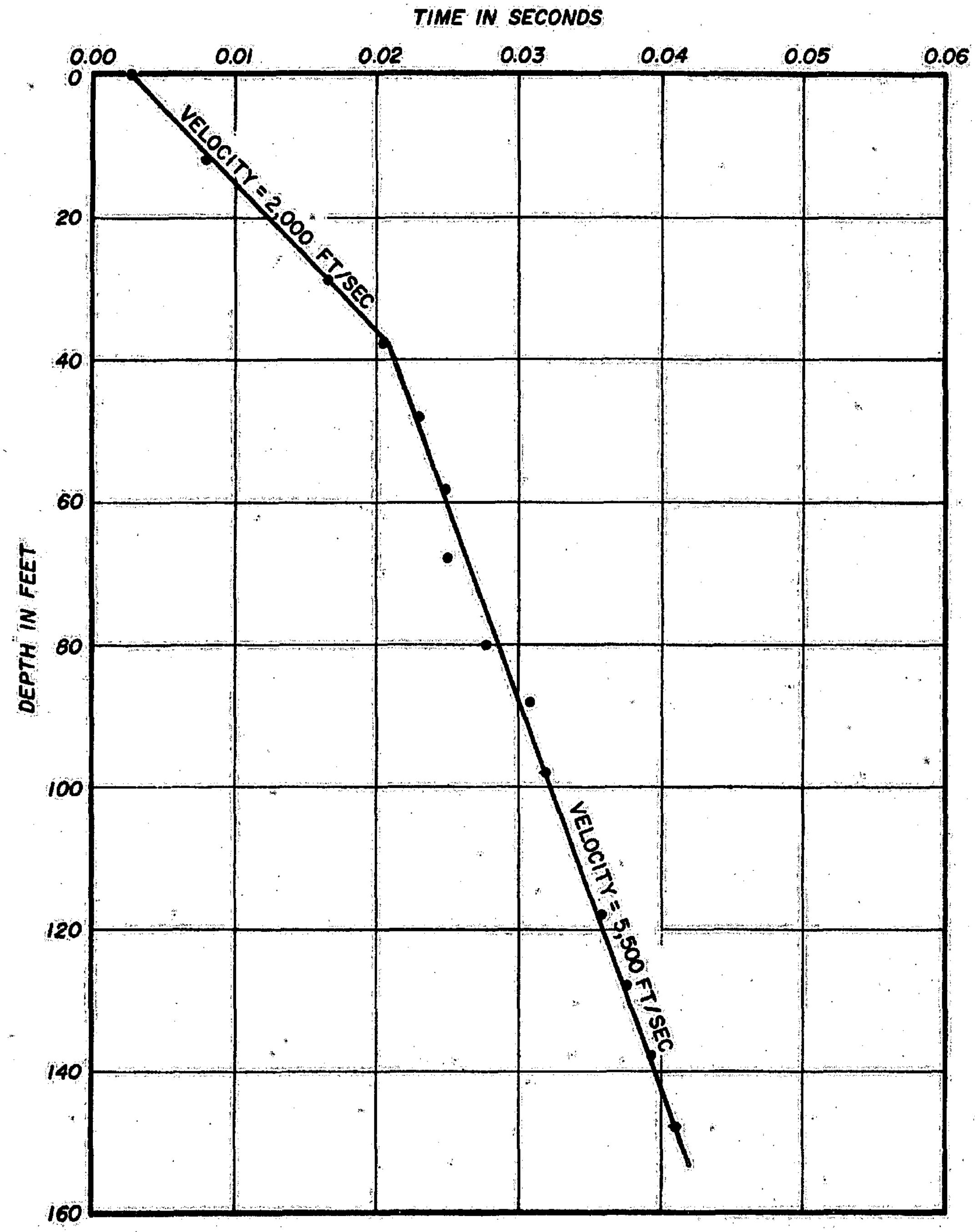


Figure 2.5-121—{Pressuremeter Data}

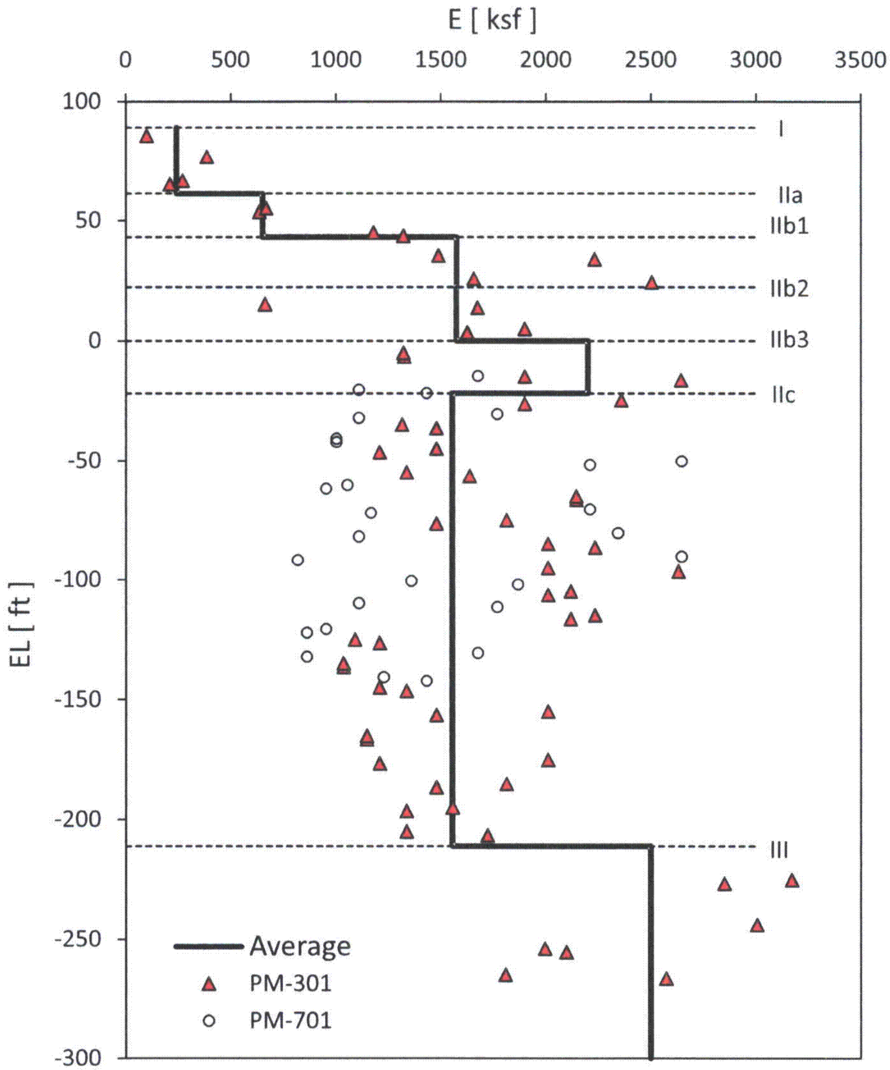
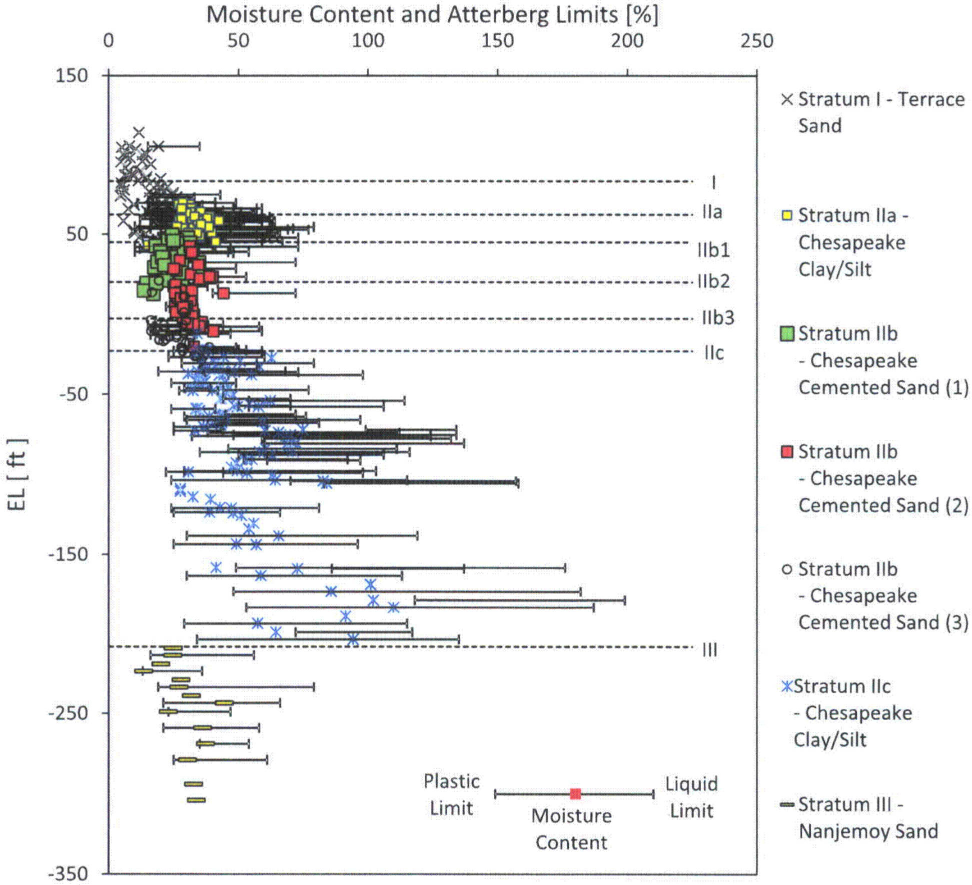


Figure 2.5-122—{Moisture Content and Atterberg Limits, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-123—{Moisture Content and Atterberg Limits, Intake Area}

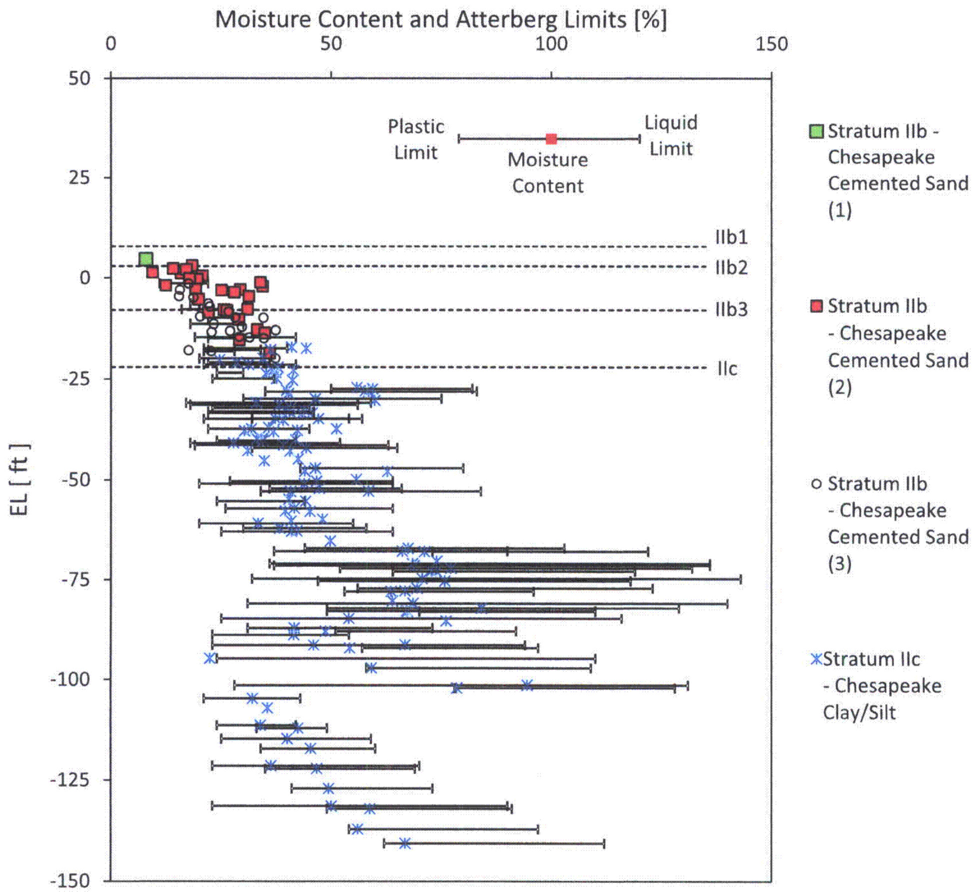
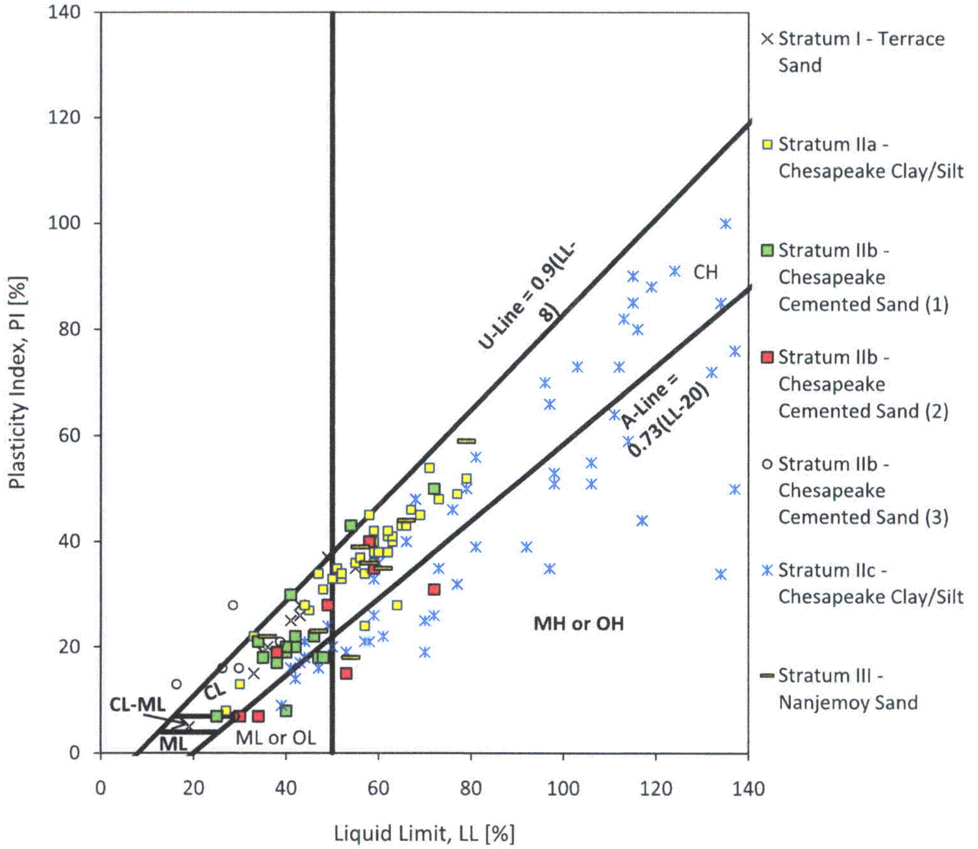


Figure 2.5-124—{Plasticity Chart, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-125—{Plasticity Chart, Intake Area}

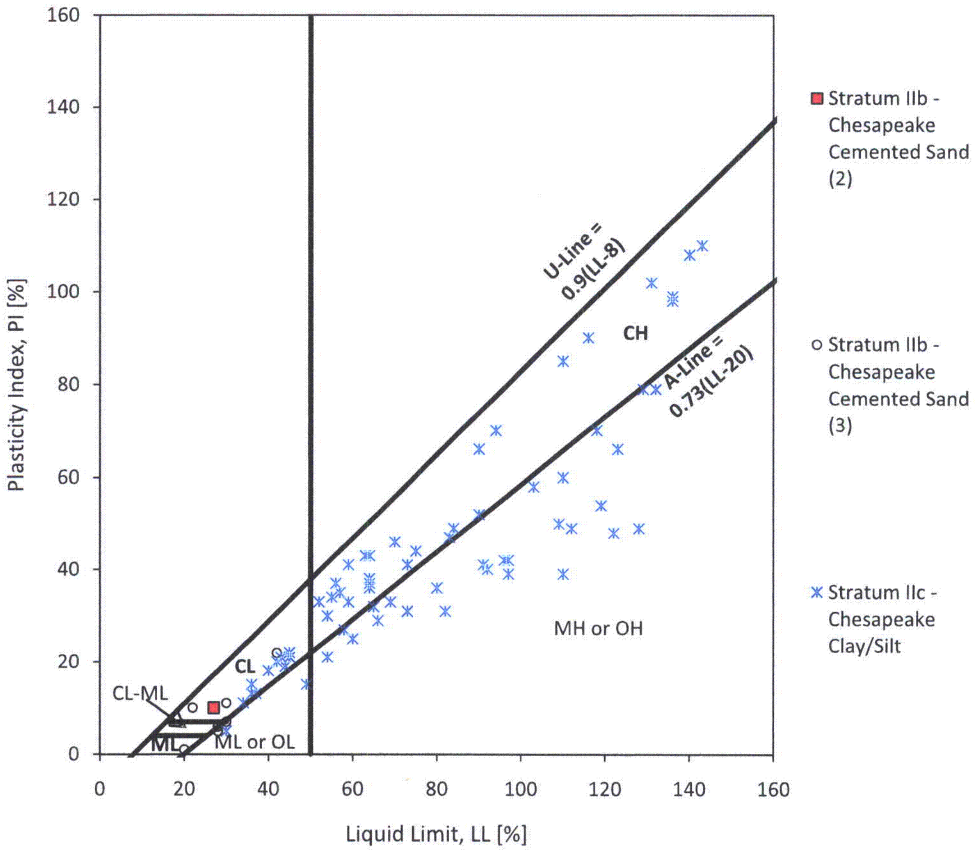
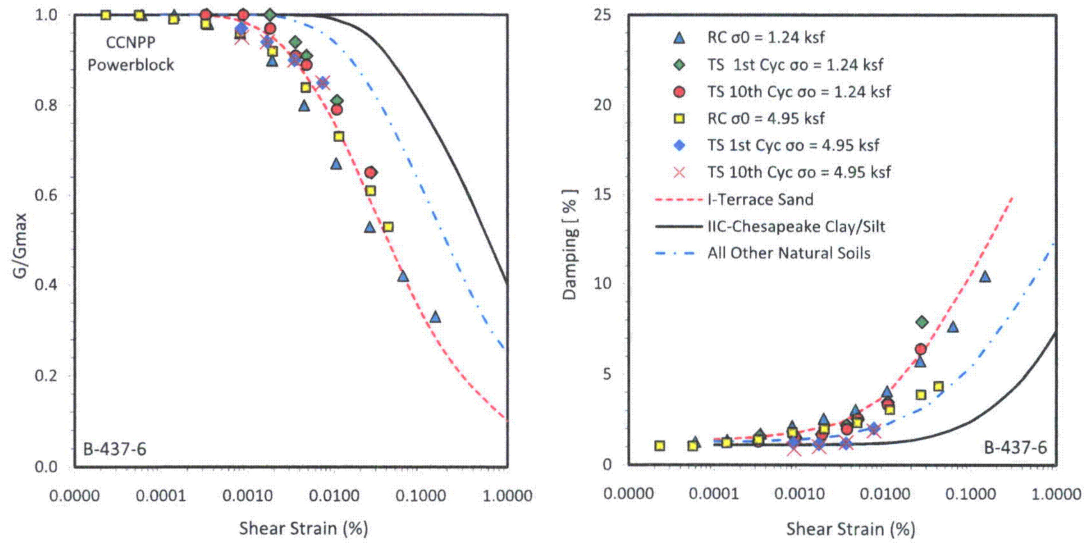
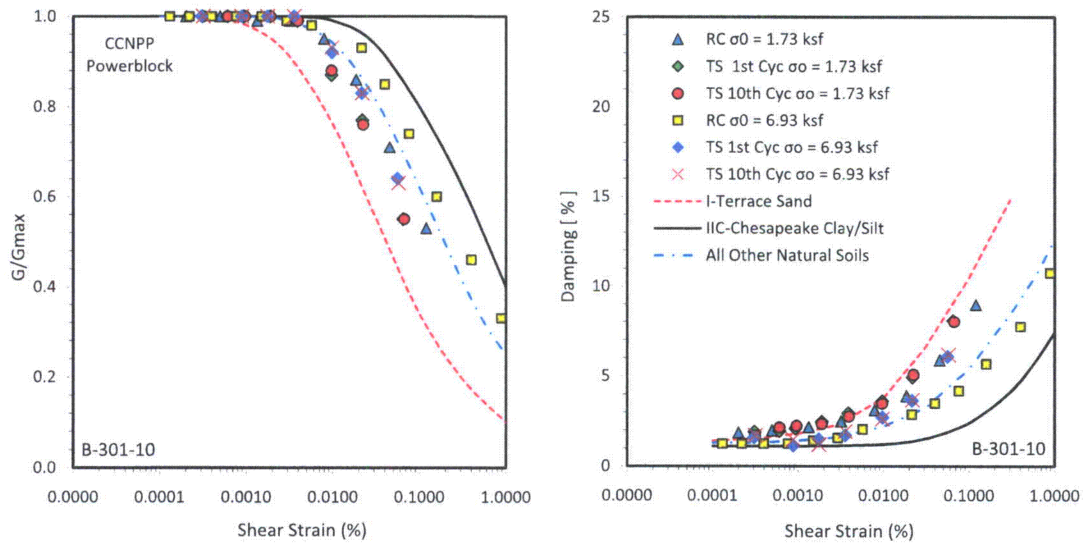


Figure 2.5-126—{RCTS Testing Sample B-437-6, Powerblock Area}



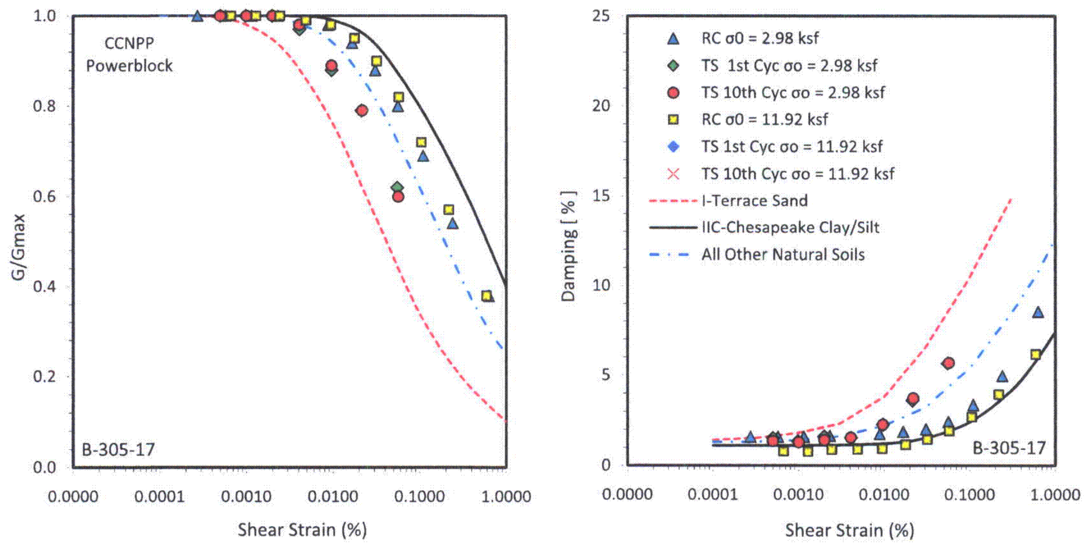
October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-127—{RCTS Testing Sample B-301-10, Powerblock Area}



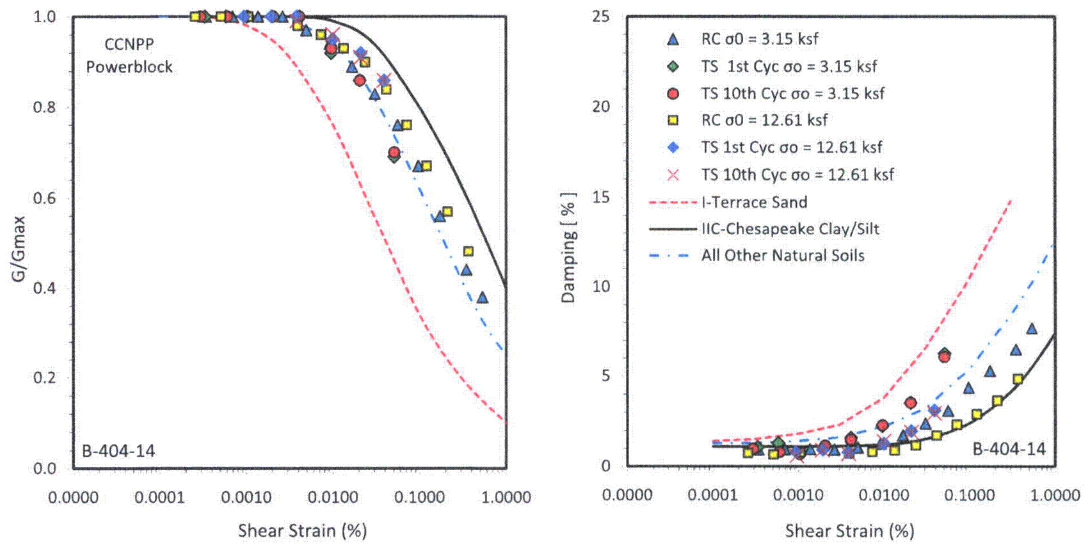
October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-128—{RCTS Testing Sample B-305-17, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-129—{RCTS Testing Sample B-404-14, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-130—{RCTS Testing Sample B-401-31, Powerblock Area}

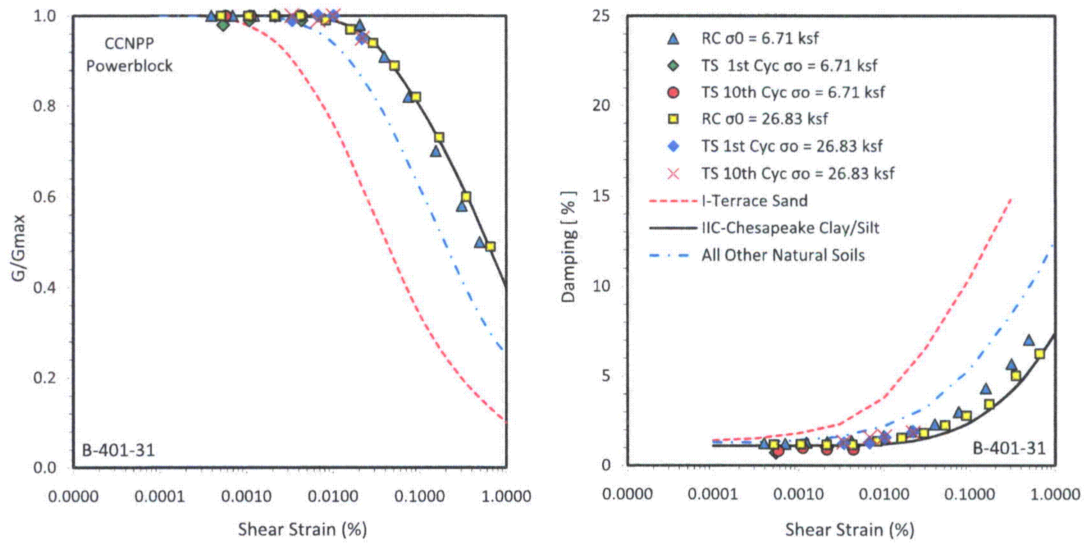
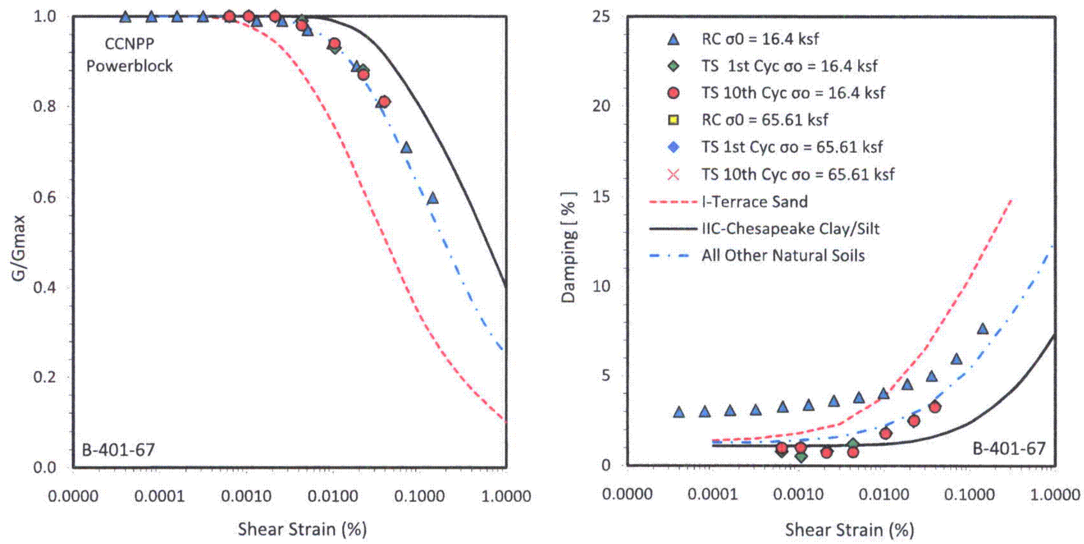
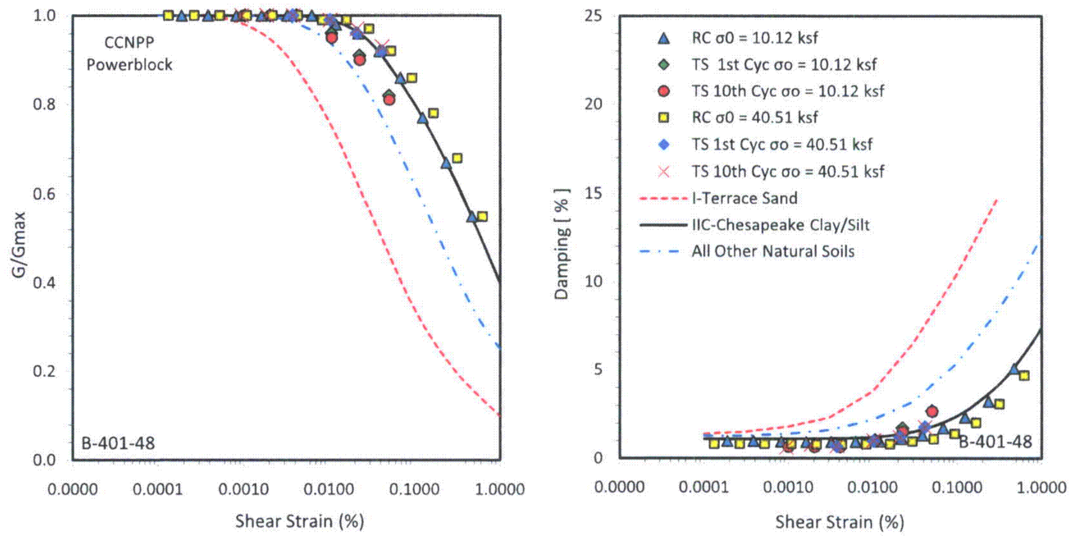


Figure 2.5-131—{RCTS Testing Sample B-401-67, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-132—{RCTS Testing Sample B-401-48, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-133—{RCTS Testing Sample B-301-78, Powerblock Area}

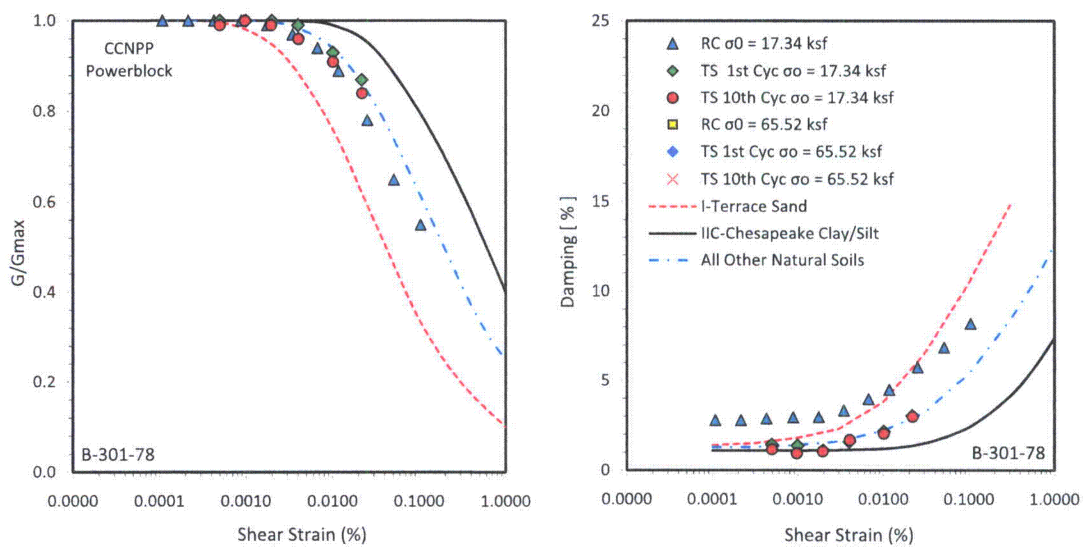
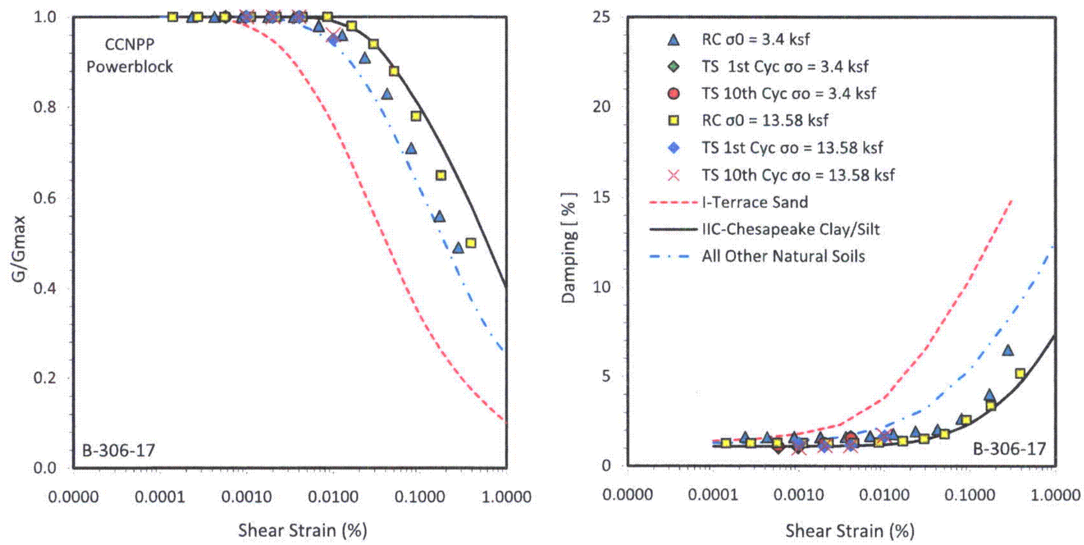
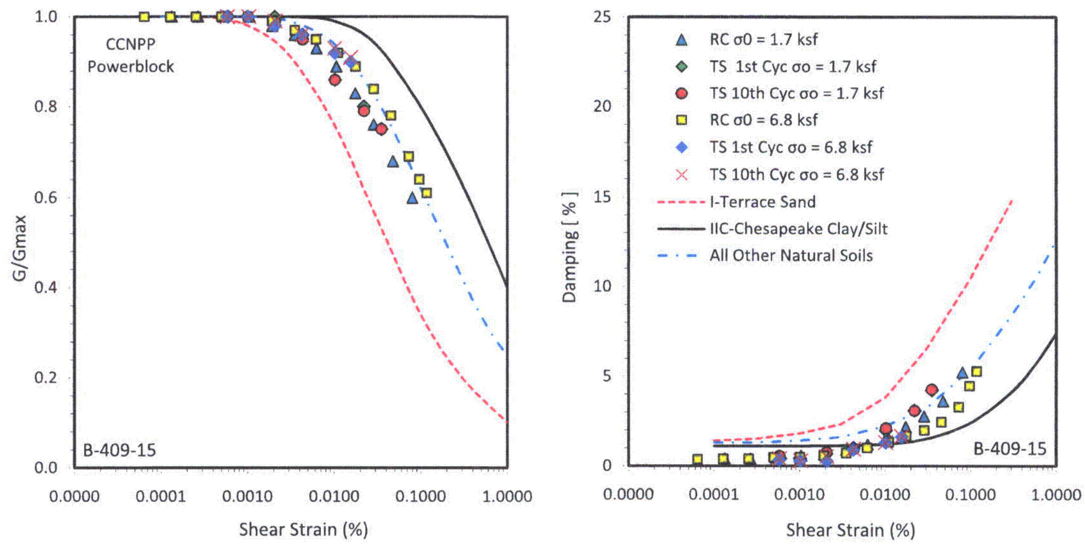


Figure 2.5-134—{RCTS Testing Sample B-306-17, Powerblock Area}



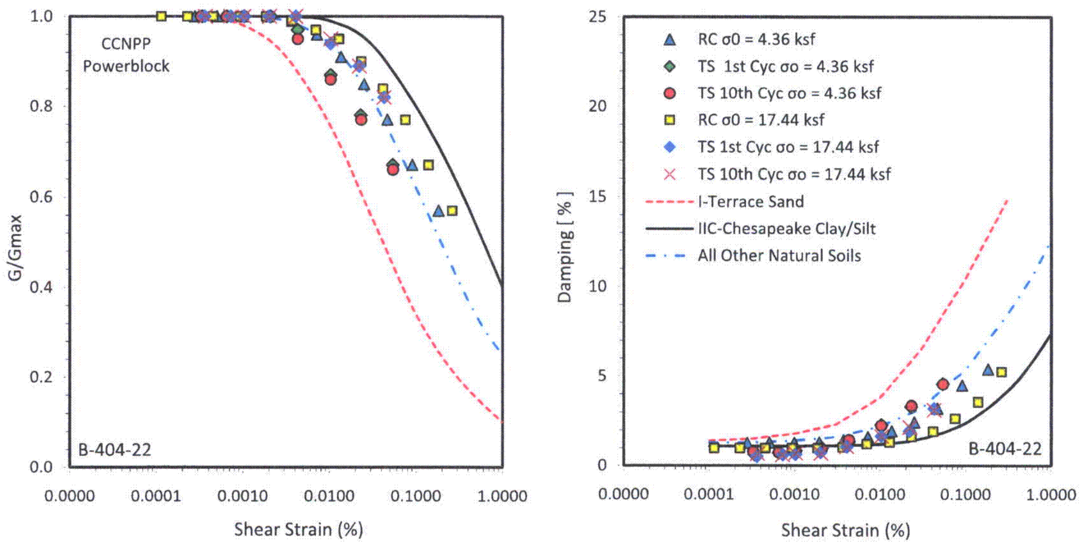
October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-135—{RCTS Testing Sample B-409-15, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-136—{RCTS Testing Sample B-404-22, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-137—{RCTS Testing Sample B-401-42, Powerblock Area}

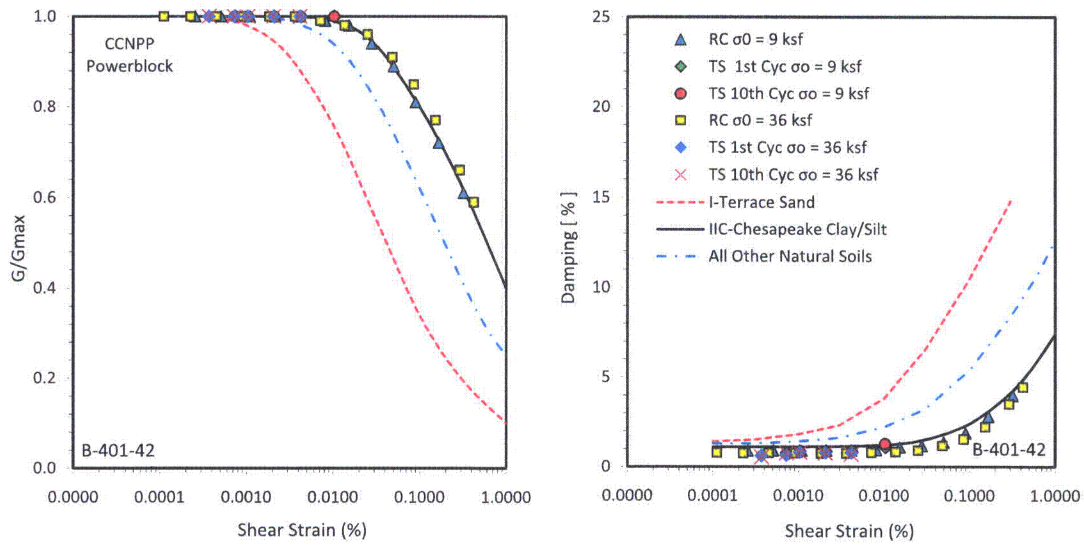
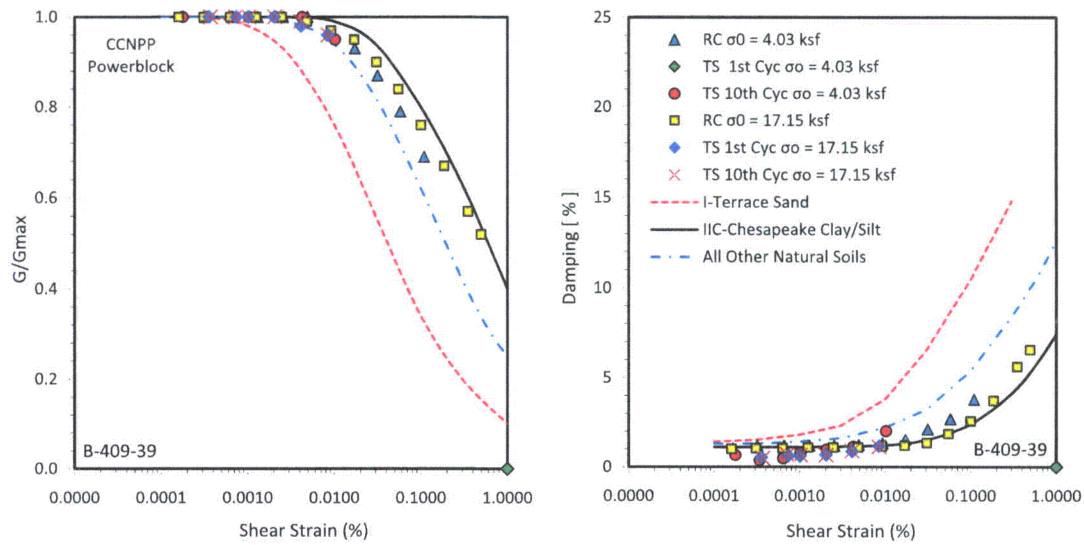
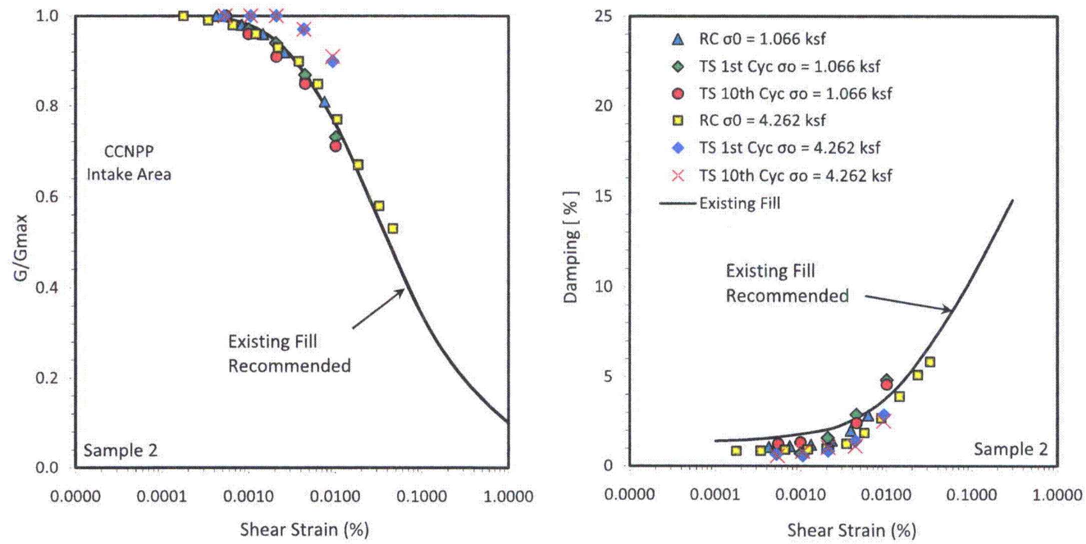


Figure 2.5-138—{RCTS Testing Sample B-409-39, Powerblock Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-139—{RCTS Testing Sample B-773-2, Intake Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.

Figure 2.5-140—{RCTS Testing Sample B-773-3, Intake Area}

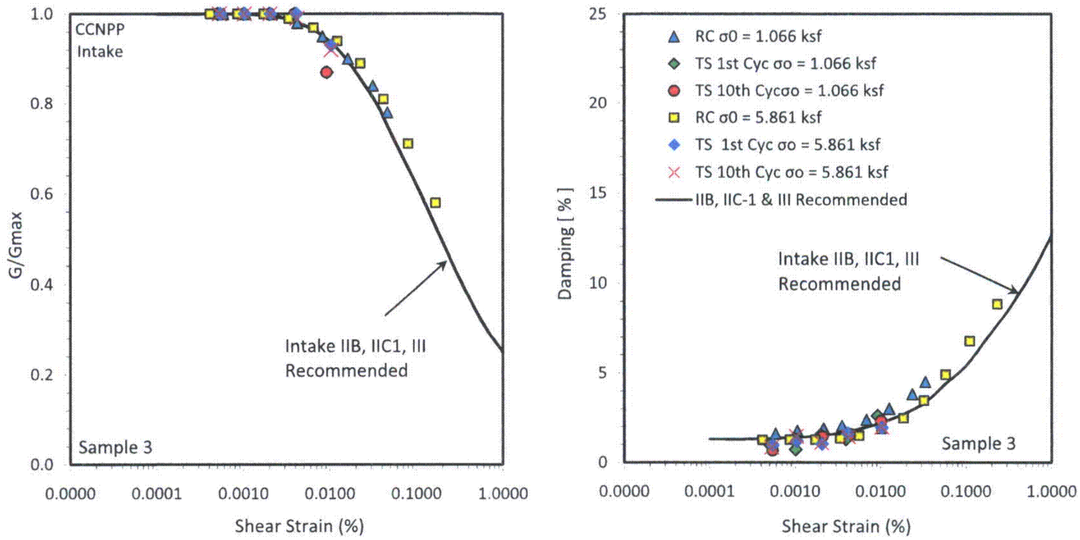


Figure 2.5-141—{RCTS Testing Sample B-773-4, Intake Area}

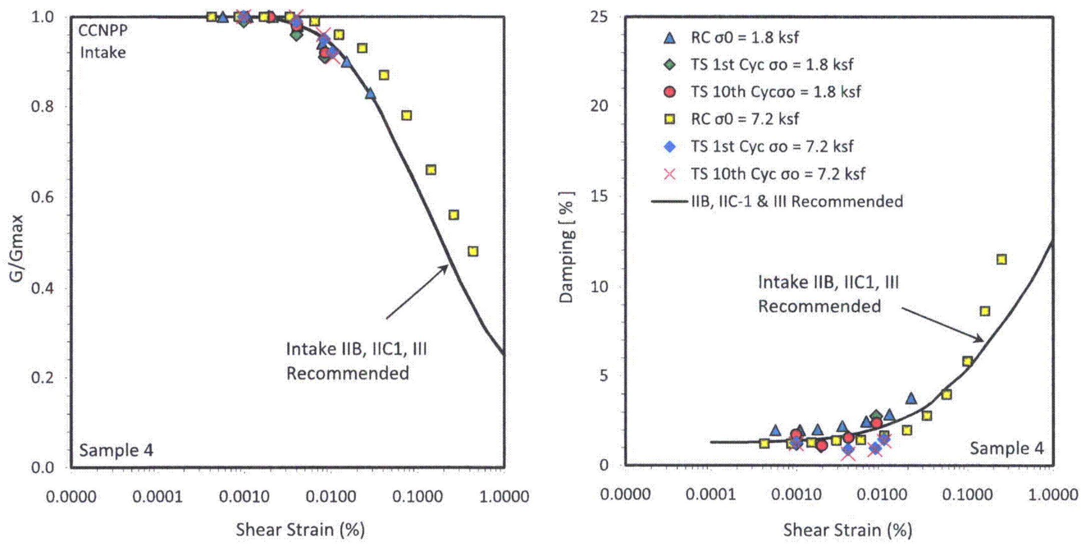
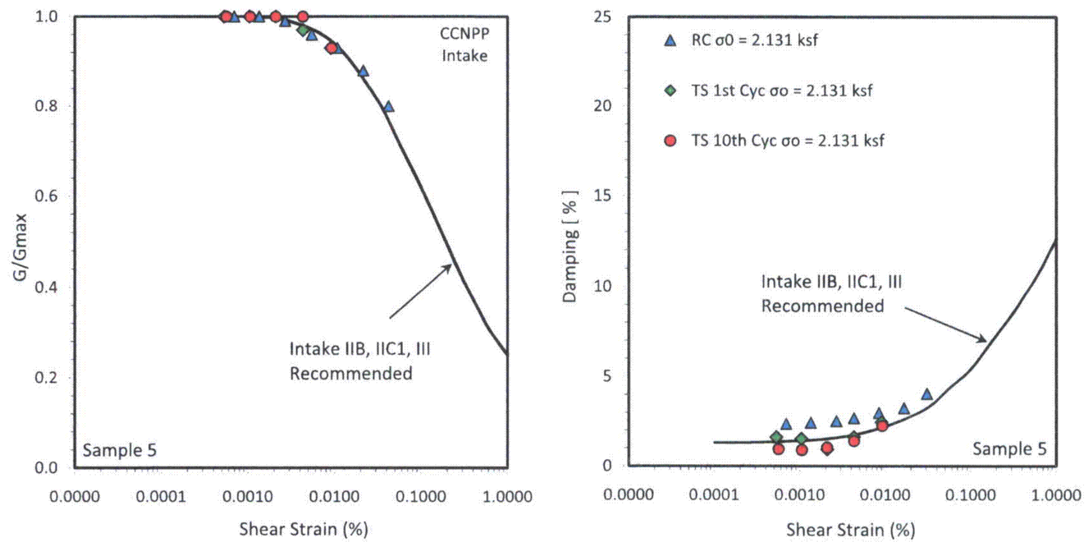


Figure 2.5-142—{RCTS Testing Sample B-773-5, Intake Area}



October 9, 2009 re-write of FSAR Sections 2.5.4 and 2.5.5.