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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

April 07, 2010

UN#10-105

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

Subject: UniStar Nuclear Energy, NRC Docket No. 52-016
Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3,
RAI No. 218, Stability of Subsurface Materials and Foundations

Reference: Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 218 RGS1 4332" email dated March 7, 2010

This letter is provided in response to request for additional information (RAI) 218. RAI 218 contains fourteen questions regarding Stability of Subsurface Materials and Foundations as discussed in Section 2.5.4 of the Final Safety Analysis Report (FSAR), as submitted in Part 2 of the Calvert Cliffs Nuclear Power Plant (CCNPP) Unit 3 Combined License Application (COLA), Revision 6.

The enclosure to this letter provides the response to RAI 218, Question 02.05.04-04, 06, 07, 11, 12, 14, 15 and 16. This response includes revised COLA content.

UniStar Nuclear Energy requires additional time to perform the additional analyses that required to finalize the response to RAI Questions 2.05.04-03, 2.05.02-08, and 02-05-13. The responses to these questions will be provided by June 30, 2010.

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NRO

The responses to Questions 2.05.04-05, 02.05.04-09 and 2.05.04-10 are tied to the resolution of technical issues that will be answered in the responses to questions in RAI 144 and 145. These responses will be provided by July 23, 2010.

The enclosed responses do not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

If there are any questions regarding this transmittal, please contact me at (410) 470-4205, or Mr. Wayne A. Massie at (410) 470-5503.

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

Executed on April 07, 2010

for Greg Gibson

Enclosure: Response to NRC Request for Additional Information, RAI No. 218; Stability of Subsurface Materials and Foundations, Questions 02.05.04-04, 06, 07, 11, 12, 14, 15 and 16; Calvert Cliffs Nuclear Power Plant, Unit 3

cc: Surinder Arora, NRC Project Manager, U.S. EPR Projects Branch
Laura Quinn, NRC Environmental Project Manager, U.S. EPR COL Application
Getachew Tesfaye, NRC Project Manager, U.S. EPR DC Application (w/o enclosure)
Loren Plisco, Deputy Regional Administrator, NRC Region II (w/o enclosure)
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GTG/SJS/mdf

Enclosure

Response to NRC Request for Additional Information

RAI No. 218, Stability of Subsurface Materials and Foundations, Questions 02.05.04-04, 06, 07, 11, 12, 14, 15 and 16

Calvert Cliffs Nuclear Power Plant Unit 3

RAI 218 Question 02.05.04-4

Section 2.5.4.5.2 presents information on the planned extent of excavation and fills to be placed in and around the Category 1 structures and indicates that the extent of excavation will be based on the observation of actual conditions at the time of the excavation. The applicant is requested to describe the procedures that will be used by field investigators to judge if in-situ soils are to be left in place.

Response

The design intent for the foundation soils is to provide a competent foundation for Calvert Cliffs Nuclear Power Plant Unit 3 that is not susceptible to liquefaction. The existing in-situ materials that meet this design intent are defined as Stratum IIb-Chesapeake Cemented Sand. These materials are typically light to dark gray in color and have a SPT N value generally greater than 20.

Once the design elevation is reached, two methods of verifying the competent foundation soils may be used. The first method is to proof roll the entire excavated area with a compaction vehicle or approved equivalent until the grade offers a relatively unyielding surface (i.e., less than one inch). Any areas that exhibit excessive (i.e., greater than one inch) rutting, pumping, or yielding will be identified by the Geotechnical Engineer; and the construction contractor will undercut these areas until the intended competent Stratum is encountered, as verified by additional proof rolling.

The second method is to perform in-situ compaction testing by means of ASTM D7380-08 "Standard Test Method for Soil Compaction Determination (Dynamic Cone Penetration)" and/or ASTM D1556 "Test Method for Density and Unit Weight of Soil in-Place by Sand-Cone Method."

Structural backfill placement will not begin until the unsuitable material of the final excavation grade has been verified and approval received from the Geotechnical Engineer. The Geotechnical Engineer will be responsible for final approval of the foundation soils. A geologist will map the exposed stratum and photos and videotape will be collected for documentation. Finally, acceptance will be documented on a Final Foundation Acceptance form that is completed by the responsible parties and included in the report.

COLA Impact

The following paragraph will be inserted in CCNPP Unit 3 FSAR Section 2.5.4.5.2. Note that this markup is to the FSAR text provided in UniStar Nuclear Energy (UNE) letter UN#09-427[1] on October 9, 2009.

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

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[1] G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

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.

Once the design foundation elevation is reached, two methods of verifying the competent foundation soils may be used. The first method is to proof roll the entire excavated area with a compaction vehicle or approved equivalent until the grade offers a relatively unyielding surface (i.e., less than one inch). Any areas that exhibit excessive (i.e., greater than one inch) rutting, pumping or yielding will be identified by the Geotechnical Engineer and the construction contractor will undercut these areas until the intended competent Stratum is encountered as verified by additional proof rolling. The second method is to perform in-situ compaction testing by means of ASTM D7380-08 "Standard Test Method for Soil Compaction Determination (Dynamic Cone Penetrometer)"(ASTM 2008b) and/or ASTM D1556 "Standard Test Method for Density and Unit Weight of Soil in-Place by Sand-Cone

Method" (ASTM 2007b). Structural backfill placement will not begin until the unsuitable material of the final excavation grade has been verified and approval received from the Geotechnical Engineer.

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.

RAI 218 Question 02.05.04-6

Section 2.5.4.2.5.8 presents the low strain dynamic properties for the backfill soil and indicates that the shear wave velocity for the backfill below the EPGB is about 900 fps. This velocity is lower than the minimum shear velocity (1,000 fps) specified in the U.S. EPR standard design and thus was identified as a departure in this COL application. In addition, the minimum shear wave velocity definition was also revised in the latest U.S. EPR standard design, which no longer uses the "best estimate" concept. Please update the corresponding ITAAC to reflect the changes of the DCD and the departure. In addition, please refer the NRC's August 7, 2009 letter to NEI regarding the NRC staff position and standard wording for backfill ITAAC under Category I structures.

Response

UNE letter UN#10-027[2] dated January 29, 2010 provided an update to the Shear Wave Velocity ITAAC in COLA Part 10 and an update to the departure in COLA Part 7.

COLA Impact

None

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[2] G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Shear Wave Velocity Inspections, Tests, Analyses, and Acceptance Criteria (ITAAC) Update and Departure" Letter UN#10-027, dated January 29, 2010.

RAI 218 Question 02.05.04-7

Section 2.5.4.5.3 states that structural fill will be compacted to a minimum 95 percent of its maximum dry density, and within 3 percent of its optimum moisture content, based on the Modified Proctor Compaction test procedure. Section 2.5.4.5.3 further states that the in-place density and moisture content testing frequency will be a minimum of one test per 10,000 square feet fill placed. Please justify whether the backfill field density test parameter (one test for every 10,000 ft^2) is adequate by itself without specifying other controls or procedures, such as no lift should be more than 8 inches in thickness and a routine acceptance control test should be conducted for at least every 200 cubic yards of compacted backfill material in critical areas.

Response

The minimum of one test per 10,000 ft^2 of fill is sufficient provided the following quality controls are taken:

Structural Fill
Structural fill should be granular in nature, with well-graded sand, gravel, or crushed gravel, and typically should not contain more than 10 percent by weight of material passing No. 200 sieve and no less than 95 percent by weight passing the 3/4-inch (in.) sieve. The maximum allowable aggregate size shall be 1 in. Gradation shall be determined in accordance with ASTM D422 and D1140. The fill should consist of durable materials free from organic matters or any other deleterious or perishable substances, and of such a nature that it can be compacted readily to a firm and non-yielding state.

Compaction Requirements
Structural fill will be compacted at a moisture content of ±3 percent of the optimum, and compaction will be done to 95 percent of Modified Proctor optimum dry density. The maximum dry density and optimum moisture content is determined in accordance with ASTM D1557.

Clearing and Preparing Fill Areas
Prior to placing structural fill, the excavation bottom to receive fill needs to be observed, probed, tested, and approved by qualified personnel as a part of the quality control measures.

Placing, Spreading and Compacting Fill Material
Fill materials must be placed in horizontal layers usually not greater than 8 inches in loose thickness. Each layer is required to be spread evenly and mixed thoroughly to obtain uniformity of material and moisture in each layer. When the moisture content of the fill material is below that specified, water needs to be added until the moisture content is as specified. When the moisture content of the fill material is too high, the fill material needs to be aerated through blading, mixing, or other satisfactory methods until the moisture content is as specified. After each fill layer has been placed, mixed and spread evenly, it needs to be thoroughly compacted to the specified degree of compaction. Compaction needs to be accomplished by acceptable types of compacting equipment. The equipment is required to be of such design and nature that it is able to compact the fill to the specified degree of compaction. Compaction should be continuous over the entire area and the equipment should make sufficient passes to obtain the desired uniform compaction.

Observation and Testing of Fill Placement
Continuous geotechnical engineering observation and inspection of fill placement and compaction operations is required to certify and ensure that the fill is properly placed and compacted in

accordance with the project plans and specifications. Field density tests in accordance with ASTM D1556 are required to be performed for each layer of fill. Moisture content may be determined in the laboratory (ASTM D2216) or in the field (ASTM D6938). If the surface is disturbed, the density tests are to be made in the compacted materials below the disturbed zone. When tests indicate that the degree of compaction of any layer of fill or portion thereof does not meet the specified minimum requirement, the particular layer or portions requires reworking until the specified relative compaction is obtained.

At least one in-place moisture content and field density test is required on every 10,000 ft^2 of each lift of fill, and further placement is not allowed until the required relative compaction has been achieved. The number of tests is increased if a visual inspection determines that the moisture content is not uniform or if the compacting effort is variable and not considered sufficient to meet the project specification. For critical areas, at least one in-place moisture content and field density test is required for every 200 cubic yards of compacted fill.

COLA Impact

FSAR Section 2.5.4.5.3 will be updated as follows. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[3] on October 9, 2009.

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.~~

Testing of structural backfill is described in Section 2.5.4.2.4. For foundation support and backfill against walls, structural fill should be granular in nature, with well-graded sand, gravel or crushed gravel, and typically should not contain more than 10 percent by weight of material passing No. 200 sieve and no less than 95 percent by weight passing the 3/4-inch sieve. The maximum allowable aggregate size shall be 1 inch. Gradation shall be determined in accordance with ASTM D422 and D1140. The fill should consist of durable materials free from organic matters or any other deleterious or perishable substances, and of such a nature that it can be compacted readily to a firm and non-yielding state.

Structural fill will be compacted at a moisture content of ±3 percent of the optimum, and compaction will be done to 95 percent of Modified Proctor optimum dry density. The maximum dry density and optimum moisture content is determined in accordance with ASTM D1557 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3(2700 kN-m/m^3))," (ASTM, 2009).

3 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

Fill materials need to be placed in horizontal layers not greater than 8 inches in loose thickness. Each layer is required to be spread evenly and mixed thoroughly to obtain uniformity of material and moisture in each layer. When the moisture content of the fill material is below that specified, water needs to be added until the moisture content is as specified. When the moisture content of the fill material is too high, the fill material needs to be aerated through blading, mixing, or other satisfactory methods until the moisture content is as specified. After each fill layer has been placed, mixed and spread evenly, it needs to be thoroughly compacted to the specified degree of compaction. Compaction needs to be accomplished by acceptable types of compacting equipment. The equipment is required to be of such design and nature that it is able to compact the fill to the specified degree of compaction. Compaction should be continuous over the entire area and the equipment should make sufficient passes to obtain the desired uniform compaction.

Continuous geotechnical engineering observation and inspection of fill placement and compaction operations is required to certify and ensure that the fill is properly placed and compacted in accordance with the project plans and specifications. Field density tests in accordance with ASTM D1556 "Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method, American Society for Testing and Materials" (ASTM, 2007b) are required to be performed for each layer of fill. Moisture content may be determined in the laboratory in accordance with ASTM D2216, "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass" (ASTM, 2005c) or in the field using nuclear methods in accordance with ASTM D6938 "Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)," (ASTM, 2008b). If the surface is disturbed, the density tests are to be made in the compacted materials below the disturbed zone. When these tests indicate that the degree of compaction of any layer of fill or portion thereof does not meet the specified minimum requirement, the particular layer or portions requires reworking until the specified relative compaction is obtained.

At least one in-place moisture content and field density test is required on every 10,000 square feet of each lift of fill, and further placement is not allowed until the required relative compaction has been achieved. The number of tests is increased if a visual inspection determines that the moisture content is not uniform or if the compacting effort is variable and not considered sufficient to meet the project specification. For critical areas, at least one in-place moisture content and field density test is required for every 200 cubic yards of compacted fill.

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~~ inches) 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.

Add the following references to FSAR Section 2.5.4.13:

ASTM, 2005c, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass, American Society for Testing and Materials, ASTM 2216-05 [Report], 2009.

ASTM, 2007b, Standard Test Method for Density and Unit Weight of Soil in Place by Sand-Cone Method, American Society for Testing and Materials, ASTM 1556-07 [Report], 2009.

ASTM, 2008a, Standard Test Method for Soil Compaction Determination at Shallow Depths Using 5-lb (2.3 kg) Dynamic Cone Penetrometer, ASTM D7380-08 [Report], 2009.

ASTM, 2008b, Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth), American Society for Testing and Materials, ASTM 6938-08a, [Report], 2009.

ASTM, 2009, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3(2700 kN-m/m^3)), American Society for Testing and Materials, ASTM 1557-09 [Report], 2009.

RAI 218 Question 02.05.04-11

Section 2.5.4.2.5.8, which provides low strain dynamic properties for the subsurface materials at the site, as well as for the backfill soil used, states that the groundwater level is at an approximate depth of 16 ft for the powerblock area. Once the construction is finalized, the expected depth of the groundwater is 30 ft due to new drainage patterns. Also, Section 2.5.10.2.2 states that the post-construction groundwater elevation in the powerblock area was assumed at El. 55 ft for the settlement analysis, which is about 28 ft below grade surface (El. 83 ft). However, it is stated in Section 2.4.12.5 that the maximum pre-construction groundwater level is currently at or slightly above the proposed grade level in the nuclear island area, while post-construction groundwater level ranges from approximately 6 ft to 16 ft below ground surface. Since ground water level will affect the stability of site subsurface materials, foundations, structures and slopes, please explain the discrepancy of post-construction ground water levels provided in Section 2.4.12.5 and Section 2.5.4. If the post–construction groundwater level is as stated in Section 2.4.12.5, please discuss the impact of using higher ground water level on site seismic response, SSI, settlement and lateral earth pressure analyses.

Response

The analyses presented in FSAR Section 2.5.4 are based on the lower groundwater elevation level of 55 feet, which is the correct groundwater elevation. The higher groundwater elevation currently described in FSAR Section 2.4.12.5 will be updated as part of the responses to RAI 101, Questions 02.04.12-9 through 02.04.12-11 (currently in development).

COLA Impact

None

RAI 218 Question 02.05.04-12

Section 2.5.4.2.5.9 described states that "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." Although the Bechtel report describes how the strain dependent properties were developed for Strata 1, IIa, IIb, IIc and III soils, there is no discussion for the backfill. Please describe how the strain dependent properties for backfill soil, which are presented in Figure 2.5-172, were developed.

Response

FSAR Table 2.5-63 and Figure 2.5-172 of provide the strain-dependant properties for backfill. The process used in the determination of the shear modulus reduction and damping curves follows.

Resonant Column Torsional Shear (RCTS) tests were performed on bulk samples collected from potential borrow areas. The RCTS results are contained in the Letter Report "Structural Fill Dynamic Laboratory Testing Results, Rev. 1," dated June 30, 2009 provided in COLA Part 11J. A total of eight RCTS tests were performed and are summarized in the following table:

RCTS BACKFILL SAMPLES

SAMPLE	REMOLDED CONDITION
CR6 Blended	95 percent Modified Proctor maximum dry density
	100 percent Modified Proctor maximum dry density
GAB Blended	95 percent Modified Proctor maximum dry density
	100 percent Modified Proctor maximum dry density
57 Blended	80 percent Modified Proctor maximum dry density
	100 percent Modified Proctor maximum dry density
CR6 Vulcan Quarry Statistical Average	95 percent Modified Proctor maximum dry density
	100 percent Modified Proctor maximum dry density

The criteria used to develop the recommendation for strain-dependant properties of backfill material is as follows:

- Tests performed at 95 percent of the target optimum unit weight were selected for the recommendation. These tests provide a more realistic compaction level with slightly less stiffness values when compared to the 100 percent target case. They represent adequate compaction requirements when compared to the 80 percent target.
- Estimated confining pressures underneath the Emergency Power Generation Building (EPGB) and the Essential Service Water Building (ESWB) were compared against RCTS data points from similar confining pressures.
- Figures 2.5-149, 2.5-150, and 2.5-151 of the FSAR provide RCTS results for confining pressures that are representative of field conditions. This data was used

to develop the recommended curves for backfill.

- As indicated in FSAR Section 2.5.4.2.5.9, Electric Power Research Institute (EPRI) generic curves for granular soils are compared against the laboratory data and then adjusted to fit to the sample specific results. Figures 2.5-149, 2.5-150, and 2.5-151 show the recommended EPRI fitted curve along with the laboratory data.
- Damping is assigned using the lower range values from laboratory data.

COLA Impact

None

RAI 218 Question 02.05.04-14

Section 2.5.4.8 presents liquefaction potential analysis results and concludes that liquefaction is not a concern for this site. However, the data also show that the upper soil layer (Terrace Sand) does have some potential for liquefaction. Since seismic Category I electrical duct banks and pipes will be located at shallow depths, please discuss the liquefaction potential of soil where these components will be located.

Response

Figure 2.5-176 of the FSAR indicates a factor of safety (FOS) <1.1 against liquefaction for the Powerblock Area in Stratum I Terrace Sand. In locations where seismic Category I electrical duct banks and pipes are to be laid, Stratum I Terrace Sand should be excavated in its entirety to the top of Stratum IIa Chesapeake Clay/Silt. Pipe bedding and engineered backfill should be placed around electrical duct banks and pipes. Above the pipe zone (backfill to grade elevation), general structural fill may be used with a similar degree of compaction as specified for the bedding materials.

COLA Impact

The following paragraph will be inserted in FSAR Section 2.5.4.5.2. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[4] on October 9, 2009.

2.5.4.5.2 Extent of Excavations, Fills and Slopes

. . .

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.

[The response to question 2.5.4-4 is located here]

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

4 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

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.

Excavation for Seismic Category I electrical duct banks and pipes in the Powerblock Area involve the removal of Stratum I Terrace Sand in its entirety to the top of Stratum IIa Chesapeake Clay/Silt. Such excavation is required since the Stratum I layer has potential for liquefaction, as indicated in Section 2.5.4.8. Pipe bedding and engineered backfill should be placed around electrical duct banks and pipes. Above the pipe zone (backfill to grade elevation), general structural fill may be used with a similar degree of compaction as specified for the bedding materials.

RAI 218 Question 02.05.04-15

Section 2.5.4.10.1 states that three cases were considered during bearing capacity calculations. For the general case, the bearing capacity equation for homogeneous soil was used by applying weighted average values of soil parameters in the analysis, with the weight factors based on the relative thickness of each stratum within a specific depth. For the case of a footing supported on a dense sand stratum over a soft clay stratum, Meyerhof's model (Meyerhof, et al., 1978) was used to estimate ultimate static bearing capacity. Since the results of the bearing capacity analysis were controlled by the models, assumptions and parameters, the applicant is requested to:

1. Provide details on how the weight factors were determined for all subsurface soil strata;

2. Clarify and justify if soil compressibility was considered during the analysis since a clayey sand layer (Layer IIb2) is presented;

3. Discuss whether the dimension of a structure will affect the analysis results for footing supported on a dense sand stratum overlying on a soft clay stratum, because the Meyerhof model is based on the assumption that one dimension of the rectangular foundation is much larger than the other. Also, please clarify why the equation of q_{ult} presented in page 2-1252 is different from Meyerhof's equation by a factor of 2.

Response

Sub-Question 1:

The ultimate static bearing capacity of the soil subsurface for all buildings was calculated using the following methods:

1. The model proposed by Vesic (1973, 1975) for footings supported on homogeneous soils. This method considers a homogeneous isotropic continuous medium underneath the foundation. Therefore, weighted average values of c', φ' and γ', based on the relative thickness of each stratum in the zone between the bottom of the foundation and a certain influence depth, were used in the calculations to obtain equivalent properties for the continuous medium. For this analysis, an influence depth of 1B was considered to be adequate, where B is the building least lateral dimension. Two different cases are considered in the analysis:

   a) Soil subsurface including all strata: weighted average values of c', ϕ' and γ' are used to obtain equivalent properties for the continuous medium.

   b) Soil subsurface including only stratum IIb Chesapeake Cemented Sand, with its corresponding three sublayers. Soil parameters of this layer are used to obtain equivalent properties for the continuous medium. For this case the depth of influence is considered to be the thickness of stratum IIb, instead of 1B.

2. The model proposed by Meyerhof (1980, 1974) for footings supported on a dense sand stratum over a soft clay stratum. Weighted average parameters of case b) of Vesic solution were used for the stiff sand layer and values corresponding to layer IIc Chesapeake Clay/Silt were used for the clay layer.

Sub-Question 2:

Soil compressibility was not considered during the analysis. Soil compressibility will be addressed in RAI Question 02.05.04-3 by introducing variability in compressible layer thickness and soil properties.

Sub-Question 3:

The equations recommended in the Meyerhof model that were used in the analysis are obtained from Technical Engineering and Design Guides, No. 7: "Bearing Capacity of Soils," American Society of Civil Engineers, 1993. Moreover, the information provided in this reference is based on the study performed by Hanna and Meyerhof "Design Charts for Ultimate Bearing Capacity of Foundations on Sand Overlying Soft Clay" Canadian Geotechnical Journal Volume 17, 1. Based on this publication, the following equation is recommended to estimate the ultimate bearing capacity for a strip footing of width B and depth D supported on a dense sand stratum over a soft clay stratum:

$$q_u = q_b + \gamma_1 H^2 \left(1 + \frac{2D}{H}\right) \frac{K_s tan\phi_1}{B} - \gamma_1 H \leq q_t$$

The authors propose that this equation can be extended to circular footings as follows:

$$q_u = q_b + 2\gamma_1 H^2 \left(1 + \frac{2D}{H}\right) \frac{S_s K_s tan\phi_1}{B} - \gamma_1 H \leq q_t$$

Where
S_s varies between 1.1 and 1.27 and may be taken as unity, for conservative design; and
B is the diameter of the circular foundation or width in case of a rectangular foundation.

COLA Impact

None

RAI 218 Question 02.05.04-16

1. Please verify that in the last paragraph of page 2-1247, "Only data points in the upper layers resulted in FOS >1.1" should be " … FOS <1.1."
2. Please verify that the term N' used in equation for qult, (page 2-1251), and Ng in the note should be Nγ.
3. Please verify that in equation term notes (page 2-1252), q_u should be q_{ult}.

Response

1. The last paragraph of page 2-1247, "Only data points in the upper layers resulted in FOS>1.1," should read "Only data points in the upper layers resulted in FOS<1.1."
2. The term N' used in equation for q_{ult}, (page 2-1251) and Ng in the note should both be Nγ.
3. In equation term notes (page 2-1252), q_u should be q_{ult}.

COLA Impact

The following changes will be made in FSAR Section 2.5.4.8.6 to correct the text as described above. Note that this markup is to the FSAR text provided in UNE letter UN#09-427[5] on October 9, 2009.

Part 1:

> Only data points in the upper layers resulted in ~~FOS>1.1~~ 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.

Part 2:

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

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

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

5 G. Gibson (UniStar Nuclear Energy) to Document Control Desk (U.S. NRC), "Update to Calvert Cliffs Nuclear Power Plant, Unit 3 FSAR Sections 2.5.4 and 2.5.5," Letter UN#09-427, dated October 9, 2009.

Where:

q_{ult}	→	Ultimate bearing capacity;
c	→	Cohesion;
~~N_c, N_g, N_q~~	~~→~~	~~Bearing capacity factors;~~
N_c, N_γ, N_q	→	Bearing capacity factors;

and Part 3:

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):

Where:

~~q_u~~	~~→~~	~~Ultimate bearing capacity;~~
q_{ult}	→	Ultimate bearing capacity;