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UniStar NUCLEAR

10 CFR 52.3

October 30, 2007

ATTN: Document Control Desk
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

UN#07-015

Subject: UniStar Nuclear, NRC Project No. 746
Response to NRC August 23 Letter Regarding Status of
Acceptance Review of Part 1 of the Combined License Application

References:

1) Letter UN#07-008, from R. M. Krich (UniStar) to U.S. Nuclear Regulatory Commission, "UniStar Nuclear, NRC Project No. 746, Submittal of a Partial Combined License Application for the Calvert Cliffs Nuclear Power Plant, Unit 3, Application for Withholding of Documents, and Request for Exemption," dated July 13, 2007

2) Letter from David B. Matthews (NRC) to R. M. Krich (UniStar), "Calvert Cliffs Nuclear Power Plant, Unit 3, Status of the Acceptance Review of Part One of the Combined License Application," dated August 23, 2007

UniStar Nuclear submitted a partial Combined License (i.e., COL) application to the NRC by letter dated July 13, 2007 (Reference 1) for a new nuclear power plant to be located at the current site of the Calvert Cliffs Nuclear Power Plant (CCNPP), Units 1 and 2. By letter dated August 23, 2007 (Reference 2), the NRC stated that it was nearing completion of its acceptance review of our partial COL application, but noted that certain issues in the application were not addressed to the level of detail expected by the NRC. As a result, the NRC requested in its August 23 letter that UniStar provide a plan for submitting sufficient information to address these issues. This letter provides our plan for addressing these issues and in some cases provides the proposed changes that address specific issues.

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The plan for addressing each of the eight issues transmitted by the NRC's August 23, 2007 letter and the proposed changes to the application to address certain issues are provided in the enclosure. As noted in the enclosure, the submitted partial COL application contained the information required by the applicable regulations or either met regulatory guidance, e.g., Regulatory Guide (RG) 1.206, "Combined License Applications for Nuclear Power Plants," dated June 2007, or proposed acceptable alternatives. We note also that relative to the issues in which questions were raised that involved information in the Design Certification (DC) application, the NRC has recently completed a detailed readiness assessment of the AREVA NP DC application including the safety analysis and probabilistic risk assessment sections. These two DC application sections are referred to in the enclosed response.

If you have any questions or need additional information, please contact me at (410) 470-5518.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on October 30, 2007

R. M. Krich
UniStar Nuclear Development, LLC

Enclosure:
Response to NRC August 23, 2007 Letter Regarding Status of Acceptance Review of Part 1 of the Combined License Application

cc: U.S. NRC Region I

ENCLOSURE

Response to NRC August 23, 2007 Letter Regarding Status of Acceptance Review of Part 1 of the Combined License Application

1. Final Safety Analysis Report (FSAR) Section 2.4.2, "Floods"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206, "Combined License Applications for Nuclear Power Plants." RG Section C.I.2.4.2 guidance indicates that a COL applicant should "describe the effects of local probable maximum precipitation (see Section C.I.2.4.3.1 of this guide) on adjacent drainage areas and site drainage systems, including drainage from the roof structures." Also the applicant should "provide sufficient details concerning the site drainage system to permit the following actions: (1) independent review of rainfall and runoff effects on safety-related facilities, (2) judgment concerning the adequacy of design criteria, and (3) independent review of the potential for blockage of site drainage as a result of ice, debris, or similar material."

In its application the applicant mentioned bio-retention ditches, overflow pipes, and culverts (in FSAR Chapter 2.4.2) but did not specify the locations and dimensions. Information must be submitted to provide sufficient details concerning the on-site drainage system to ensure the safety of the proposed structures, systems, and components from local flooding. The applicant has not submitted this information nor did it propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

UniStar Nuclear Response:

UniStar Nuclear has provided information on the location of bio-retention ditches and overflow pipes in three separate figures (FSAR Figures 2.4.2-1 to 2.4.2-3). The partial Combined License (i.e., COL) application also includes information related to the dimensions of the ditches, including discharge rates, minimum and maximum elevations, velocity, and Froude number, at 100 foot intervals along the ditches.

Importantly, in the analysis addressing the effects of local intense precipitation, UniStar Nuclear did not take credit for culverts or overflow pipes (i.e., these were assumed to be plugged), and further assumed that the ditches were full when the precipitation event began. UniStar Nuclear did assume that the ditches are used as a means of conveying water. In this regard, the analysis is conservative and bounding.

The information provided in the partial COL application constitutes the necessary input data for the hydrologic runoff analysis model used by UniStar Nuclear. Regulatory Guide 1.206 does not specify the model to be used by the COL applicant. If as a result of its technical review the NRC needs additional information to run its hydrologic models, that information would be provided in response to a Request for Additional Information.

Additional details on the dimensions of the bio-retention ditches will be provided in a revision to the Calvert Cliffs Nuclear Power Plant (CCNPP), Unit 3 partial COL application that will be submitted to the NRC by December 14, 2007. The associated pages of the CCNPP Unit 3 partial COL application, with the proposed changes identified, are attached.

The maximum water level due to local intense precipitation or the local probable maximum precipitation (PMP) is estimated and discussed in Section 2.4.2.3. The maximum water level in the CCNPP Unit 3 power block area, due to a local PMP, is at Elevation 81.5 ft (24.8 m). This water level becomes the design basis flood elevation for all safety-related facilities in the power block area. All safety-related building entrances in the power block are located above this elevation. The effects of local intense precipitation at the UHS makeup water intake are not estimated since the design basis flood elevation from the PMH will completely submerge this area.}

2.4.2.3 Effects of Local Intense Precipitation

{The design basis for the local intense precipitation is the fall season 1 square mile or point PMP as obtained from the U.S. National Weather Service (NWS) Hydro-meteorological Report Number 52 (NOAA, 1982). Table 2.4.3-1 presents the 1 square mile PMP for various durations at the CCNPP site.

As described in Section 2.4.1, CCNPP Unit 3 is located adjacent to the existing CCNPP Units 1 and 2. The site layout and drainage system are shown in Figure 2.4.2-1. The site grade completely fills in the upper reaches of the two unnamed branches (Branch 1 and Branch 2) shown on Figure 2.4.1-1 such that the streams will now begin just east of the CCNPP Unit 3 plant boundary area. Additionally, the drainage area for these streams, at the headwater, consists of only the CCNPP Unit 3 power block area. Since the power block area is at a much higher elevation than the existing streams, flood flows in these streams will not affect the CCNPP Unit 3 power block area. Thus, local PMP analysis on these two streams was not performed.

As indicated on Figure 2.4.2-1, the containment, fuel and safeguards buildings are located in the center and along the high point of the CCNPP Unit 3 power block area. From the high point, site grading falls at a 1% slope to bio-retention drainage ditches located along the northern and southern edges of the CCNPP Unit 3 area. There are four bio-retention ditches which drain the power block and the Turbine Building areas. Three of them run in the east-west direction; one north of CCNPP Unit 3, (North Ditch), one south of CCNPP Unit 3 and between CCNPP Unit 3 and the area reserved for equipment laydown (Center Ditch) and one south of the equipment laydown area (South Ditch). The fourth ditch (East Ditch) is located along the eastern edge of CCNPP Unit 3 and the equipment laydown area. It collect flows from the other three ditches. The East Ditch is divided in two, to allow passage of the CCNPP Unit 3 security fence. Flows in the South Ditch and the southern half of the East Ditch do not have an impact on the PMP flood levels in CCNPP Unit 3 and are not discussed in this section. The dimensions of the center, north, and east bio-retention ditches are provided in Table 2.4.2-6.

The bio-retention ditches are constructed with base materials that promote infiltration of runoff from low intensity rainfall events. However, for large storms, the infiltration capacity of the base materials would be exceeded and overflow pipes are provided to direct the runoff to the stormwater basin located to the east of the CCNPP Unit 3 power block. For the assessment of the local PMF levels, the overflow pipes and culverts in the drainage system are assumed to be clogged as a result of ice or debris blockage. In that case, PMP storm runoff from the area collected in the North and East Ditches would overflow along the northern and eastern edges (top of berm at Elevation 79 ft (24.1 m)), spilling out to the areas north and east of the CCNPP Unit 3 power block down the bluff to Chesapeake Bay. Channels and diversion walls will be provided on the north side of the site to direct North Ditch overflows to the east and eventually to the Chesapeake Bay. Flows from the Center Ditch will discharge into the East Ditch before overflowing the eastern edge of the East Ditch.

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Table 2.4.2-6 Bio-Retention Ditch Dimensions
(Page 1 of 1)

Ditch	Invert Elevation (ft, NGVD 29)	Top of Ditch Elevation (ft, NGVD 29)		Side Slopes	Bottom Width (ft)
		Left Bank	Right Bank		
Center	76.0	79.0	80.4	3:1	47.0
North	76.0	79.0	79.0	3:1	37.0
East	74.0	79.0	79.0	3:1	25.0

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2. FSAR Section 2.4.10, "Flooding Protection Requirements"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206, "Combined License Applications for Nuclear Power Plants." RG Section C.I.2.4.10 guidance indicates that a COL applicant should ". . . describe the static and dynamic consequences of all types of flooding on each pertinent safety-related facility. It should present the design bases required to ensure the safety-related facilities will be capable of surviving all design flood conditions, and reference appropriate discussions in other section of the FSAR where the design bases are implemented. The applicant referred to but did not provide the information to describe the forces and design basis loadings required for flood protections of the intake structures. The staff recognizes that this information is usually addressed in FSAR Section 3.8 but this information is material to the staff's review of FSAR Section 2.4.10.

Information must be submitted to provide sufficient details concerning flood protection requirements. The applicant has not submitted this information nor did it propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

UniStar Nuclear Response:

UniStar Nuclear has provided the information required by 10 CFR 2.101(a)(5) and 10 CFR 50.34(a)(1). Final Safety Analysis Report (FSAR) Section 3.8 of the COL application is not required to be submitted for a partial COL application under these sections.

Regulatory Guide 1.206, Section C.I.2.4.10, permits a COL applicant to "reference" appropriate discussions in other sections of the FSAR. On FSAR page 2.4.10-2 of the COL application, UniStar Nuclear specifically highlights the fact that the detailed description of the forces and design basis loadings required for flood protection of the intake structures will be found in Section 3.8 of the FSAR.

The NRC acknowledges that "the information is usually addressed in FSAR Section 3.8" but goes on to state that "this information is material to the staff's review of FSAR Section 2.4.10." While UniStar Nuclear acknowledges that FSAR Section 3.8 may be relevant to the NRC technical review of FSAR Section 2.4.10, we note that the information provided in our partial COL application complies with the applicable regulations and therefore should be sufficient for the NRC to complete its acceptance review of the partial application. FSAR Section 3.8 will be included with the remainder of the COL application that will be submitted to the NRC.

3. FSAR Section 2.5.2, "Vibratory Ground Motion"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206, "Combined License Applications for Nuclear Power Plants," which references RG 1.208, "A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion." RG 1.208, Appendix C, Section 2.5.1, states that ". . . a laboratory testing program should be carried out to identify and classify the subsurface soils and rocks and to determine their physical and engineering properties. Laboratory tests for both static and dynamic properties (e.g., shear modulus, damping, liquefaction resistance, etc) are generally required. The dynamic property tests should include cyclic triaxial tests, cyclic simple shear tests, cyclic torsional shear tests, and resonant column tests (RCTS), as appropriate." Furthermore the RG documents that, "sufficient laboratory test data should be obtained to allow for reasonable assessments of mean values of soil properties and their potential variability."

The applicant did not provide adequate test data for defining response of site-specific soil and rock materials to dynamic loading for safe shutdown earthquake (SSE) determination in FSAR Section 2.5.2. In FSAR Section 2.5.4.2.1.7, "Laboratory Testing Program" (pg 2.5.4-22), the applicant stated that RCTS testing results are expected to become available at a later date. Therefore, the applicant has not submitted this information nor did it propose an acceptable alternative. Consequently, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

4. FSAR Section 2.5.4, "Stability of Subsurface Materials and Foundations"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206. RG 1.206, Section C.I.2.5.4.7 states that an applicant should ". . . provide a description of the response of soil and rock to dynamic loading, including the following considerations: . . . (3) results of dynamic tests in the laboratory on samples of soil and rock to determine the shear modulus and damping degradation with strain; (4) results of soil-structure interaction analysis." The shear modulus reduction and damping ratio curves, which are vital to the staff's eventual of the site seismic response and SSI analyses, can only be confirmed and validated by laboratory testing. The applicant has not provided this testing data nor did it propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

UniStar Nuclear Response:

The NRC Staff correctly notes that UniStar Nuclear did not provide the results of laboratory test data from Resonant Column and Torsional Shear (RCTS) soils testing. The NRC is aware that there was limited laboratory capacity available to conduct RCTS testing and that all RCTS testing must be funneled through, and approved by, a single individual. Thus, this issue is a generic industry concern.

In view of these circumstances, UniStar Nuclear proposed an alternative approach in its partial COL application. Specifically, FSAR Section 2.5.4.7 of the partial COL application states that the applicant relied on shear modulus degradation and damping ratio curves from the available literature obtained based on generic Electric Power Research Institute (EPRI) curves and the

types of soils present at the CCNPP site. The partial COL application includes a detailed discussion of how the literature values were selected and used in the analysis.

UniStar Nuclear also stated that once the RCTS data are available, the data will be compared to the literature values used in the partial COL application to verify that they meet project requirements. If the laboratory testing results are substantially different, UniStar Nuclear has committed to adopting a revised set of data and repeating the calculations. Dynamic testing, consisting of RCTS testing, to obtain data on shear modulus and damping characteristics of the CCNPP site soils, is nearing completion. A total of 13 soil samples, from depths of about 15 feet to about 400 feet below the existing ground surface, have been assigned for RCTS testing. The results of the RCTS testing of the 13 samples will be reflected in a revision to FSAR Section 2.5.4 of the CCNPP Unit 3 partial COL application. This revision to FSAR Section 2.5.4 will be submitted to the NRC by December 14, 2007.

5. FSAR Section 2.5.4, "Stability of Subsurface Materials and Foundations"
To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206, "Combined License Applications for Nuclear Power Plants." RG Section C.I.2.5.4.5 guidance indicates that a COL applicant should ". . . discuss the following data concerning excavation, backfill and earthwork analysis at the site: (1) Sources and quantities of backfill and borrow; (2) Extent (horizontally and vertically) of all seismic Category I excavations, fills, and slopes; (3) Compaction specifications and embankment and foundation designs; (4) Dewatering and excavation methods and control of ground water during excavation to preclude degradation of foundation materials." The applicant has not submitted this information nor did it propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

UniStar Nuclear Response:

UniStar Nuclear has provided information regarding the stability of subsurface materials and foundations based on the information that is available from test pits, boreholes, and associated laboratory testing, including information that addresses each of the four categories of information mentioned in Regulatory Guide 1.206, Section C.I.2.5.4.5.

Recognizing that more detailed information in each of these areas will not be available until after detailed design begins, UniStar Nuclear proposed an alternative approach in its COL application that makes assumptions regarding materials, methods, criteria and procedures that will be used to verifying that actual conditions meet project requirements. More explicit commitments with respect to the alternative approach will be provided in a revision to the CCNPP Unit 3 partial COL application that will be submitted to the NRC by December 14, 2007. The associated pages of the CCNPP Unit 3 COL application, with the proposed changes identified, are attached.

borehole measurements, an average Poisson's ratio profile was estimated for the upper 400 ft, which is shown in Figure 2.5.4-25. The values obtained based on velocity measurements from the two deepest boreholes (B-301 and B-401) are also shown for comparison purposes.

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

2.5.4.4.2.2 CPT Seismic Measurements

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

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

2.5.4.4.2.3 Shear Wave Velocity Profile Selection

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

2.5.4.5 Excavation and Backfill

2.5.4.5.1 {Source and Quantity of Backfill and Borrow

A significant amount of earthwork is anticipated in order to establish the final site grade and to provide for the final embedment of the structures. It is 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, as shown in Figure 2.5.4-3. The maximum depth of the test pits was limited to 10 ft. Results of laboratory testing on the bulk samples collected from the test pits for moisture-density and other indices are summarized in Table 2.5.4-26, with the details included in Appendix 2.5-A. The results clearly indicate that there are both plastic and non-plastic soils included in Stratum I soils, including material designated as fill. These 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-

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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 shall be obtained from off-site borrow sources. The structural fill for CCNPP Unit 3 shall be sound, durable, well-graded sand or sand and gravel, with maximum 25 percent fines content, and free of organic matter, trash, and deleterious materials. Once the potential sources of structural fill have been identified, the material(s) are sampled and tested in the laboratory to establish their static and dynamic properties. Chemical tests are also performed on the candidate backfill materials. The results are evaluated to verify that the candidate backfill materials meet the design requirements for structural fill.

2.5.4.5.2 Extent of Excavations, Fills, and Slopes

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

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

Foundation excavations result in removing about 2 million cyd of materials. The extent of all excavations, backfilling, and slopes for Category I structures are shown in Figures 2.5.4-28 through 2.5.4-32. These sections are taken at locations identified in Figures 2.5.4-1 and 2.5.4-2. These figures illustrate that excavations for foundations of Category I structures will result in removing Stratum I Terrace Sand and Stratum IIa Chesapeake Clay/Silt in their entirety, and will extend to the top of Stratum IIb Chesapeake Cemented Sand, except in the UHS Makeup Water

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Intake Structure area. In the UHS Makeup Water Intake Structure 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 top of Stratum IIb in CCNPP Unit 3 and in CLA1 areas, as shown in Figure 2.5.4-33. This figure shows that the variation in top elevation of these soils is very little, approximately 4 ft or less (about 1 percent) 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 Cemented soils is confirmed, the excavations are backfilled with compacted structural fill to the foundation level of structures. 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 Sections 2.5.4.5.3.

Permanent excavation and fill slopes, created due to site grading, are addressed in Section 2.5.5. Temporary excavation slopes, such as those for foundation excavation, 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. These slopes are currently shown as 3:1 H:V in Figures 2.5.4-28 through 2.5.4-31.

Excavation for the Ultimate Heat Sink Makeup Water Intake Structure is different than that for other CCNPP Unit 3 structures, as shown in Figure 2.5.4-32. 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

Once structural fill sources are identified, as discussed in Section 2.5.4.5.1, several samples of the materials are obtained and tested for indices and engineering properties, including moisture-density relationships. 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, 2002c). 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 includes 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 technical specification also includes 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. The soil testing firm is required to be independent of the earthwork contractor and to have an approved quality program. A sufficient number of laboratory tests are required to be performed to ensure that variations in the fill material are accounted for. A trial fill program is normally conducted for the purposes of determining an optimum number of compactor coverages (passes), the maximum loose lift thickness, and other relevant data for optimum achievement of the specified moisture-density (compaction) criteria.

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2.5.4.5.4 Dewatering and Excavation Methods

Groundwater control is required during construction. Groundwater conditions and dewatering are addressed in Section 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 UHS Makeup Water Intake Structure area. Excavations in localized, intermittent cemented soils may require greater excavating effort, such as utilizing hoe-rams or other ripping tools; however, these zones are very limited in thickness, with probably only occasional need for expending additional efforts. Excavations for the CCNPP Unit 3 powerblock foundations 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 UHS 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 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

2.5.4.6.1 {Groundwater Conditions

The groundwater data collection and monitoring program is still in progress subsequent to the installation of observation wells during the CCNPP subsurface investigation. Details of available groundwater conditions at the site are given in Section 2.4.12. Based on available information through March 2007, the shallow (surficial) groundwater level in CCNPP Unit 3 and CLA1 areas ranges from approximately elevation 73 to elevation 85 ft, or an average elevation of 80 ft. This evaluation was used as the design groundwater elevation in the geotechnical calculations, as opposed to the design groundwater elevation of 73 ft as discussed in Section 2.4.12. The value used in the geotechnical calculations is bounded by the DCD value. Similarly, the groundwater level associated with the deeper hydrostatic surface was found to range from approximately elevation 34 ft to elevation 42 ft, with an average elevation of 39 ft. The shallow groundwater should have little to no impact on the stability of foundations, as the site grading and excavation plans will implement measures to divert these flows away from excavations, e.g., through runoff prevention measures and/or ditches. There are no Category I foundations planned within the

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upper water-bearing soils. The deeper groundwater condition, within the cemented sands, could adversely impact foundation soil stability during construction if not properly controlled, resulting in loss of density, bearing, and equipment trafficability.

2.5.4.6.2 Dewatering During Construction

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

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

2.5.4.6.3 Analysis and Interpretation of Seepage

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

2.5.4.6.4 Permeability Testing

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

2.5.4.6.5 History of Groundwater Fluctuations

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

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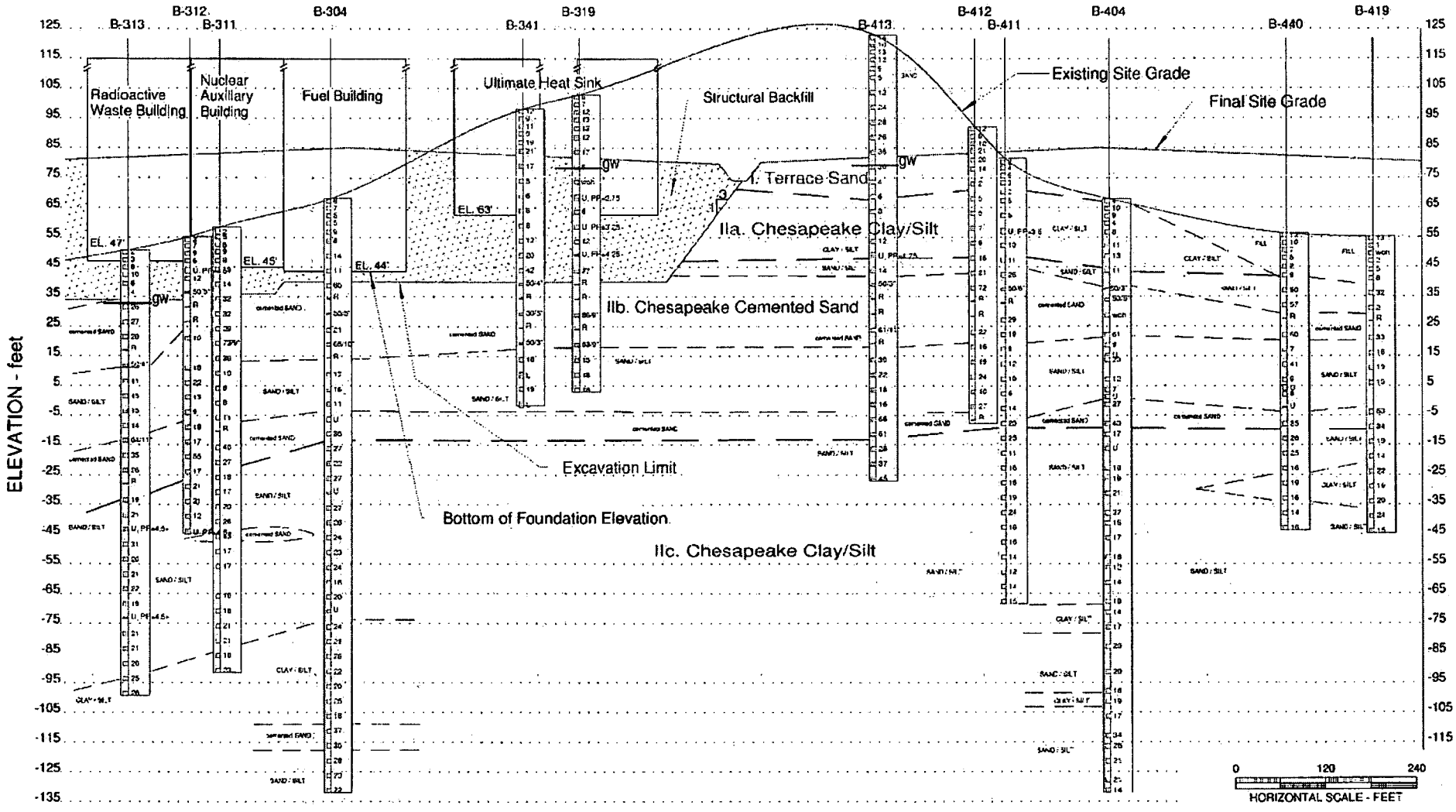

FIGURE 2.5.4-28 **Rev. 1**

EXCAVATION PROFILE NS-1

CCNPP UNIT 3 FSAR

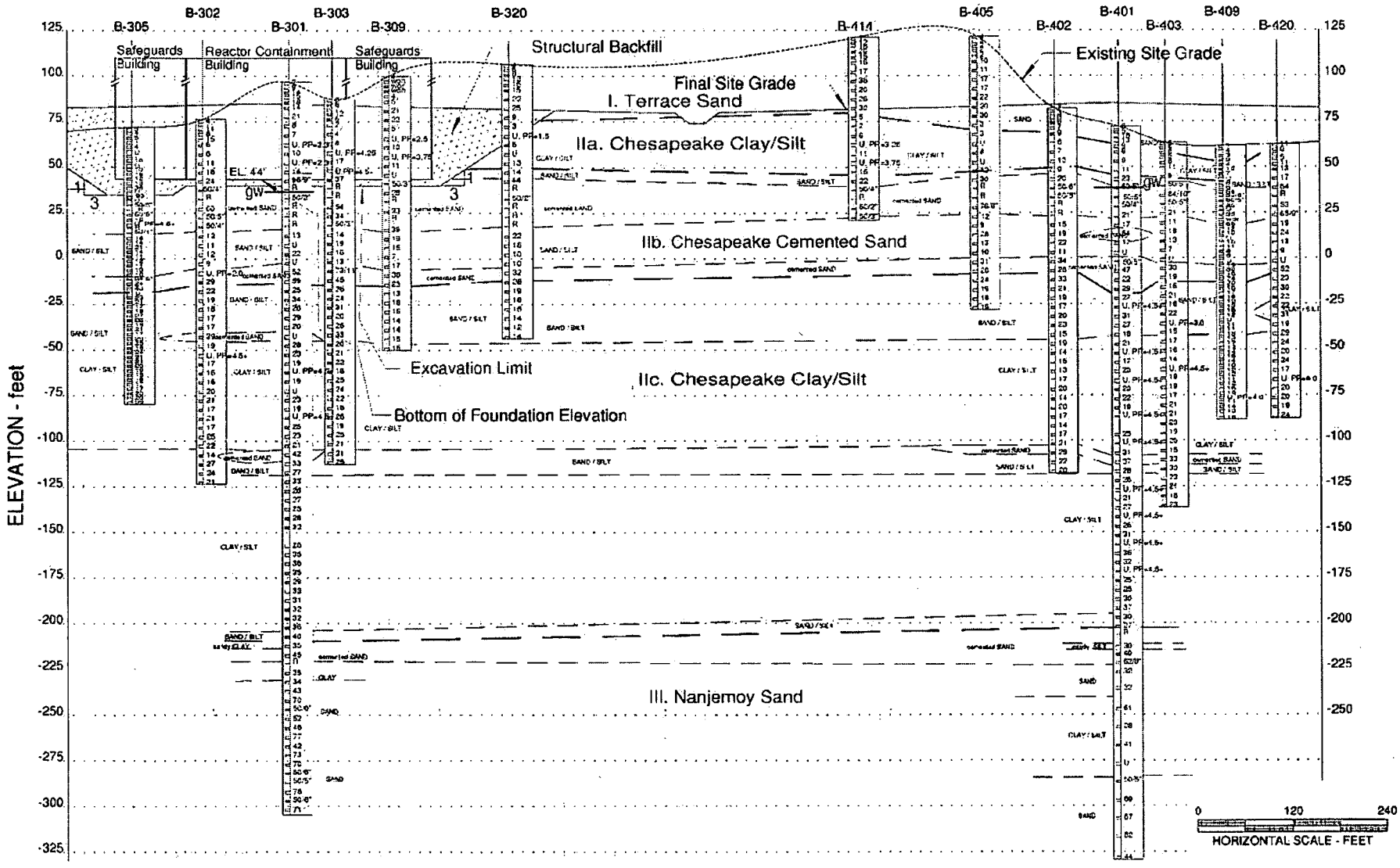

FIGURE 2.5.4-29 **Rev. 1**

EXCAVATION PROFILE NS-2

CCNPP UNIT 3 FSAR

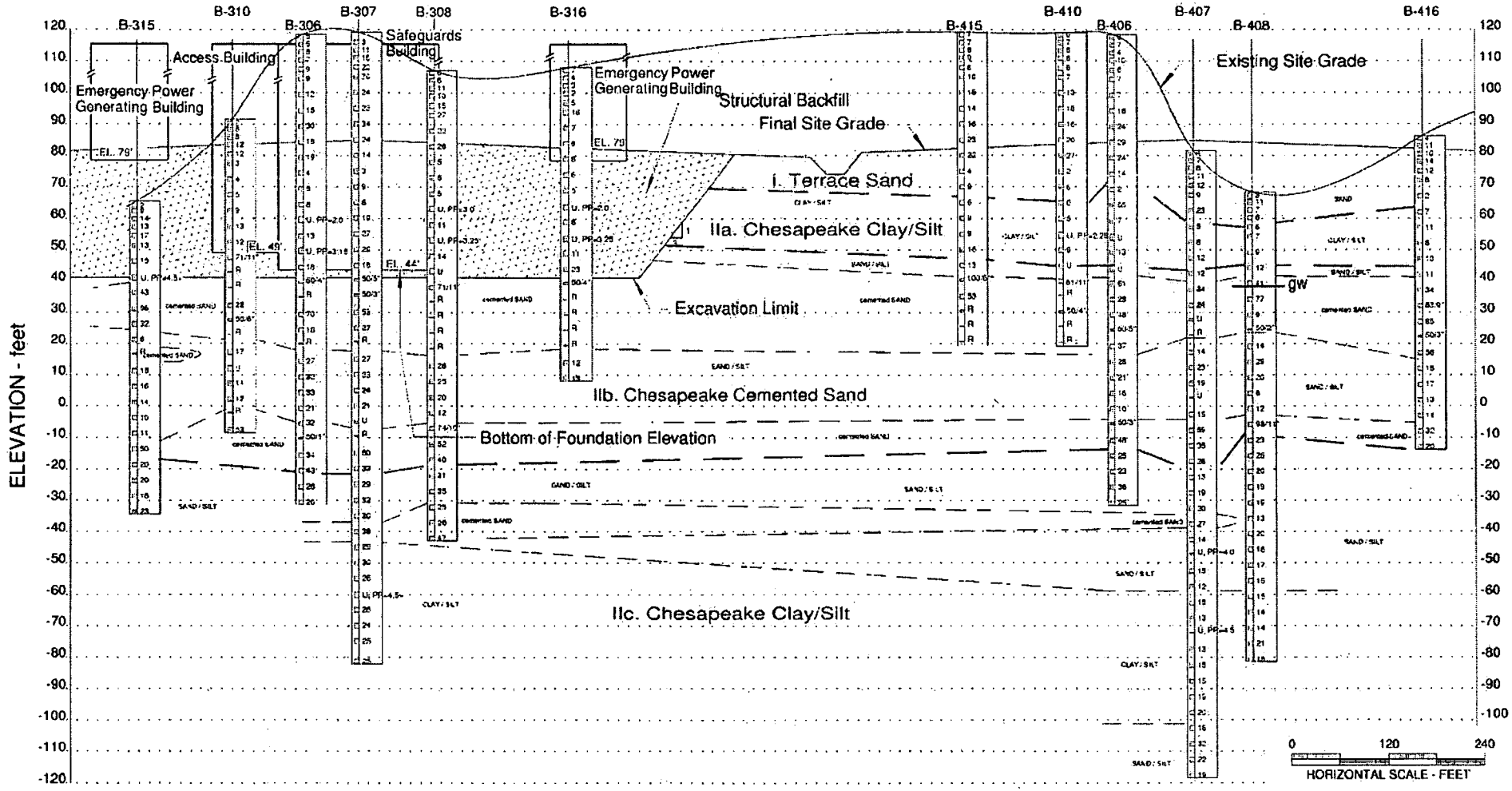

FIGURE 2.5.4-30 **Rev. 1**

EXCAVATION PROFILE NS-3

CCNPP UNIT 3 FSAR

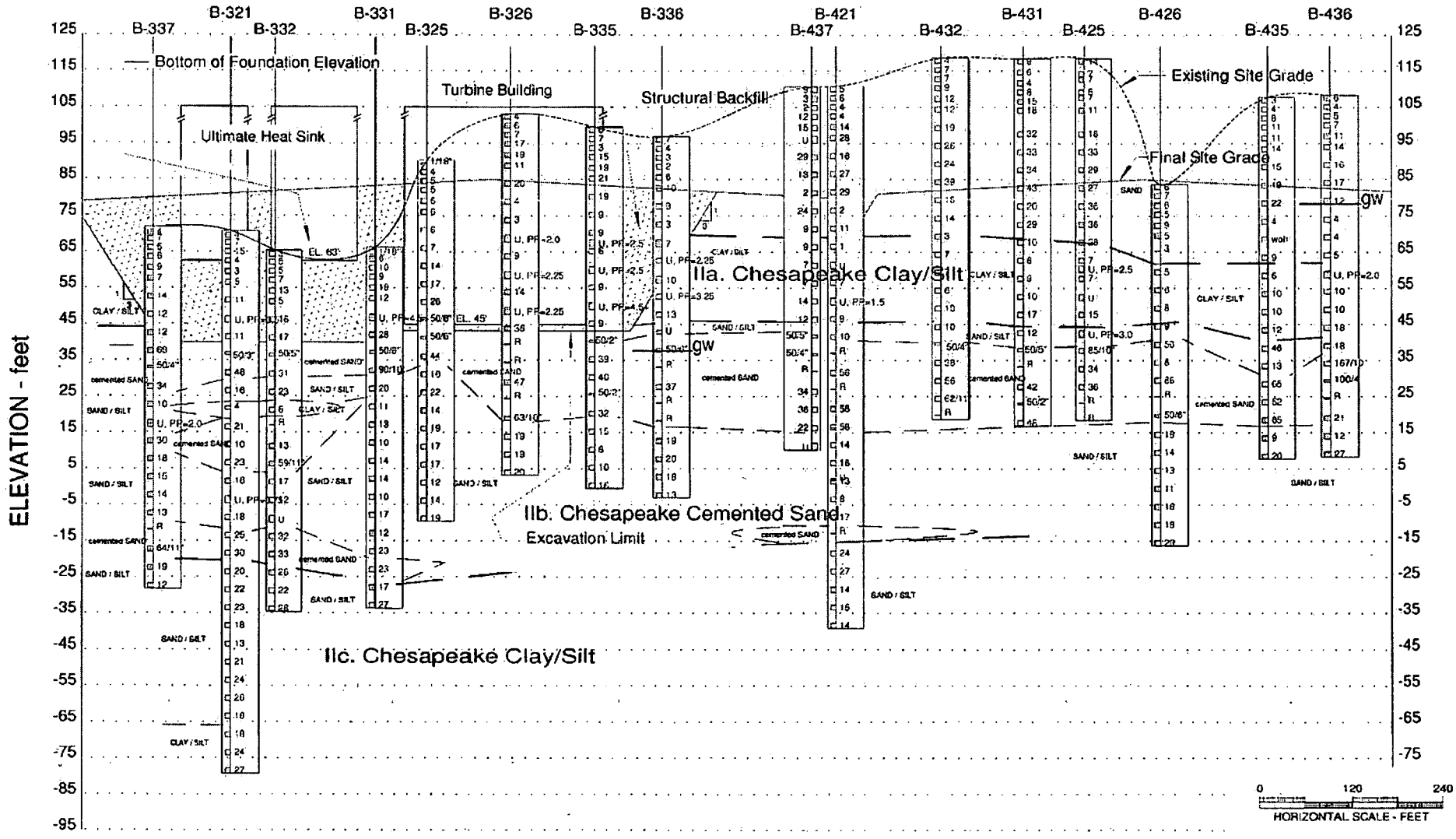

FIGURE 2.5.4-31 **Rev. 1**

EXCAVATION PROFILE NS-4

CCNPP UNIT 3 FSAR

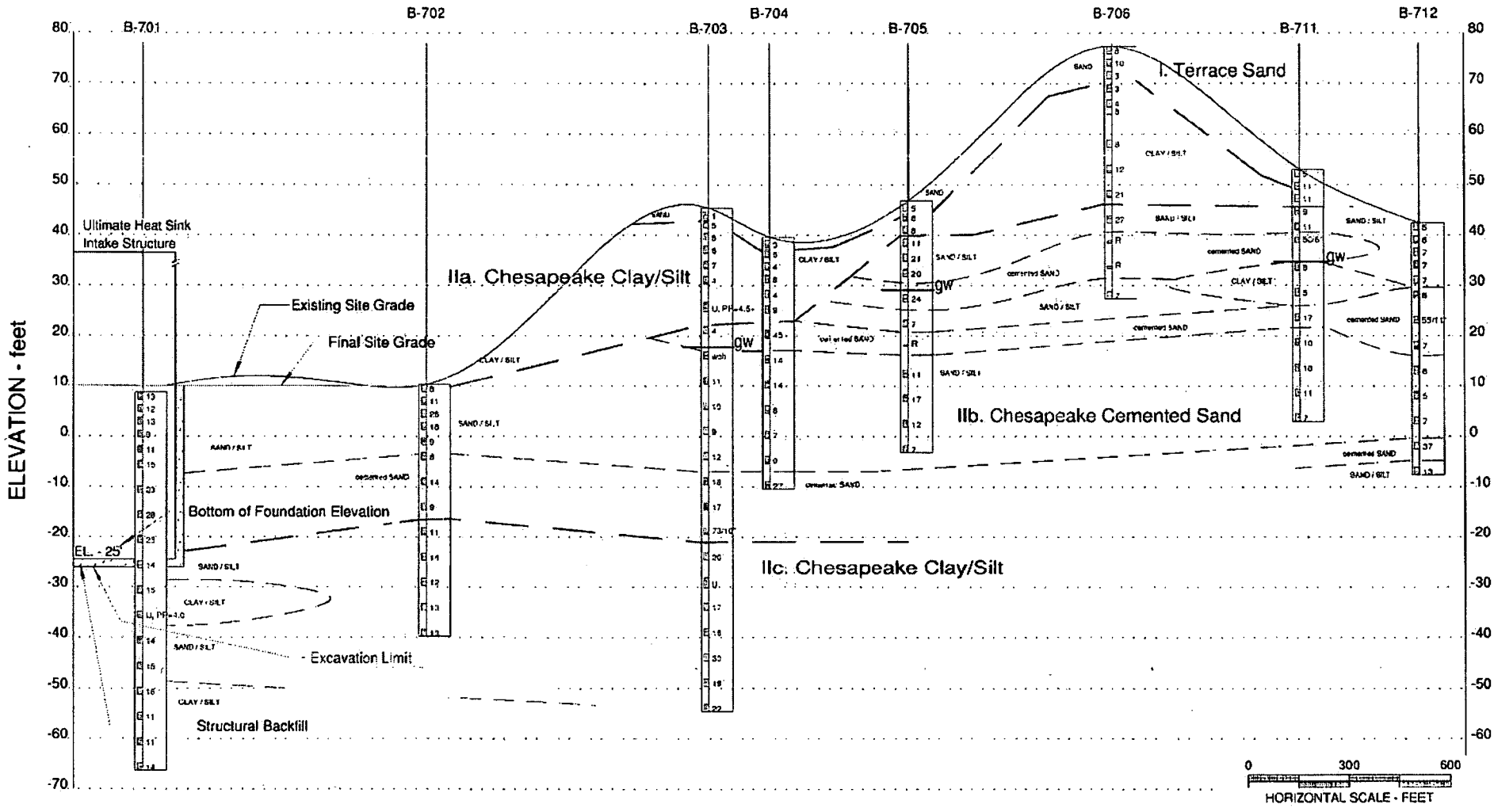

FIGURE 2.5.4-32 **Rev. 1**

EXCAVATION PROFILE IDP1

CCNPP UNIT 3 FSAR

6. FSAR Section 2.5.4, "Stability of Subsurface Materials and Foundations"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.34(a)(1), the staff uses RG 1.206, "Combined License Applications for Nuclear Power Plants." RG Section C.I.2.5.4.10 guidance indicates that a COL applicant should ". . . describe an analysis of the stability of all safety-related facilities for static loading conditions. Describe the analysis of foundation rebound, settlement, differential settlement, and bearing capacity under the dead loads of fills and plant facilities. Include a discussion and evaluation of lateral earth pressures and hydrostatic group water load acting on plant facilities." The applicant has not submitted this information nor did it propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1).

UniStar Nuclear Response:

UniStar Nuclear has provided information on foundation rebound, settlement, differential settlement, and bearing capacity under the dead loads of fills and plant facilities and did include a discussion and evaluation of lateral earth pressures and hydrostatic group water load acting on plant facilities.

Recognizing that more detailed information will only be available at the detailed design stage, UniStar Nuclear proposed alternative approaches in its partial COL application that make assumptions regarding materials, methods, criteria and procedures that will be used to verify that actual conditions meet project requirements. More explicit commitments with respect to the alternative approach will be provided in a revision to the CCNPP Unit 3 COL application that will be submitted to the NRC by December 14, 2007. The associated pages of the CCNPP Unit 3 partial COL application, with the proposed changes identified, are attached.

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

Under "Factor of Safety Against Liquefaction," NRC Regulatory Guide 1.198 (NRC, 2003c) indicates that FOS≤1.1 is considered low, FOS≈1.1 to 1.4 is considered moderate, and FOS ≥ 1.4 is considered high. A FOS=1.1 appears to be the lowest acceptable value. On the same issue, the Committee on Earthquake Engineering of the National Research Council (CEE, 1985) states that "There is no general agreement on the appropriate margin (factor) of safety, primarily because the degree of conservatism thought desirable at this point depends upon the extent of the conservatism already introduced in assigning the design earthquake. If the design earthquake ground motion is regarded as reasonable, a safety factor of 1.33 to 1.35 ... is suggested as adequate. However, when the design ground motion is excessively conservative, engineers are content with a safety factor only slightly in excess of unity." This, and a minimum FOS=1.1 in NRC Regulatory Guide 1.198 (NRC, 2003c), are consistent with the FOS=1.1 adopted for the assessment of FOSs for the CCNPP Unit 3 site soils, considering the conservatism adopted in ignoring the cementation, age, and overconsolidation of the deposits, as well as the seismic acceleration and magnitude levels. Such level of conservatism in the evaluation, in conjunction with ignoring the geologic factors discussed above, justifies the use of FOS=1.1 for liquefaction assessment of the CCNPP site soils.}

2.5.4.9 Earthquake Design Basis

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

2.5.4.10 Static Stability

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

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

* below respective final site grade

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

Construction of the Reactor basemat requires an excavation of about 41 ft (from approximately elevation 85 ft). The resulting rebound (heave) in the ground due to the removal of the soils is expected to primarily take place in Stratum IIc Chesapeake Clay/Silt soils. A rebound of about 2 in. is estimated due to excavation for the Reactor basemat, and is expected to take place concurrent with the excavation. Ground rebound is monitored during excavation. The heave estimate was made based on the elastic properties of the CCNPP site soils and the response to the unloading of the ground by about 41 ft of 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. The excavation shall remain open for a period sufficiently long such that ground heave fully develops.

2.5.4.10.1 Bearing Capacity

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

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

This COL item is addressed in the following sections.

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2.5.4.10.1.1 Bearing Condition of Units 1 and 2 Soils

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

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

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

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

2.5.4.10.1.2 Bearing Capacity of CCNPP Unit 3 Structures

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

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

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

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

γ' = effective unit weight of soil,

D_f = depth from ground surface to base of foundation,

B = width of foundation,

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

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

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

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

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

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

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

H = depth to the lower layer

The factors in Eqs. 2.5.4-18A and 2.5.4-18B, are defined as follows:

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

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

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

The subsurface conditions and material properties were described in Section 2.5.4.2. Material properties, conservatively designated for the various strata, were used for foundation evaluation, as shown in Table 2.5.4-12. The specific parameter values used in the bearing capacity evaluations are provided in Table 2.5.4-30. The following bounding property values for compacted fill were used in the analyses: a unit weight of 120 pcf, an angle of internal friction of 32 degrees, and a modulus of elasticity of 500 tsf. Compacted fill is verified to meet these design requirements during construction. Location of structures, relative to the subsurface conditions, are shown in Figures 2.5.4-28 through 2.5.4-32. An average groundwater level at elevation 80 ft was used for foundation evaluation. For the case of the UHS Makeup Water Intake Structure where the ground surface was below elevation 80 ft, the groundwater elevation was considered to be at the ground surface.

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

Category I Structure	Allowable Bearing Pressure (Layered System) (ksf)	Lower-Bound Allowable Bearing Pressure (ksf)	Recommended Max. Bearing Pressure (ksf)
Essential Service Water System (ESWS) Cooling Tower (UHS)	13 - 14	8.0	13
Emergency Power Generating Building (EDGB)	14 - 15	7.8	13
Common Basemat	24	8.3	20
UHS Makeup Water Intake Structure	---	8.0	8

Design values of foundation pressures for the Category I structures were estimated based on project knowledge and typical loading for similar structures. The design values were adopted for comparison with the allowable values above and are as follows.

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

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

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

2.5.4.10.2 Settlement

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

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

where, δ = settlement

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

p_i = vertical applied pressure at center of layer i

h_i = thickness of layer i

Ei = elastic modulus of layer i

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

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

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

Settlement estimates were made following the preceding relationships and using available soils properties given in Table 2.5.4-12. To estimate settlement values, a subsurface profile in the foundation area of interest was adopted, as shown in Figures 2.5.4-28 through 2.5.4-32. The soil layers were further subdivided into sublayers for refined estimates. From the stress distribution in Eq. 2.5.4-20, sublayer thickness, and elastic modulus for the particular soil, values for settlement were estimated using Eq. 2.5.4-19. The final settlement is the sum of the estimated values for all of the sublayers combined. Significant to estimating settlement values is the value of elastic modulus, E. This parameter was selected from the available summary of soil engineering properties, as shown in Table 2.5.4-12, complimented with estimates of elastic moduli, reduced for strain magnitude, based on the average shear wave velocity values shown in Table 2.5.4-12. Settlement estimates were made for all Category I structures, for the estimated design foundation pressures given in this subsection. They are as follows.

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

The settlement magnitudes are discussed later.

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

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

(1) Refer to Figure 2.5.4-2 for locations

Estimated settlements corresponding to the net foundation pressures are given below. It is noted, however, that the magnitude of estimated settlements are generally not significantly changed, given the typically small change in foundation pressures.

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

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

From the above estimates, average foundation settlement for the UHS Makeup Water Intake Structure is within the acceptable range of 2.5 in. to 4 in. Similarly average settlement estimates for the Emergency Power Generating Building and the ESWS Cooling Towers are within the acceptable range of 2.5 in. to 4 in. For the Common Basemat, an average settlement of about 8 in. was estimated for the 15 ksf loading. This estimated total settlement is largely the result of the extreme foundation size and loading as well as the depth of influence of the large mat.

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

Foundation settlements largely take place concurrent with construction; therefore, a majority (i.e., more than half) of the settlements will have taken place prior to placing the equipment, piping, and the final finishes. Hence, post-construction total and differential settlements are expected to be lower than the values noted herein, particularly after accounting for foundation mat rigidity.

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.

In general, the estimated foundation settlements are larger than those indicated for CCNPP Units 1 and 2, although no estimates or measured values are available for Units 1 and 2, as discussed in Section 2.5.4.10.1. The difference in settlement between the two areas is not due to differing soil conditions, as the soils are comparable. Rather, they are largely due to the difference in magnitude of net loading imposed by these structures on the soils, and foundation size. The influence of the larger and heavier Common Basemat for Unit 3 extends deeper, thereby influencing a larger volume of soils.

However, all foundations are designed to safely tolerate the anticipated total and differential settlements. Additionally, engineering measures are incorporated into design for control of

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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.

2.5.4.10.3 Earth Pressures

Static and seismic lateral earth pressures are addressed for plant below-ground walls. Seismic earth pressure diagrams are structure-specific and are, therefore, only addressed generically herein. Specific earth pressure diagrams are developed for specific structures based upon each structure's final configuration. Passive earth pressures are not addressed; they are ignored for conservatism for general purpose applications. The following soil properties were assumed for the backfill; an angle of shearing resistance of 30 degrees and a total unit weight of 120 pcf. Structural backfill material is verified to meet the design requirements prior to use during construction. A surcharge pressure of 500 psf was assumed as well. 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.

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

2.5.4.10.3.1 Static Lateral Earth Pressures

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

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

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

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

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

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

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

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

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

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

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

$$p_W = \gamma_W \cdot z_W \qquad \text{Eq. 2.5.4-25}$$

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

2.5.4.10.3.2 Seismic Lateral Earth Pressures

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

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

where, ΔK_{AE}=coefficient of active seismic earth pressure = K_{AE}-K_{AS}
K_{AE}=Mononobe-Okabe coefficient of active seismic earth thrust

Eq. 2.5.4-27

γ =unit weight of backfill at depth z
z=depth below the top of the backfill
H=below-grade height of wall

$$K_{AE}=\cos^2(\phi'-\theta)/\{\cos^2\theta \cdot [1+(\sin\phi' \sin(\phi'-\theta)/\cos(\theta))^{0.5}]^2\} \qquad \text{Eq. 2.5.4-27}$$

$$\theta=\tan^{-1}(k_h)$$

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

The at-rest seismic conditions are reported to be two times as large as the active earth pressures calculated by the Mononobe-Okabe equation (Whitman, 1991). Given that most below-grade walls actually yield to some extent, the actual "at rest" seismic pressures may not be as high as previously indicated (Whitman, 1991). Thus the "at rest" seismic earth pressures will be taken as twice the active values, or, $\Delta K_{0E} = 2\ \Delta K_{AE}$.

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, as given in Eq. 2.5.4-25. It is noted that seismic groundwater thrust greater than 35 percent of the hydrostatic thrust can develop for cases when k_h>0.3g (Whitman, 1990). Given the relatively low seismicity at the CCNPP Unit 3 site (k_h<0.3g), seismic groundwater considerations can be ignored.

2.5.4.10.3.3 Lateral Earth Pressures Due to Surcharge

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

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

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

2.5.4.10.3.4 Sample Earth Pressure Diagrams

Using the relationship outlined above and assumed backfill properties, sample earth pressures were estimated. Sample earth pressure diagrams are provided in Figures 2.5.4-55 and 2.5.4-56 for a wall height of 41 ft, level ground surface, and with groundwater level at 5 ft below the surface. The backfill is taken as granular soils, with ϕ'=30 degrees and γ=120 pcf. The horizontal ground acceleration is taken as 0.125g. A permanent uniform surcharge load of 500 psf is also included. The validity of assumptions regarding surcharge loads, backfill properties,

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and structural configurations is confirmed during the detailed design stage. Actual earth pressure evaluations are performed at that time for the design of below-grade walls, based on actual project conditions. The results of these earth pressure evaluations shall be included in an update to the FSAR at that time.

2.5.4.10.4 Selected Design Parameters

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

2.5.4.11 Design Criteria

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

2.5.4.12 Techniques to Improve Subsurface Conditions

{Major structures derive support from the very dense cemented soils or compacted structural backfill. Given the planned foundation depths and soil conditions at these depths, as shown in Figures 2.5.4-28 through 2.5.4-32, no special ground improvement measures are warranted. Ground improvement is limited to excavation of unsuitable soils, such as existing fill or loose/soft soils, and their replacement with structural backfill. 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 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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Table 2.5.4-12 Summary Average Soils Engineering Properties[1] (Page 1 of 1)

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

Notes.

(1) The values tabulated above are designated for the various strata. Reference should be made to specific boring and CPT logs and laboratory test results for appropriate modifications at specific locations and for specific calculations.

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

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Table 2.5.4-30 Bearing Capacity Evaluation Parameters
(Page 1 of 1)

Structure	Embedment, D (ft)	Length, L (ft)	Width, B (ft)	B/L	Soil Layer	c (ksf)	ϕ (deg)	N_c	N_q	N_γ	ζ_c	ζ_q	ζ_γ
ESWS Cooling Towers	13.7–22.7	147	96	0.65	Stratum II-b	0	34	42.16	29.44	41.06	1.46	1.44	0.74
					Stratum II-c	4	0	5.14	1	0	1.13	1	0.74
Emergency Power Generating Buildings	3 - 6	131	93	0.71	Stratum II-b	0	32	35.49	23.18	30.22	1.46	1.44	0.72
					Stratum II-c	4	0	5.14	1	0	1.14	1	0.72
Reactor (Common Basemat)	41	322	200	0.62	Stratum II-b	0	34	42.16	29.44	41.06	1.43	1.42	0.75
					Stratum II-c, III	2.3	16	11.63	4.34	3.06	1.23	1.18	0.75
UHS Makeup Water Intake Structure	30.5	78	47	0.60	Stratum II-c	4	0	5.14	1	0	1.12	1	---

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7. Information submitted pursuant to 10 CFR 50.34(a)(1)(ii)(D)

The information that must be submitted to address this requirement should include description and analysis that reflects the radiological consequences that might result from a postulated accident involving release of fission products. Attention should be given to a discussion about specific plant design features that may factor into this analysis such as the barriers that must be breached to release such fission products. Site characteristics must also be part of the evaluation. The applicant must perform an evaluation and analysis and provide that information in its application to allow the staff to independently evaluate the technical bases of the analyses, assumptions, and assessment leading to its conclusions. Although the application included limited site- and design-specific information with results of the analyses, the applicant did not provide the safety assessment containing an analysis and evaluation of the major structures, systems, and components of the facility that bear significantly on the acceptability of the site under the radiological consequence evaluation factors identified in 10 CFR 50.34(a)(1)(ii)(D).

With a complete COLA application, rather than one provided in two parts, the staff would expect this information to be provided in FSAR Chapter 15, "Accident and Transient Analysis." In the absence of a complete FSAR, the staff expects that some of the information listed in RG 1.206 Section C.I.15 should be included in the application and addressed to a sufficient level of detail. For example, RG 1.206 Section C.I.15.6.5, "Radiological Consequences," describes that the applicant should summarize the assumptions, parameters, and calculational methods used to determine the doses that result from accidents. Provide sufficient information to allow an independent analysis to be performed. Include all pertinent plant parameters that are required to calculate doses for the EAB [exclusion area boundary] and LPZ [low population zone], as well as those locations within the EAB where significant site-related activities may occur (e.g., the control room). While some of this information is not necessary to support the requirements of 10 CFR 50.34(a)(1)(ii)(D), some of it is necessary for example to provide sufficient detail for the staff to evaluate and perform an independent analysis of dose consequences at the EAB and LPZ.

The applicant did not provide the evaluation nor has it provided an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.34(a)(1)(ii)(D).

UniStar Nuclear Response:

UniStar Nuclear has provided the information required by 10 CFR 2.101(a)(5) and 10 CFR 50.34(a)(1). Specifically, UniStar Nuclear has provided the U.S. EPR Design Certification application doses that will result in the event of a release from the plant based on the actual US EPR source term and accident sequences (reflected in Section 7.2 of the CCNPP Unit 3 Environmental Report). Further, the analysis relies upon site parameters which bound the site-specific parameters for CCNPP Unit 3.

Based on the following information, we do not interpret 10 CFR 2.101(a)(5) and 10 CFR 50.34(a)(1) to require the level of detail that would normally be included in Chapter 15 of the FSAR.

The dose consequence criteria in 10 CFR 50.34(a)(1)(ii)(D), now 10 CFR 52.79(a)(1)(vi), were first incorporated into 10 CFR 50 (moved there from 10 CFR 100) in 1996 (61 FR

65157, dated December 11, 1996). According to the Statements of Consideration accompanying the final rule for this change, by moving the dose consequence limits into 10 CFR 50, the NRC effected a "partial decoupling of siting from accident source term and dose calculations" (61 FR 65159). Because it considers the exclusion area to be an essential feature of a reactor site, the NRC retained the requirement that the applicant "verify that [the] proposed exclusion area distance is adequate to assure that the radiological dose to an individual will be acceptably low in the event of a postulated accident" (61 FR 65159). (emphasis added). Similarly, for siting requirements related to the low population zone (LPZ), the NRC imposed a "limit" on the dose consequence at the outer boundary of the LPZ (61 FR 65161).

The dose limits in 10 CFR 50.34(a)(1)(ii)(D), now 10 CFR 52.79(a)(1)(vi), therefore reflect "go/no-go" criterion related primarily to the proposed reactor design, as influenced by site-specific characteristics (*i.e.*, χ/Qs). This is because the NRC "discourages" additional design features in a standardized design to compensate for otherwise poor site conditions.

The same framework is reflected in the NRC's guidance documents and Standard Review Plans as follows.

> NUREG-0800, Section 2.2.3 – Acceptance criteria for COL applications are based on meeting the requirements of 10 CFR 52.79(a)(1)(vi). [Go/No-Go Criteria]
>
> NUREG-1555, Section 7.1 – Reviewer should "verify that the applicant's proposed exclusion area and low population zone distances are adequate." [i.e., Confirm, not Review]
>
> NUREG-0800, Section 15.0.3 – For COL review, the "application is acceptable with regard to the radiological consequences of analyzed DBAs if the calculated TEDEs at the EAB and the LPZ outer boundary do not exceed the dose acceptance criteria." [Confirm, not Review]

Also, in 10 CFR 52.79(a)(1)(vi), the NRC eliminated the language from 10 CFR 50.34(a)(1)(ii) regarding "safety features that are to be engineered into the facility and those barriers that must be breached as a result of an accident before a release of radioactive material to the environment can occur." The negative implication of this change is that the NRC is not seeking a discussion of those safety features in the partial COL. Instead, those safety features will be discussed in Chapter 15, which will be included in the second portion of the COL application.

Thus, UniStar Nuclear interprets 10 CFR 52.79(a)(1) to require that the applicant confirm that a release – from the specific design at a specific site – not cause a dose in excess of the specified limit. A detailed discussion of the accident analyses will be provided in the U.S. EPR Design Certification application. AREVA NP will formally submit the U.S. EPR Design Certification application including the safety analysis and Probabilistic Risk Assessment (PRA) sections, to the NRC by December 14, 2007.

8. ENVIRONMENTAL REPORT (ER), Chapter 7, "Environmental Impact of Postulated Accidents Involving Radioactive Materials"

To assist in determining how information submitted in this area may or may not address the requirements of 10 CFR 50.30(f) regarding impacts of accidents, the staff uses insights from NUREG-1555, "Standard Review Plans for Environmental Reviews for Nuclear Power Plants," Sections 7.1, 7.2, and 7.3. The industry has expressed interest in using NUREG-1555, rather than the RG 4.2, that is cited in RG 1.206, "Combined License Applications for Nuclear Power Plants," because NUREG-1555 aligns more closely with current practices. NUREG-1555 indicates that at least some in-depth design-and site-specific design-basis accident analysis and probabilistic risk assessment information is vital to the review of ER Sections 7.1, 7.2, and 7.3. Portions of these sections for a COL application overlap with the environmental analysis that is conducted in the evaluation of standardized designs.

The information that is needed to consider design-basis accidents (Section 7.1) should include description and analysis that reflects the radiological consequences that might result from a postulated accident involving release of fission products specific to the site. The information for this environmental analysis is comparable to the information needed for the site safety analysis described in FSAR Chapter 15 as characterized in the discussion immediately above. With respect to the impacts of severe accidents (Section 7.2), the application provides information unrelated to the reactor design being contemplated for Calvert Cliffs Nuclear Power Plant, Unit 3 and relies on a qualitative correlation and comparison rather than a site-specific severe accident impact analysis. With respect to severe accident mitigation alternatives (Section 7.3), the applicant did not provide an analysis; UniStar does provide a brief discussion asserting that the risk from its bounding analysis is so low that there would be no cost-effective severe accident mitigation design alternatives; additionally, there was no discussion of severe accident mitigation alternatives related to procedure development and training.

The information in these sections is not based on the plant-specific risk assessments that are expected to be available when the U.S. EPR design certification application is tendered; furthermore, if the approach to certify the U.S. EPR design is consistent with earlier design certification applications, then the assessment is likely to rely upon assumptions regarding site parameters rather than the site characteristics of a particular location. Consequently, the information in Sections 7.1, 7.2, and 7.3, do not provide the design- and site-specific information nor did the applicant propose an acceptable alternative. Therefore, the staff cannot evaluate how this information may or may not meet the requirements of 10 CFR 50.30(f) regarding impacts of accidents.

UniStar Nuclear Response:

- With respect to the additional information requested concerning Section 7.1 of the CCNPP Unit 3 Environmental Report, UniStar Nuclear does not interpret 10 CFR 2.101(a)(5) to require the level of detail that would normally be included in Chapter 15 of the FSAR. In addition, the guidance in the Environmental Standard Review Plan (i.e., NUREG-1555) suggests that the reviewer examine Chapter 15 of the FSAR, but the regulations do not require the applicant to submit Chapter 15 in this partial COL application.

  While UniStar Nuclear acknowledges that FSAR Chapter 15 may be relevant to the NRC technical review of Chapter 7 of the CCNPP Unit 3 Environmental Report, we note that

the information provided in our partial COL application complies with the applicable regulations and therefore should be sufficient for the NRC to complete its acceptance review of the partial application. AREVA NP will formally submit the U.S. EPR Design Certification application, including the safety analysis and PRA sections, to the NRC by December 14, 2007.

- With respect to the comments regarding Sections 7.2 and 7.3 of the CCNPP Unit 3 Environmental Report, UniStar Nuclear has provided an analysis that is based on the key attributes of the U.S. EPR design and the CCNPP site. For example, the analysis is based on a U.S. EPR-specific Core Damage Frequency and risk profile, and takes into account site-specific parameters (e.g., χ/Q) of the CCNPP site. UniStar Nuclear considers this alternative approach to be both bounding and conservative.

  The detailed information requested by the NRC can only be provided from the PRA for the U.S. EPR. The unavailability of a PRA was an issue that UniStar Nuclear raised with the NRC in April 2006. At that time, UniStar Nuclear was informed that the absence of a PRA would not prevent the NRC from docketing the COL application.

  Additional detail concerning the alternative approach included in Sections 7.2 and 7.3 of the CCNPP Unit 3 Environmental Report is provided in the Section 5.2 of NUREG-1437, "Generic Environmental Impact Statement for License Renewal of Nuclear Plants," Supplement 1, Regarding Calvert Cliffs Nuclear Power Plant, Units 1 and 2, dated October 1999. This document is available on the NRC website at http://www.nrc.gov/reading-rm/doc-collections/nuregs/staff/sr1437/supplement1/.

  In addition, a discussion of severe accident mitigation alternatives relative to procedure development and training will be provided in a revision to the CCNPP Unit 3 partial COL application that will be submitted to the NRC by December 14, 2007. The associated pages of the CCNPP Unit 3 partial COL application, with the proposed changes identified, are attached.

7.3 SEVERE ACCIDENT MITIGATION ALTERNATIVES

The purpose of severe accident mitigation alternatives (SAMA) analysis is to review and evaluate plant-design alternatives that could significantly reduce the radiological risk from a postulated severe accident. Plant changes are reviewed that would reduce the likelihood of a core damage event and that could mitigate the consequences should an accident occur.

{As part of the License Renewal at the Calvert Cliffs Nuclear Power Plant (CCNPP) site, a detailed SAMA evaluation was completed. As discussed in Section 7.2.2.1, scaling of the CCNPP Unit 1 results for the U.S. EPR is possible and would result in a multiplication factor of approximately 0.01 on the consequence results. This reduction when applied to the CCNPP Unit 1 results would yield a conservative estimate for the total severe accident population dose out to 50 mi (80 km) of about 1 person-rem/reactor-year. As described in Supplement 1 of the GEIS (NRC, 1999), elimination of all risk from severe accidents at the CCNPP site is estimated to equate to approximately $8.6 million, with the inclusion of the additional onsite economic costs. Applying the scaling factor would yield a maximum averted cost risk for the U.S. EPR at the CCNPP site of $86,000 (0.01 x $8.6E6). This cost refers to the maximum benefit obtained by the elimination of all severe accident risk and serves as a conservative upper bound cost for screening out potential plant modifications. Beyond what has already been implemented into the U.S. EPR design (refer to Section 7.2.2 for a list of unique features which reduce the severe accident risk and have already been incorporated into the U.S EPR design), there are no additional modifications costing less than $86,000 that would be determined to be cost beneficial. This maximum averted cost risk is judged to be conservative relative to the U.S. EPR design.

The SAMA cost benefit evaluation compares the cost associated with the incremental reduction in consequences for a particular plant modification with the actual cost of implementation. Supplement 1 of the GEIS for CCNPP Units 1 and 2 (NRC, 1999) includes the cost benefit obtained for the top ten potential plant modifications. Using best estimate methods for assessing the benefit of candidate plant modifications, the maximum estimated cost benefit among the top ten SAMAs for CCNPP Units 1 and 2 was approximately $882,000. This represents about 10% of the maximum obtainable reduction if all severe accident risk was eliminated ($882,000 / $8.6E6). Applied to the estimated maximum averted cost risk for the U.S. EPR, the benefit cost would be $8,600 ($86,000 x 0.1). Implementation costs for any additional plant changes would be expected to exceed this maximum value.}

To further investigate the potential for additional cost effective plant modifications, a review of recent License Renewal submittals has been performed to investigate the types of plant changes being considered for pressurized water reactors (PWRs). While there is some variability among different plants, there seems to be several common issues that are included in all of the evaluations. Table 7.3-1 provides a list of some of the more common SAMAs that have been considered to be cost beneficial and a brief description of how the issue is being addressed in the U.S. EPR design.

As can be seen in Table 7.3-1, the typical PWR SAMA candidates are already considered as part of the U.S. EPR design. Since the U.S. EPR is specifically designed to prevent and mitigate severe accidents, no additional plant changes are judged to be cost beneficial. Evaluation of administrative SAMAs (i.e., those SAMAs related to procedures and training) is not appropriate until the plant design is finalized and the plant procedures and training program are being developed. However, the plant procedures and training program will be developed to address appropriate maintenance and use of the U.S. EPR design features which have been credited with the reduction of risk associated with postulated severe accidents. As such,

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appropriate administrative controls on plant operation will be incorporated into the {CCNPP Unit 3} management systems as part of the initial procedure and training development process.

7.3.1 REFERENCES

{NRC, 1999. Generic Environmental Impact Statement for License Renewal of Nuclear Plants, Calvert Cliffs Nuclear Power Plant, NUREG-1437, Supplement 1, Nuclear Regulatory Commission, October 1999.**}**

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