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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

March 31, 2011

UN#11-113

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. 268, Stability of Subsurface Materials and Foundations

References:
1) James Steckel (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 268 RGS2 5120" email dated November 4, 2010

2) UniStar Nuclear Energy Letter UN#11-067, from Greg Gibson to Document Control Desk, U.S. NRC, Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 268, Stability of Subsurface Materials and Foundations, dated January 31, 2011

The purpose of this letter is to respond to the request for additional information (RAI) identified in the NRC e-mail correspondence to UniStar Nuclear Energy, dated November 4, 2010 (Reference 1). This RAI addresses 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 7.

Reference 2 provided a March 31, 2011 date for the response to RAI 268 Questions 02.05.04-26, 02.05.04-27, and 02.05.04-28.

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NRO

The Enclosure provides our response to RAI 268 Questions 02.05.04-26 and 02.05.04-27, and includes revised COLA content. A Licensing Basis Document Change Request has been initiated to incorporate these changes into a future revision of the COLA.

Our response does not include any new regulatory commitments. This letter does not contain any sensitive or proprietary information.

A response to Question 02.05.04-28 will be provided by April 29, 2011.

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 March 31, 2011

Greg Gibson

Enclosure: Response to NRC Request for Additional Information, RAI 268 Questions 02.05.04-26 and 02.05.04-27, Stability of Subsurface Materials and Foundations, 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)
Charles Casto, Deputy Regional Administrator, NRC Region II (w/o enclosure)
Silas Kennedy, U.S. NRC Resident Inspector, CCNPP, Units 1 and 2
U.S. NRC Region I Office

Enclosure

Response to NRC Request for Additional Information

RAI 268 Questions 02.05.04-26 and 02.05.04-27, Stability of Subsurface Materials and Foundations

Calvert Cliffs Nuclear Power Plant Unit 3

RAI 268

Question 02.05.04-26:

In response to RAI Question 02.05.04-17, you provided settlement analysis results using the Soil Hardening (SH) constitutive model and the Middle Topography 2 (MT2) model. In order for staff to complete a detailed review to ensure the stability of foundations in accordance with 10 CFR 100.23, please provide the following information:

1. You state that the MT2 model is discussed in Section 2.5.4.10 of the FSAR, however the staff could not find any mention of the MT2 model in this section. Please explain this discrepancy.

2. For both the MT2 and SH models analyses, discuss a) the adequacy of finite element mesh size, which can affect the plastic model analysis results; b) the effect of distances between the edge of the foundations and the fixed boundary elements; and c) the effect of non-uniform loading conditions.

3. In Part 2 of your response, you estimated the potential for liquefaction related settlement using both Tokimantsu and Seed, and Lee methods, and presented the results in Tables 4 and 5 of this RAI response. The calculated values for t_0/s_0 and $[t_0/s_0]_M$ in Table 4, and values under "Conditional" and e_v[%] columns do not agree with the expressions for $CSR_{7.5}$ and e_c provided in this RAI response. Please explain.

4. Tables 13 and 14 in Part 3 of your response provide comparisons of building center settlements and tilts using the best estimate and the lower bound soil property parameters in the MT2 models. Although the lower bound parameters are based on the 16th percentile, the approach is reasonable due to the notable variations in soil parameters. Analysis results show that the maximum total settlement will exceed 20 inches and the tilt will exceed 1.0 in/ 50 ft with the lower bound soil parameters. Since the standard design tilt differential settlement limit is 0.5 in/ 50 ft and the requested departure and exemption in this COL application is 1.0 in/ 50 ft, please discuss and justify the adequacy of the departure and the exemption of differential settlement that is requested in the CCNPP COL application

5. In Part 5 of your response, you state that the Boussinesq solution is used in a hand calculation of settlement because "both theory and experience have shown that the shape of the pressure bells (induced stress distribution) is more or less independent of the physical properties of the loaded subsurface. That is, the stress increase due to external loads is not a function of soil properties." You also state that the comparison of the two approaches (Boussinesq solution and PLAXIS 2D finite element model) indicated that the difference between the theoretical solution without any stiffness input and a layered subsurface model is marginal. Please provide additional information on the following items:

   a) Provide references that support the statement, "both theory and experience have shown that the shape of the pressure bells (induced stress distribution) is more or less independent of the physical properties of the loaded subsurface" for layered soil since it is well known that the Boussinesq solution is an elastic solutions based on assumptions of load acting on a weightless material in a linear-elastic homogeneous isotropic half-space and not subject to initial stress. Experience has shown that the actual stresses beneath the center of shallow footing may exceed the Boussinesq values by 15 to 30 percent in clays and 20 to 30 percent in sands and that stress distribution in layered soil cannot be accurately estimated by Boussinesq solution without corrections (see Burmister, D. M 1954[1], 1963[2] and 1965[3]).

b) Provide additional details on the differences in the calculated stresses (in percentage), between the theoretical solution without any stiffness input and a layered soil finite element model (PLAXIS 2D model).

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1. Burmister, D. M. 1954. "Influence Diagram of Stresses and Displacements in a Two-Layer Soil System With a Rigid Base at a Depth H," Contract No. DA-49-129-ENG-171 with US Army Corps of Engineers, Columbia University, New York, NY. Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.
2. Burmister, D. M. 1963. "Physical, Stress-strain, and Strength Responses of Granular Soils," Field Testing of Soils, ASTM Special Technical Publication No. 322, pp 67-97, Available from American Society for Testing and Materials, 1916 Race Street, Philadelphia, PA 19103.
3. Burmister, D. M. 1965. "Influence Diagrams for Stresses and Displacements in a Two-Layer Pavement System for Airfields," Department of the Navy, Washington, DC. Available from Department of the Navy, Washington, DC 20350.

Response

Item 1

The Middle Topography 2 (MT2) model described in RAI Response 02.05.04-17[a] corresponds to the Medium Elevation E Revert (2) model discussed in COLA FSAR Revision 7 Section 2.5.4.10, on page 2-1026.

Item 2

The MT2 and SH models have 42,130 and 88,124 elements, respectively. The effect of mesh size on the settlement estimate is checked by changing the mesh of the MT2 model. Both the SH and MT2 models can account for the possible plastic states and the SH model mesh already has a higher number of elements with a finer mesh. To address the adequacy of the mesh refinement and the distance from the edge of the foundation to the lateral model boundaries, four models were created as shown in Table 1 (MT2-1, MT2-FV, MT2-FH, and MT2-WI models). The representation of the models is shown in Figure 1.

The MT2-1 model has the same mesh and model dimensions as the FSAR MT2 model. For the purpose of the comparison of settlements that result from evaluations with models of different mesh sizes, the end-of-construction loads were applied in one step and the primary loading elastic modulus was used instead of the unload/reload soil modulus.

Table 2 indicates the variability in the settlement results underneath the footprint center of each building. The impact of the mesh refinement is negligible, which indicates that the element size and configuration of the MT2 model is adequate and computationally optimal. The maximum difference in settlement of the models with respect to MT2-1 model is less than 1 percent.

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[a] G. Gibson (UniStar Nuclear Energy) to Document Control Desk ((NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 218 and 229, Stability of Subsurface materials and Foundations," letter UN#10-207 dated July 23, 2010.

The settlement obtained from the MT2-WI model is less than 1.4 percent different from the settlement obtained from the MT2-1 model for any building on the CCNPP Unit 3 site. Therefore, the distance between the edge of the foundation and the fixed boundaries for the MT2 and MT2-1 models, as well as for the SH model, is adequate.

Table 1
Descriptions of Models

MODEL	ELEMENTS	SIZE [ft]	FEATURES
MT2-1	42,130	2500 x 2500	Same mesh as the MT2 Model.
MT2-HF	89,360	2500 x 2500	Finer mesh in the horizontal plane of the foundation clusters.
MT2-VF	114,406	2500 x 2500	Finer mesh in the horizontal and vertical planes.
MT2-WI	32,712	4000 x 4000	Slightly coarser in horizontal and vertical planes.

Table 2
Settlement Variability with Respect to MT2-1

BUIULDING	SETTLEMENT VARIABILITY WITH RESPECT TO MT2-1 Results [%]			
	MT2-1(1)	MT2-HF(2)	MT2-VF(3)	MT2-WI(4)
REACTOR	0	0.12	0.12	0.42
FB	0	0.23	0.20	0.44
SGB1	0	0.23	0.25	0.63
SGB23	0	0.01	0.02	0.48
SGB4	0	0.08	0.07	0.25
NAB	0	-0.19	-0.19	1.12
AB	0	0.15	0.19	1.77
RWPB	0	-0.46	-0.42	-0.52
ESWB1	0	0.69	0.70	1.11
ESWB2	0	0.48	0.48	1.36
ESWB3	0	0.31	0.31	-0.09
ESWB4	0	0.41	0.42	-0.27
EPGB1	0	0.02	0.02	0.31
EPGB2	0	-0.75	-0.74	0.51
TB	0	-0.04	0.36	-0.46

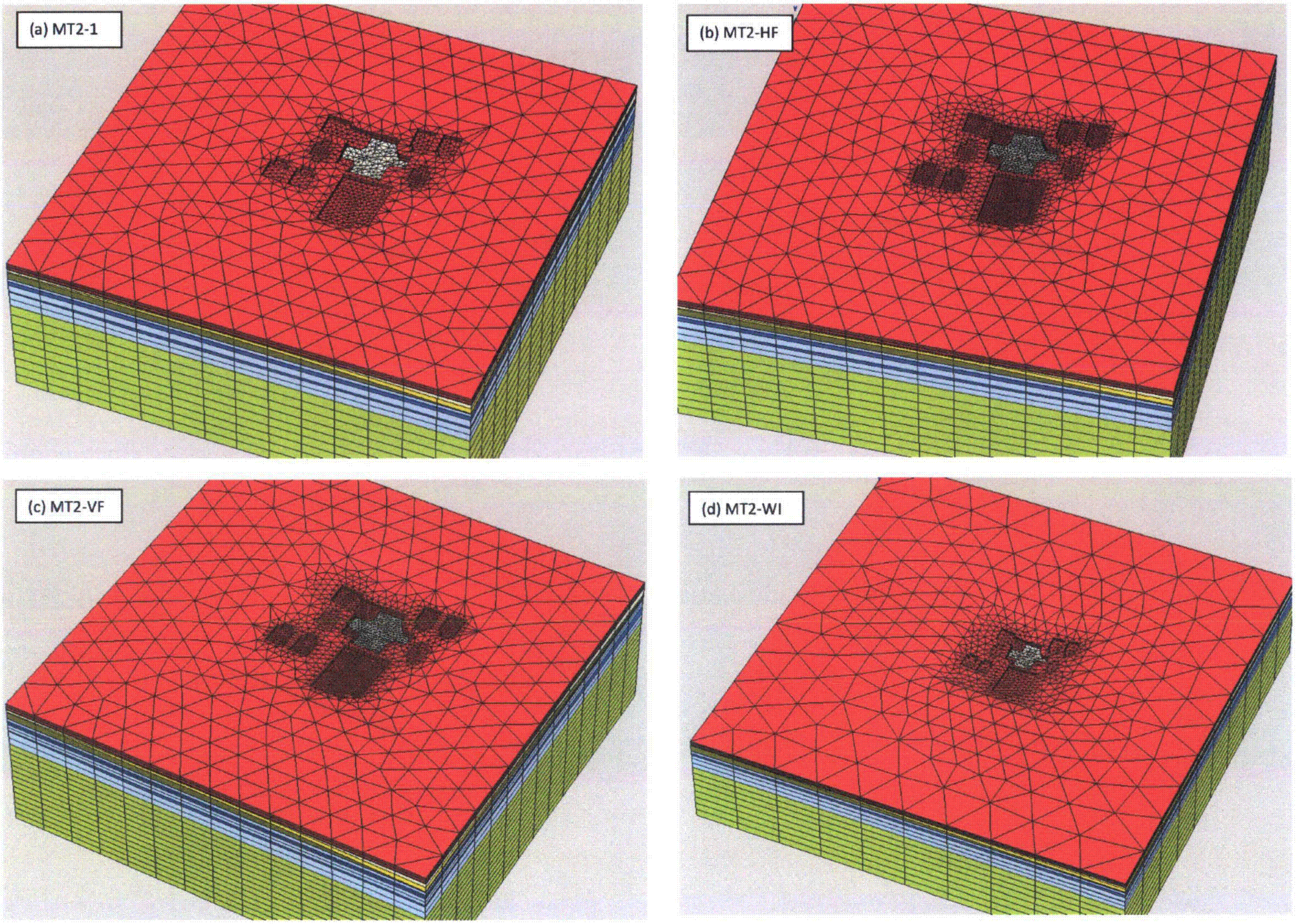
NOTES:
(1) Analogous to MT2 Model described in COLA Rev.7, 42130 elements
(2) The mesh of MT2 refined in only horizontal plane, 89360 elements
(3) The mesh of MT2 refined in vertical and horizontal planes, 114406 elements
(4) The counterpart of MT2-1 model with the model dimensions of 4000 ft by 4000 ft, 32712 elements

For the MT2 model, the maximum effective vertical stress at the bottom of the model for the initial configuration is 51,190 psf, increasing to 52,380 psf at the end of the excavation. The change in effective vertical stress is about 2.3 percent of the initial effective overburden stress. For the SH model, the effective vertical stress at the bottom of the model for the initial configuration (with different elevations and surface loading) is 51,200 psf, increasing to 53,300 at the end of loading step 8. The change in effective vertical stress is less than 4.1 percent of the initial effective overburden stress. Therefore, the depth selected in both models satisfies the requirement stated by U.S. Nuclear Regulatory Commission Regulatory Guide (1.132).

Because the models are FEM three-dimensional models, they inherently account for the load non-uniformities.

In conclusion, the implemented finite element mesh and model dimensions have been optimized for computational efficiency and are adequate representations for settlement estimation purposes.

Figure 1
Representation of the Mesh for the Four Settlement Models



Item 3

This inconsistency is due to an error in the tables provided as Table 4 and 5 in the Response to RAI 229 Question 02.05.04-17[a]. However, this error did not impact the conclusion about the seismic settlements provided in the response. Corrected tables are provided below as Tables 3 and 4.

TABLE 3
SETTLEMENT ANALYSIS BASED ON TOKIMANTSU AND SEED (1987)

SOIL LAYER	DEPTH [ft]	THICKNESS [ft]	MIDPOINT [ft]	γ_{moist} [pcf]	w [%]	γ_{dry} [pcf]	e [ ]	γ_{sat} [pcf]	σ_0 [TSF]	u[TSF]	σ'_0 [TSF]	N_{60}	C_N	$(N_1)_{60}$	r_d [ ]	τ_{av}/σ'_0 [ ]	$[\tau_{av}/\sigma'_0]_{7.5}$ [ ]	ε_c [%]	ρ_c [in]
Stratum I, Terrace Sand	28.0	28.0	14.0	121	15.8	104	0.85	133	0.9	0.4	0.5	14	1.45	20	0.9347	0.1715	0.1299	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 1	71.0	24.0	12.0	122	24.1	98	0.80	126	3.8	1.8	1.9	89	0.74	66	0.6000	0.1148	0.0870	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 2	94.0	23.0	11.5	123	30.5	94	0.80	122	5.2	2.6	2.6	24	0.63	15	0.6000	0.1156	0.0876	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 3	110.0	16.0	8.0	123	26.0	98	0.64	122	6.4	3.2	3.2	63	0.57	36	0.6000	0.1163	0.0881	0.00	0.0
Stratum III, Nanjemoy Sand	411.0	108.0	54.0	127	29.1	98	1.00	130	20.7	11.1	9.6	75	0.33	25	0.6000	0.1246	0.0957	0.00	0.0

TABLE 4
SETTLEMENT ANALYSIS BASED ON LEE (2007)

SOIL LAYER	DEPTH [ft]	THICKNESS [ft]	MIDPOINT [ft]	γ_{moist} [pcf]	w [%]	γ_{dry} [pcf]	e [ ]	γ_{sat} [pcf]	σ_0 [TSF]	u[TSF]	σ'_0 [kPa]	σ'_0 [TSF]	N_{60}	C_N	$(N_1)_{60}$	r_d [ ]	$CSR_{7.5}$ [ ]	$CSR/(N_1)_{60}$ >0.01	CONDITIONAL (1)	ε_v [%]	S [in]
Stratum I, Terrace Sand	28.0	28.0	14.0	121	15.8	104	0.85	133	0.9	0.4	47	0.5	14	1.45	20	0.9347	0.1319	0.00649	Lower than 0.01	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 1	71.0	24.0	12.0	122	24.1	98	0.80	126	3.8	1.8	183	1.9	89	0.74	66	0.6000	0.0883	0.00134	Lower than 0.01	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 2	94.0	23.0	11.5	123	30.5	94	0.80	122	5.2	2.6	253	2.6	24	0.63	15	0.6000	0.0889	0.00589	Lower than 0.01	0.00	0.0
Stratum IIb, Chesapeake Cemented sand, Layer 3	110.0	16.0	8.0	123	26.0	98	0.64	122	6.4	3.2	308	3.2	63	0.57	36	0.6000	0.0895	0.00249	Lower than 0.01	0.00	0.0
Stratum III, Nanjemoy Sand	411.0	108.0	54.0	127	29.1	98	1.00	130	20.7	11.1	919	9.6	75	0.33	25	0.6000	0.0972	0.00393	Lower than 0.01	0.00	0.0

NOTE:

(1) If the value of $CSR/(N_1)_{60}$ is lower than 0.01, the liquefaction is not likely to occur and is Volumetric Strain (ε_v) is considered as zero.

Item 4

Statistical assessment of the lateral variability was presented in the response to RAI 229[(a)]. Lower bounds for the parameters used in the settlement analysis were presented in that discussion. However, the response concluded that the use of lower bound soil properties at CCNPP Unit 3 is conservative, and did not necessitate a change to the use of pseudo-elastic analysis presented in Section 2.5.4 of COLA FSAR Revision 7. Further, the determination of tilt as a difference between the high and low revert models was unnecessary. The previous departure on NI tilt was removed from the COLA in the response to RAI 145 Question 03.08.05-02[b]. A settlement monitoring program has been described in COLA FSAR Section 2.5.4.10.2.2. In the case that the settlements measured in the construction are different than the expectations presented in the COLA, actions such as the following can be taken.

- Extension of dewatering efforts, as to increase the period of time that effective stresses below the foundations are maximized.
- Intentional delays in making connections (piping and conduit banks) between the Nuclear Island and the adjacent structures (particularly ESWB), as to minimize differential settlement after the connections are made.
- Re-sequencing of backfill operations, possibly even placing backfill on one side of the Nuclear Island in advance of that on the other side of the Nuclear Island.

Item 5, Part (a)

The reference for the statement given in the response is:

> Terzaghi, K., Peck, B.P., Mesri, G,. (1998) Soil Mechanics in Engineering Practice, Third Ed, Wiley and Sons, NY. page 292 sec 40.1.

In addition to the reference provided, additional analyses have been performed to assess the adequacy of the stress values beneath the building facilities. To support this discussion, a summary of soil parameters is provided in Table 5 and an idealized soil profile is provided in Figure 2. Note that the depth of the foundation is 41.5 ft.

The following elastic solution methods were studied in order to evaluate the effect of a multi-layer soil profile on the distribution of the vertical stress:

(i) Two layer solution for vertical stress distribution developed by Fox (1948). The site specific CCNPP Unit 3 soil profile was adapted to the methodology by reducing the number of layers with equivalent weighted values of the elastic modulus.
(ii) PLAXIS 2D Finite element solution.

Table 6 provides a comparison of the stress/load ratios at different elevations for both solutions along with the Boussinesq values. The agreement between the different approaches indicates that the Boussinesq solution can be used to estimate stresses for the settlement hand

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b G. Gibson (UniStar Nuclear Energy) to Document Control Desk ((NRC), "Response to Request for Additional Information for the Calvert Cliffs Nuclear Power Plant, Unit 3, RAI 145, Foundations," letter UN#11-085 dated February 22, 2011.

calculations at the CCNPP Unit 3 project. Note that the methodology that best adapts to site specific three dimensional layering is the FEM. The agreement between FEM and Boussinesq provides indication that the latter is appropriate for settlement verification purposes.

Table 5
Engineering Properties of Soils beneath the Foundation of the Nuclear Island

SOIL TYPE	LAYER THICKNESS [ft]	E_{50} [psf]
Stratum IIb -1	29	2.53E+06
Stratum IIb -2	20	1.03E+06
Stratum IIb -3	10	2.62E+06
Stratum IIc	190	2.37E+06
Stratum III	390	3.17E+06

Figure 2
Idealized Soil Profile and Load

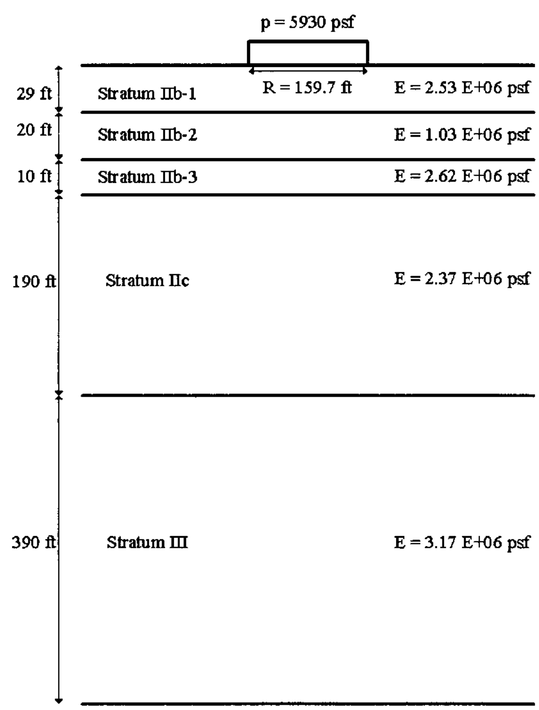


Table 6
Comparison of Vertical Stresses Obtained by the Different Methods

DEPTH [ft]	σ_{zi} / p		
	PLAXIS	BOUSSINESQ	FOX (1948)
29	0.98	0.98	0.96
49	0.96	0.96	0.95
249	0.44	0.41	0.48

References

Burmister, D.M., (1945) "Theory of stresses and displacements in layered systems and applications to the design of airport runways" Proc. Highway Res. Board, Vol. 23, pp. 127-148.

Burmister, D.M., (1962) "Application of layered system concepts and principles to interpretations and evaluations to asphalt pavement performances and to design and construction" Proc. Intr. Conf. on Structural Design of Pavements. Univ. of Michigan, Ann Arbor, Mich., pp. 441-453.

Das, B.M. (1985) Advanced Soil Mechanics. McGraw-Hill, NY.

Fox, L., (1948) "Computations of traffic stresses in a simple road structure" Proc. 2nd Int. Conf. Soil Mechs.Fndn.Eng., Vol 2., p. 236-246.

Johns, A., (1962) "Tables of stresses in three-layer elastic systems" High. Res. Board, Bull. 342, pp. 176-214.

Poulos, H.G., Davis, E.H., (1974) "Elastic solutions for soil and rock mechanics" John Wiles and Sons, NY.

Item 5, Part (b)

Table 7 presents the percent difference between the stresses obtained from the theoretical solution without any stiffness input and those from a layered soil finite element model (Plaxis 2D). Foundation pressure on the ground surface is considered as p = 5930 psf.

Table 7
Comparison of Vertical Stresses Obtained by Plaxis and Boussinesq Methods

DEPTH [ft]	VERTICAL STRESS [psf]		DIFFERENCE PLAXIS - BOUSSINESQ	
	PLAXIS	BOUSSINESQ	Δ	$\lvert \Delta / \sigma_{zi\ Bous} \rvert$ [%]
29	5895	5925	-30	0.5
49	5787	5805	-18	0.3
97	5213	5180	33	0.6
154	4140	3977	163	3.9
211	3153	2987	166	5.3
350	1626	1495	131	8.1

The results provided by Table 7 indicate that the theoretical solution is applicable to the hand calculations that are used for verification purposes.

COLA Impact

Section 2.5.4.10.2.2 will be updated in a future revision of the COLA as shown below.

2.5.4.10.2.2 Settlement and Heave Analysis in the Powerblock Area

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

. . .

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

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

2. *Settlement Representative of Medium Surface Elevation Zones:* The unloading/reloading modulus was used until the end of the third [Medium Elevation E Revert (1)] and fourth [Medium Elevation E Revert (2)] loading steps. These cases represent the stress-stiffness correspondence for the parts of the Powerblock Area with an initial pre-excavation ground surface of about El. 80 ft. These two cases cover the elevation range of most of the Powerblock Area.

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

RAI 268

Question 02.05.04-27:

Results of settlement sensitivity analyses show that:

1. Settlements computed using a non-linear Cam-Clay model calibrated to the available tri-axial stress-strain data are on the order of 4.2 feet (ft), as compared to 1.4 ft of settlement estimated using an elastic model.

2. When using the mean consolidation test data to develop Cam-Clay model parameters, settlements of the Nuclear Island are on the order of 2.2 ft. However, if lower bound (16th percentile) test data are used the computed settlements increase to about 5.5 ft.

Since the settlement sensitivity analysis results indicate that the potential differential settlements between buildings may be much greater than those estimated in the FSAR, discuss and justify the following regarding settlement of Category 1 structures and to ensure the stability of foundations at the CCNP site in accordance with 10 CFR 100.23:

1. Given the non-linear character of the soils at the Site (as evidenced by the tri-axial tests results), justify the adequacy of the soil models used in predicting settlements for the CCNP site in the FSAR.

2. Given the wide range variation of consolidation properties for the tested soils and a lack of data sufficient to establish a verifiable spatial correlation of the properties, provide an assessment of how large differential settlements will be incorporated into the design of the NI structures given the relatively small differential settlement allowances in the standard design.

3. Given the expected large settlements and potential large differential settlements, sequencing of the construction process will be critical to assuring the assumptions used in the standard design are valid for the Site. Provide a detailed discussion of the construction sequencing that will be used to assure that the design basis contained in the standard design is maintained based on the site-specific settlement analysis.

4. Recognizing that actual settlements at this site are likely to be highly variable when compared to the settlements estimated prior to construction, settlement monitoring is essential. Please discuss why the proposed settlement monitoring program is sufficient, provide a detailed description of the actions required to evaluate measured settlements if they are inconsistent with the predictions; and discuss potential impacts and actions to the construction sequencing due to settlements that exceed predictions.

Response

Items 1 and 2

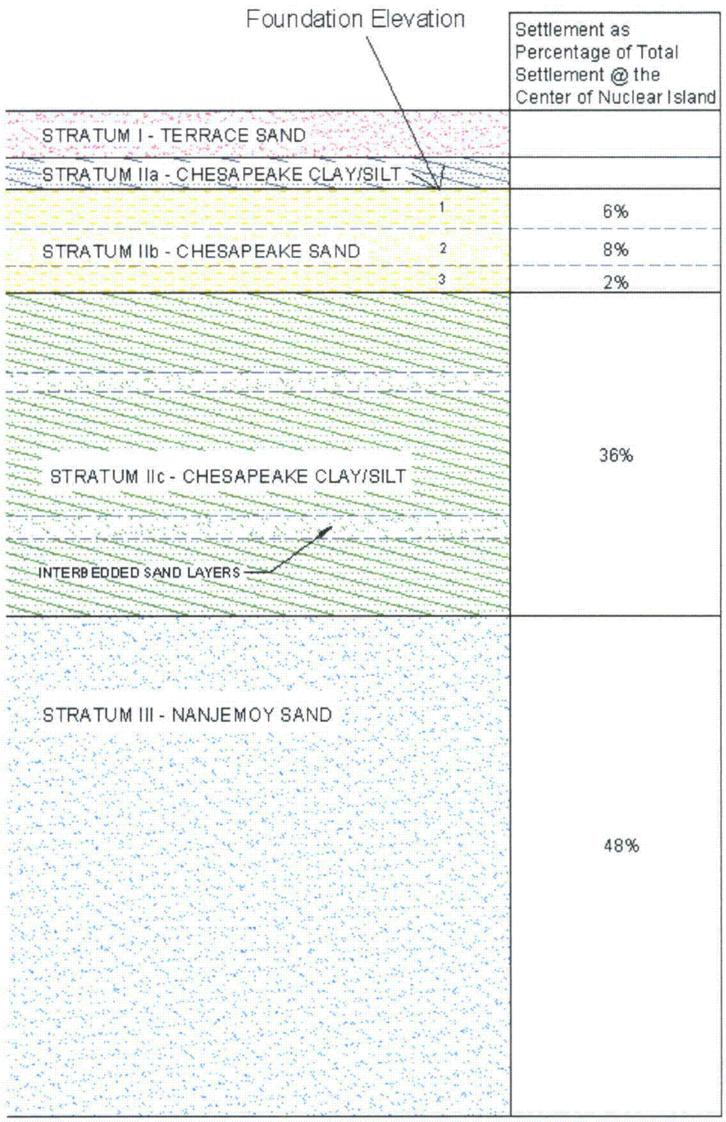
There are three generally accepted approaches for developing soil properties and geo-mechanical models for estimating settlement of deep profiles (such as exists at the CCNPP site) as follows:

- Use consolidation test data and constitutive relationships such as that generally known as the Terzaghi Theory of Consolidation for clays, and one of several empirical models for sand usually derived from field tests (e.g., SPT or CPT). The approach requires a wide range and a large number of consolidation tests (oedometer). Sample disturbance can be an issue, as well as the presence of silt or sand lenses in the clay, as the interpretation of the test results for use in a settlement analysis can be problematical.
- Use triaxial test data in a constitutive model such as a Cam-Clay Model that explicitly accommodates non-linear behavior of soils. This approach requires a wide range and a large number of triaxial tests conducted on truly undisturbed samples. Sands should not be cemented or over-consolidated, as these characteristics are often lost during sampling. Clays should not be sensitive and should be able to withstand the impacts of sampling, transport, handling, and storage.
- Use field data from in-situ tests such as available from SPT or CPT data or shear wave velocity measurements or pressuremeter tests to develop pseudo-elastic consolidation parameters. This approach circumvents issues with cementation and over-consolidated sands, sample disturbance with clay, effects of silt, and sand lenses in clay horizons in addition to better addressing the in situ stress conditions and soil fabric. On the other hand, non-linear behavior, if significant, is not as easily addressed.

All three approaches were considered and, after an assessment of the test results (specifically the consolidation test results, the triaxial test results and the in-situ test results), the third approach was chosed as the best methodology for assessing settlement at the CCNPP Unit 3. The following paragraphs discuss the logic and the laboratory test data and serve to explain the variability in predictions of settlement discussed in the staff's comments.

Figure 3 below is a broad interpretation of the soil profile at the CCNPP site and is used solely as a reference when considering the various triaxial and consolidation tests.

Figure 3
Soil Profile and Approximate Settlement Percentages by Layers

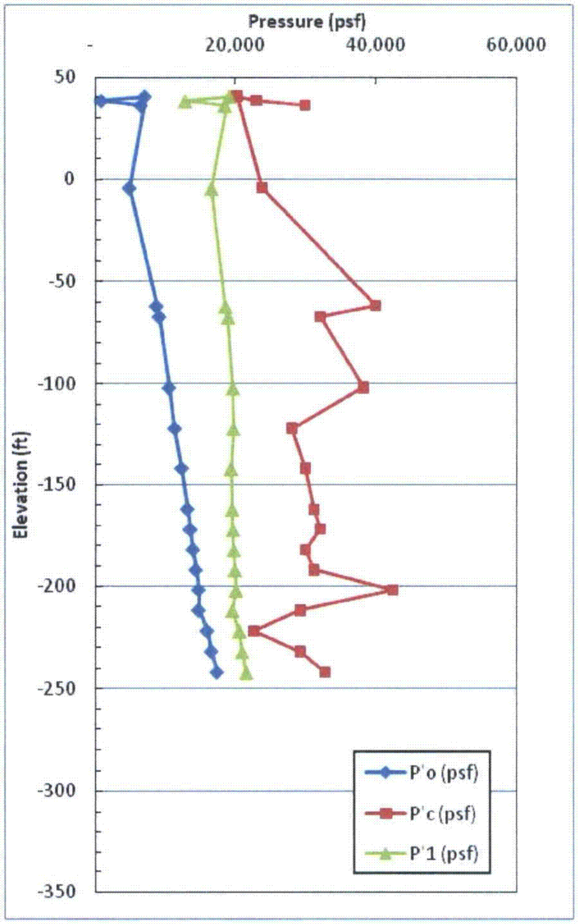


The right side of Figure 3 provides the rough percentage of the total settlement attributed to Stratum IIb, Stratum IIc, and Stratum III. It is noted that about 84 percent of the settlement is associated with consolidation of the Stratum IIc - Chesapeake Clay/Silt and the Stratum III - Nanjemoy Sand. Therefore, the focus is the best models applicable to these two Strata, the field data for these two strata, and on the tests performed on samples obtained from these two strata.

STRATUM IIC – CHESAPEAKE CLAY SILT – CONSOLIDATION TEST DATA

Stratum IIc is a clay with silt or silty clay that is basically over-consolidated, probably by migrating dunes resulting in an overconsolidation ratio of at least two and possibly as high as five or higher, depending on which oedometer test is considered. The effect of the overconsolidation is important, both from a modeling perspective and from a property standpoint. Figure 4 shows the data points at each elevation for the existing overburden stress, the final stress after the foundation is in place and dewatered level is back to normal, and the pre-consolidation stress as "best estimated" from the oedometer consolidation tests. It is seen that the final stress is less than the pre-consolidation stress at practically all elevations. Thus, Stratum IIc will be loaded along an over-consolidated portion of an e-log p curve, as per the Terzaghi Consolidation Theory curve associated with an oedometer test. This implies a pseudo-elastic behavior, practically time-independent.

Figure 4
Stress Distribution by Depth



P'o = In situ overburden stress, P'c = Preconsolidation pressure, P'1 = Final stress after loading

To strengthen this point of pseudo-elastic behavior, Table 8 compares the Young's Modulus estimated with various approaches.

Table 8
Young's Modulus for Stratum IIc (psf)

E_{oed}, as determined from oedometer tests using Cr = 0.06 and e_o = 1.609 and average stress over time for sample (not used in Mohr Coulomb Analysis). Note that $E_{oed} = 1/m_v$ where $m_v = 0.435Cr/(1+e_o)\ \sigma_v$.(1)	1.49E+06
E, as determined from Vs, pressuremeter tests and empirical correlations with published undrained shear strength correlations. (Used in Mohr Coulomb Analysis for all steps if pre-construction in-situ stress exceeded when loading along Er portion of load-deformation curve.)	2.37E+06
Er, as determined by multiplying the Er/E ratio obtained from pressuremeter tests by the E determined from Vs, pressuremeter tests and empirical correlations with published undrained shear strength correlations (used in excavation and early loading steps in Mohr Coulomb Analysis).	7.11E+06
E_{oed}, as determined from oedometer tests using Cc = 0.947, e_o = 1.609, and average stress over time for sample (not used in Mohr Coulomb Analysis). Note that $E_{oed} = 1/m_v$ where $m_v = 0.435Cc/(1+e_o)\ \sigma_v$. (1) (Not used in settlement analysis because pre-consolidation stress not exceeded.)	9.41E+04

(1) σ_v = average of initial in situ vertical stress and the vertical stress after loading. Determined at every 10 ft increment, and then the average for the layer is calculated.

The CCNPP Unit 3 settlement analysis used an E value of 2.37E+06 psf as the best value to represent the behavior of Stratum IIc. In further support of the decision to rely on the field data to develop suitable parameters for settlement analysis, Table 9 presents a summary of the oedometer tests for samples of the soil extracted from Stratum IIc.

TABLE 9
Summary Assessment of Consolidation Data
Stratum IIc

BORING	ELEVATION [ft]		Gs	e_0	LL	Cc	Cr	P'c [tsf]	P'o [tsf]	OCR [ ]	SOIL	ε_{vo} @ σ'_{ov} (%)	$\Delta e/e0$ @ σ'_{ov}	OBSERVATIONS
	TOP	BOTTOM												
B-301	-64.00	-65.10	2.68	1.01	76.00	0.4850	0.0116	19.00	4.30	4.42	CH	2.0	0.040	Cc is relatively high but ok; Cr is reasonable
B-301	-74.00	-75.90	2.62	1.82	112.00	1.2756	0.0365	15.25	4.49	3.40	CH	2.5	0.030	Cc out of normal range; Cr is high but acceptable. High LL
B-304	-30.50	-31.50	2.65	1.03	79.00	0.5116	0.0083	20.00	5.63	3.55	SC	2.5	0.030	Cc high; Cr is high but acceptable Sample Quality Fair
B-304	-70.50	-71.30	2.65	0.95	43.00	0.2790	0.0083	16.00	7.89	2.03	SC	2.5	0.040	Good test with reasonable Cc and Cr Sample Quality Fair
B-313	-42.80	-44.00	2.69	1.07	49.00	0.3455	0.0116	14.00	5.49	2.55	CL	2.5	0.040	Good test with reasonable Cc and Cr Sample Quality Fair
B-313	-72.80	-73.60	2.67	1.07	49.00	0.4053	0.0050	22.00	7.21	3.05	SC	2.2	0.040	Acceptable test with acceptable Cc and Cr. Sample Quality Fair.
B-327	-26.60	-27.30	2.70	1.34	60.00	0.8737	0.0116	20.00	3.42	5.85	CH	2.5	0.038	Cc high; Cr is high but acceptable Sample Quality Fair
B-328	-47.20	-48.80	2.76	1.54	72.00	0.8670	0.0332	9.00	3.38	2.66	MH	2.5	0.026	Cc high; Cr is high but acceptable; Sample Quality Fair
B-401	-51.40	-52.70	2.65	1.74	85.00	1.1693	-0.0066	14.00	4.28	3.27	MH	2.5	0.030	Cc too high, Cr negative. High fines (82%). Sample Quality Fair; not an indicative test.
B-401	-86.70	-87.20	2.65	1.56	81.00	0.8870	0.0233	14.00	5.01	2.80	MH	2.5	0.038	Cc is relatively high but ok; Cr is reasonable Sample Quality Fair.

TABLE 9
Summary Assessment of Consolidation Data
Stratum IIc

BORING	ELEVATION [ft]		Gs	e_0	LL	Cc	Cr	P'c [tsf]	P'o [tsf]	OCR [ ]	SOIL	ε_{vo} @ σ'_{ov} (%)	$\Delta e/e0$ @ σ'_{ov}	OBSERVATIONS
	TOP	BOTTOM												
B-401	-101.40	-102.30	2.76	2.80	57.00	2.0463	0.0399	12.00	5.25	2.29	CH	6.0	0.050	Cc too high, Cr is somewhat high but acceptable. High fines (82%). Sample Quality Poor, not an acceptable test.
B-401	-171.40	-172.30	2.36	2.41	140.00	1.7739	0.0233	18.00	6.45	2.79	MH	1.5	0.017	Cc high. High content of water. Graph atypical. HIGH LL, not an acceptable test
B-423	-48.40	-50.00	2.70	1.46	74.00	0.7773	0.0100	9.25	4.61	2.01	OH	2.8	0.047	Cc out of normal range; Cr is high but acceptable. Sample Quality Fair.
B-423	-68.40	-69.70	2.63	1.71	64.00	0.8471	0.0532	6.75	5.10	1.32	SM	7.0	0.145	Cc and Cr are both high. Sample Quality Poor. Not an acceptable test
B-301A	-102.30		2.60	1.77	115.00	1.4616	0.0332	23.50	5.13	4.58	CH	3.0	0.050	Cc and Cr are both high. Sample Quality Fair. Considered as non-indicative. High LL
B-301A	-122.30		2.55	1.02	66.00	0.3602	0.0332	10.00	5.60	1.79	SC	3.5	0.078	Acceptable test with acceptable Cc and Cr. Sample Quality Fair.
B-301A	-142.30		2.53	1.28	96.00	0.6968	0.0440	22.50	6.02	3.74	CH	2.0	0.039	Cc and Cr are both high. Considered as non-indicative.
B-301A	-162.30		2.47	1.48	113.00	0.8039	0.1960	17.00	6.38	2.66	CH	1.5	0.270	Cc and Cr high. Graphic looks discontinuous. Considered non-indicative. High LL
B-301A	-172.30		2.34	2.06	182.00	1.2457	0.1080	14.50	6.51	2.23	CH	2.0	0.034	Cc and Cr are both high. Considered as non-indicative. High LL

TABLE 9
Summary Assessment of Consolidation Data
Stratum IIc

BORING	ELEVATION [ft]		Gs	e_0	LL	Cc	Cr	P'c [tsf]	P'o [tsf]	OCR [ ]	SOIL	ε_{vo} @ σ'_{ov} (%)	$\Delta e/e0$ @ σ'_{ov}	OBSERVATIONS
	TOP	BOTTOM												
B-301A	-182.30		2.32	2.43	133.00	1.5402	0.0861	28.00	6.63	4.22	CH	3.5	0.046	Cc and Cr are both high. Sample Quality Fair. Considered as non-indicative. High LL
B-301A	-192.30		2.48	1.46	115.00	0.8804	0.0581	22.50	6.82	3.30	CH	2.5	0.034	Cc and Cr are both high. Sample Quality Fair Considered as non-indicative. High LL
B-301A	-202.30		2.40	2.33	135.00	1.4949	0.0664	17.50	6.97	2.51	CH	3.0	0.040	Cc and Cr are both high. Sample Quality Fair. Considered as non-indicative. High LL

The following observations may be made from Table 9:

1. The oedometer tests highlighted in Green are considered to be good tests on samples of at least "fair" quality. The Cc values range from 0.279 to 0.51, the Cr values range from 0.005 to 0.02, and the pre-consolidation pressures range from 5.6 tsf to 7.89 tsf.

2. The oedometer tests shown in Yellow are judged as acceptable for assessing Cr and the pre-consolidation pressure, but are considered not indicative with respect to Cc. This is largely judged to be the result of sample quality.

3. The oedometer tests shown in Orange are judged to be not indicative and are considered to be less valuable that those highlighted in Green or Yellow.

4. The oedometer tests highlighted in Red are considered unacceptable and should be discarded either because of poor testing technique or excess disturbance.

Thus, only five laboratory tests are considered to be good tests.

The quality of the samples was judged on the basis work of Andresen and Kolstad (1979), who derive a relationship for sample quality on the basis of the volumetric strain ε_{vo} experienced by a sample, when it is consolidated back to the in-situ effective stress or σ'_{ov}. (Table 10)

TABLE 10
Sample Disturbance Criterion for Consolidation Tests

VOLUME CHANGE - ε_{vo}.	TEST SPECIMEN QUALITY
< 1%	Very Good to Excellent
1-2%	Good
2-4%	Fair
4-10%	Poor
>5%	Very Poor

As may be seen by comparing the sample volumetric strain in Tables 9 and 10, practically all of the samples are in the range of Fair with a few Poor Quality Samples.

On the basis of these observations, it is concluded that, for the Chesapeake Clay/Silt Stratum IIc, the consolidation test results should not be used in Terzaghi Consolidation Model for the CCNPP Unit 3 Site, and that the behavior is best represented by the in-situ tests as was done in the settlement analysis discussed in Section 2.5.4 of COLA FSAR Revision 7.

STRATUM III – NANJEMOY SAND – CONSOLIDATION TEST DATA

The Nanjemoy Sand is interpreted as predominantly SM and SC soil. Only the SC materials were tested in an oedometer, specifically four tests summarized in Table 11 as follows:

TABLE 11
Summary Assessment Consolidation Data Stratum III

BORING	DEPTH (FT)	e_o	Cc	Cr	P'c (tsf)	P'o (tsf)	OCR	OBSERVATIONS
B-301A	310	0.73	0.26	0.026	15	7.29	2.06	Results ok
B-301A	320	0.41	0.06	0.007	15	7.64	1.96	Results ok
B-301A	330	0.85	0.42	0/011	21	7.96	2.64	Results ok
B-301A	340	1.42	0.99	0.032	15.5	8.26	1.88	Cc is high, Cr is ok
Used For E_{oed}		1.00	0.53	0.045				

From the summary above, the four tests are reasonable, except that the Cc for the deepest sample is high. All tests indicate that the Nanjemoy is slightly over-consolidated with an OCR equal to about 2. The interpreted values that were used to estimate E_{oed} (associated with Cc) and E_{oed} (associated with Cr) in the estimates of Young's Modulus for Stratum III are as indicated in Table 12 below.

TABLE 12
YOUNG'S MODULUS FOR STRATUM III (psf)

E_{oed}, as determined from oedometer tests using Cr = 0.045, and e_o = 1.001, and average stress over time for sample (not used in Mohr Coulomb Analysis). Note that $E_{oed} = 1/m_v$ where $m_v = 0.435Cr/(1+e_o)\sigma_v$. [1]	1.94E+06 increasing with depth to 2.87E+06
E, as determined from Vs, pressuremeter tests and empirical correlations with published SPT correlations. (Used in Mohr Coulomb Analysis for all steps if pre-construction in-situ stress exceeded when loading along Er portion of load-deformation curve.)	3.17E+06
Er, as determined by multiplying the Er/E ratio obtained from pressuremeter tests by the E determined from Vs, pressuremeter tests and empirical correlations with published SPT correlations (used in excavation and early loading steps in Mohr Coulomb Analysis).	9.70E+06
E_{oed}, as determined from oedometer tests using Cc = 0.529, e_o = 1.001, and average stress over time for sample (not used in Mohr Coulomb Analysis). Note that $E_{oed} = 1/m_v$ where $m_v = 0.435Cc/(1+e_o)\sigma_v$. [1] (Not used in settlement analysis because pre-consolidation stress not exceeded.)	1.66E+05 increasing with depth to2.46E+05

[1] σ_v = average of initial in situ vertical stress and the vertical stress after loading. Determined at every 10 ft increment, and then the average for the layer is calculated.

In summary, the four oedometer tests for SC samples of "stringers" in the Nanjemoy Sand are acceptable, except possibly one where the Cc value is high. Nevertheless, if these four are considered indicative of the entire Nanjemoy formation, the deformation properties are in broad agreement with the deformation properties developed from in-situ testing, specifically, shear wave velocity measurements, pressuremeter tests, and SPT tests. Consequently, this re-affirms the interpretation that the use of a pseudo-elastic model, referred to as Mohr-Coulomb Model, is appropriate and yields reasonable, adequately conservative settlement estimates in the Nanjemoy Sand.

CHESAPEAKE CLAY SILT AND NANJEMOY SAND – TRIAXIAL TEST DATA

Table 13 is a summary of the triaxial test results performed for the project. It is noted that one test is available for Stratum IIc and none for the Nanjemoy Sand (Stratum III). It should also be noted that this sample is from a boring that was not drilled in the areas of the NI structures. The number of triaxial tests was limited by the number of good quality samples, which were difficult to obtain given the depth of sampling and the soil type.

TABLE 13
Summary of Results - E_{50} Modulus from Triaxial Tests (1)

BOREHOLE	DEPTH (ft)		E_{50} (1) (psf)	σ_3 (2) (psi)	E_{50} (1) (psf)	σ_3 (2) (psi)	E_{50} (1) (psf)
	TOP	BOTTOM					
B-320	38.5	40.5	1.90E+05	13	8.05E+05	25	9.62E+05
B-317	28.5	30.5	2.19E+05	20	2.20E+05	40	4.80E+05
B-317	48.5	50.5	5.07E+05	30	1.36E+06	60	2.56E+06
B-316	53.5	55.5	2.26E+05	20	9.70E+05		
B-414	68.0	70.0	1.74E+05	100	1.52E+05		
B-433	48.5	50.5	9.87E+04	60	1.30E+05		
B-328	63.5	65.5	2.18E+05	40	2.15E+05	65	8.85E+05
B-423	103.5	105.5	2.97E+05	55	7.58E+05	107.5	1.94E+06
B-321	73.5	75.5	2.48E+05	20	4.40E+05	80	1.23E+06
B-420	128.5	130.5	7.98E+05	120	3.21E+06		

NOTE:

(1) Secant modulus:

$$E_{50}=\frac{(\sigma_1-\sigma_3)_{50\%}}{\delta_{50\%}}$$

where

$(\sigma_1-\sigma_3)_{50\%}$ is the deviator stress at 50 percent of the ultimate load

$\delta_{50\%}$ is the axial strain at 50 percent of the ultimate load

(2) Confinement stress (total stress)

Recalling that the focus is on two layers, Stratum IIc and Stratum III, Table 14 shows the modulus values from the triaxial test for the sample obtained from Stratum IIc compared with the values derived from the field test measurements and used in our settlement analysis.

TABLE 14
Comparison Table
Young's Modulus for Stratum IIc (psf)

E_{50} from triaxial tests with Confining Stress = 30 psi for Statum IIc	7.98E+05
E_{50} from triaxial tests with Confining Stress = 120 psi for Statum IIc	3.21E+06
E as determined from Vs, pressuremeter tests, and empirical correlations with published undrained shear strength correlations. (Used in Mohr Coulomb Analysis for all steps if pre-construction in-situ stress exceeded when loading along Er portion of load-deformation curve.)	2.37E+06
Er, as determined by multiplying the Er/E ratio obtained from pressuremeter tests by the E determined from Vs, pressuremeter tests, and empirical correlations with published undrained shear strength correlations (used in excavation and early loading steps in Mohr Coulomb Analysis).	7.11E+06

The settlement analysis with pseudo-elastic parameters used an E value of 2.37E+06 psf, as this value best represents the behavior of Stratum IIc. This may be compared with the two values from the triaxial tests, which "straddle" the value used in the analysis for all steps, if pre-construction in-situ stress is exceeded when loading along the Er portion of the load-deformation curve.

Reiterating comments above, the pseudo-elastic analysis, as reported in the COLA FSAR Section 2.5.4, provides the best estimate of the settlement to be experienced by the CCNPP Unit 3 structures. Sampling difficulties of the over-consolidated Stratum IIc and Stratum III support using field in-situ data, as opposed to unacceptable consolidation tests (oedometer); and the lack of adequate triaxial test samples from deep sand in Stratum III and clay in Stratum IIc dissuade one from using laboratory test data. Furthermore, in situ tests have the advantage of maintaining the soil fabric and stress conditions in the field, which impacts the soil behavior.

The response to Item 4 of RAI 268 Question 02.05.04-26 (this letter) provides a discussion of the settlement predictions and the lower bound settlement model. That response, as do Items 3 and 4 below, outlines actions that could be taken if settlement varies from the predicted values.

References

Andresen A.A., Kolstad, P. (1979) "The NGI 54-mm Samplers for Undisturbed Sampling of Clays and Representative Sampling of Coarser Materials." International Symposium on Soil Sampling, Singapore, 13-21.

Item 3 and 4

We concur with the Staff that construction sequencing and schedule has a major impact on the settlement that will occur during construction and in the long term after construction is complete. The planned construction schedule has been factored into the analysis, but the industry experience during construction of the previous generation of plants suggests that deviations from the planned schedule can occur, primarily within the middle third of the overall planned schedule. Early tasks in the schedule usually meet target dates, tasks in the middle third can be delayed and tasks in the last third were extended. Delays in the middle third are of major interest to settlement analysis, whereas delays in the last third are not so important because the majority of loads, such as the weight of civil structures, are already in place.

During the construction program, the behavior of the structures and the pore pressures will be monitored with settlement monuments, telltales, and piezometers. Actual deformations, settlements, and pore pressures will be compared with predicted parameters derived from the three-dimensional, time-dependent (construction sequence dependent) analysis. Differences between the predicted and actual parameters could be associated with any or several of the following:

- Greater than expected spatial variation of soil properties
- Clay/Sand behavior inconsistent with properties used in the settlement analysis
- Deviations in the duration of construction tasks or sequence of tasks, including concrete placement and backfill placement
- Deviations in the schedule and sequence of the placement of major equipment loads such as the reactor vessel, steam generators, or steam turbine generator
- Deviations in the rate and duration of dewatering efforts

In the event that major differences occur, the following are the type of actions that could be implemented if settlement varies from the predicted model:

- Extension of dewatering efforts, as to increase the period of time that effective stresses below the foundations are maximized.
- Intentional delays in making connections (piping and conduit banks) between the Nuclear Island and the adjacent structures (particularly ESWB), as to minimize differential settlement after the connections are made.
- Re-sequencing of backfill operations, possibly even placing backfill on one side of the Nuclear Island in advance of that on the other side of the Nuclear Island.

COLA Impact

There are no changes to the CCNPP Unit 3 COLA as a result of this response.