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UniStar
NUCLEAR ENERGY

10 CFR 50.4
10 CFR 52.79

September 6, 2011

UN#11-245

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. 307, Stability of Subsurface Materials and Foundations

Reference: (1) Surinder Arora (NRC) to Robert Poche (UniStar Nuclear Energy), "FINAL RAI 307 RGS2 5741" email dated May 3, 2011

(2) UniStar Nuclear Energy Letter UN#11-240, from Greg Gibson to Document Control Desk, U.S. NRC, RAI Closure Plan, dated August 23, 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 May 3, 2011 (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.

A schedule for the response to RAI 307 Question 02.05.04-31 was provided in Reference 2. The enclosure provides our response to RAI No. 307, Question 02.05.04-31, 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.

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NRO

Our response does 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 September 6, 2011

Greg Gibson

Enclosure: Response to NRC Request for Additional Information RAI No. 307, Question 02.05.04-31, 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

GTG/AJF/mdf

Enclosure

**Response to NRC Request for Additional Information
RAI No. 307, Question 02.05.04-31,
Stability of Subsurface Materials and Foundations,
Calvert Cliffs Nuclear Power Plant, Unit 3**

RAI No. 307

Question 02.05.04-31

In response to RAI Questions 02.05.04-26 and -27, you provided additional details on your settlement analysis, including the models and parameters that were applied. In order for staff to complete its review to ensure the stability of foundations, and in accordance with 10 CFR 100.23, please provide the following additional information:

1. In the RAI response you state 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." COLA FSAR Revision 7, 2.5.4.2.5.3 "Performance Properties Under Static Conditions," states that "the selected values for the consolidation properties are based on average parameters obtained from laboratory testing." Please clarify how the consolidation property parameters were determined if consolidation test results were not used, and provide a justification for the parameter values used in the settlement analysis. In addition, please clarify whether you took the standard design lateral-uniformity requirement into consideration for soil layers underneath the foundation.

2. Although you state that the settlement will be monitored during construction and describe measures that will be taken in the event differences occur between actual and predicted settlement, you did not state whether these methods will control the anticipated large total settlement at the CCNPP Unit 3 site. Please clarify if these methods will control the predicted large total settlement. Also, please discuss how the proposed measures are related to the U.S. EPR standard design construction sequence requirement.

3. In the RAI response you state that 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. However, staff's confirmatory analysis showed a much larger settlement using a non-linear Cam-Clay model, which is generally considered to be a realistic model for clay-type soils, such as the Chesapeake Clay/Silt Stratum IIc. Please justify why a non-linear model was not considered in your settlement estimate for the CCNPP Unit 3 Site.

Response

Question Part 1:

In order to satisfy the documentation requirements of RG 1.206, the Combined License Application (COLA) Final Safety Analysis Report (FSAR) Revision 7 provides a summary of all laboratory testing results, including those from the consolidation tests. The input to the settlement analysis reported in COLA FSAR Revision 7 is based on a statistical and qualitative analysis of field and laboratory tests performed for the site. The consolidation parameters deduced from the field and laboratory test data were reviewed and, after an assessment of quality and adequacy of the consolidation test results, and the triaxial test results, the field test results were selected as the most representative for estimating settlements at the Calvert Cliffs

Nuclear Power Plant Unit 3 (CCNPP3) site. The justification for the parameters used in settlement analysis is provided in detail in the response to RAI 268 Question 02.05.04-27[1]. As stated in that response, sampling difficulties of the over-consolidated Stratum IIc and Stratum III, 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. This supports using field in-situ data, rather than the less representative consolidation tests (oedometer).

The standard design lateral uniformity requirement is taken into consideration, as addressed in the response to RAI 276 Question 02.05.04-29[2]. In that response, it is shown that shear wave velocity measurements taken from different locations in the Powerblock area are equivalent, and that for engineering analysis purposes the subsurface conditions are uniform.

Question Part 2:

The COL Item in Section 2.5.4.10.2 of U.S. EPR FSAR does not specify a total settlement criterion, but limits the differential settlement (tilt) to 0.5 inches per 50 feet. The tilt on the Nuclear Island (NI) common basemat is not expected to exceed 0.5 inches per 50 feet during or after the construction. However, slightly larger tilts may occur for the Emergency Power Generation Building (EPGB) and Emergency Service Water Building (ESWB). Larger tilts for these buildings are discussed and justified in Sections 3.8.5.5.2 (EPGB) and 3.8.5.5.3 (ESWB) of CCNPP3 FSAR Rev. 7. Measures are taken if the actual differential settlement exceeds the predicted values defined in Section 3.8 of CCNPP3 FSAR Rev. 7. Similarly, the measures provided in the response to RAI 268 Question 27[1] are taken if the angular distortions exceed the limits provided in the U.S. EPR FSAR.

The response to RAI 268 Question 02.05.04-27[1] states that, in the event that large differential settlements occur, measures are taken to control the differential settlements, such as extending the duration of dewatering, delaying connections, and rearranging backfill.

The impact control measures for larger than predicted total settlement include delaying the connections between the building(s), adjusting the plant grade, reassessing surface drainage features, and delaying connections to pipelines that connect the Powerblock with external facilities. Total settlement is continuously monitored during construction in order to promptly implement settlement control measures.

AREVA's response to U.S. EPR FSAR RAI 354, Question 3.8.5-22[3], defines an additional requirement for the angular distortion of the NI, EPGB and ESWB. The requirement stipulates that a COL applicant compare the estimated site specific angular distortions in the north-south and east-west directions with angular distortions obtained from the differential settlement contours provided in the response to U.S. EPR FSAR RAI 354 Question 3.8.5-22[3]. This comparison is made in the context of the construction steps defined in the response. The reconciliation for this requirement is included in Section 3.8 of the CCNPP3 FSAR and will be

[1] UniStar Nuclear Energy Letter UN#11-113, from Greg Gibson to Document Control Desk, U.S. NRC, RAI 268, Stability of Subsurface Materials and Foundations dated March 31, 2011

[2] UniStar Nuclear Energy Letter UN#11-119, from Greg Gibson to Document Control Desk, U.S. NRC, RAI 276, Stability of Subsurface Materials and Foundations dated March 30, 2011

[3] Response to Request for Additional Information No. 354, Supplement 20, U.S. EPR Standard Design Certification AREVA NP Inc., Docket No. 52-020, dated March 16, 2010

provided as part of the response to RAI 308 Question 03.07.05-08. Furthermore, site specific settlement measurements are compared against the U.S. EPR FSAR angular distortions in order to promptly implement control measures.

The differential settlement requirement is not defined for a specific construction sequence. The tilt on the CNNPP3 site is continuously monitored to ensure that the limit is not exceeded throughout the construction. The angular distortion requirement is closely monitored during construction. Angular distortions are calculated based on monitored settlements at different locations of the NI, the EPGBs, and the ESWBs, at the time steps provided in the response to RAI 354 Question 3.8.5-22[3]. The measured angular distortions are compared to those obtained from the differential settlement contour plots in the U.S. EPR FSAR.

Question Part 3:

As indicated by the Staff, soil behavior is typically non-linear. The ideal non-linear soil behavior assessment requires simulating the in situ soil fabric, stress states, and expected stress paths. Triaxial tests have been commonly employed to analyze soil behavior. The successful implementation of triaxial test data depends on either obtaining truly undisturbed samples or using laboratory specimen reconstitution techniques that mimic the natural deposition process. The response to RAI 268 Question 02.05.04-27[1] states that the pseudo-elastic analysis, as reported in the COLA Rev. 7, provides the best estimate of the settlement to be experienced by the CCNPP structures. Sampling difficulties of the over-consolidated Stratum IIc and Stratum III, 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.

Given the number of reliable samples, the triaxial or consolidation test results available for the CCNPP3 site are not suitable for settlement estimates regardless of the type of soil model used (pseudo-linear model, non-linear Cam Clay type soil model, or other type of non-linear models). The in situ tests are numerous and adequate to assess the behavior of soils at the CCNPP3 site.

The following text provides a justification of the use of a pseudo-linear model for the estimation of settlements at the CCNPP Unit 3 site.

As presented in RAI 268 Question 02.05.04-27, rather than the type of the soil model selected (linear vs. non-linear), additional evaluations indicate that the input parameters used in the settlement analysis are of greater impact on the predicted settlement. This is illustrated by comparing the estimates of the Cam Clay type non-linear model settlements with the results from the MT2 model calibrated to the triaxial test data, and also by comparing the non-linear soil hardening model with the MT2 model, both calibrated to the field test results.

As stated in the response to RAI 268 Question 02.05.04-27[1], the most critical soil layers in terms of contribution to settlement are Stratums IIc and III. One triaxial sample is available for the Stratum IIc (Boring B-420 at a depth of 128.5 ft), while none is available for Stratum III. The samples are subjected to two different confining pressures, σ_3=30 psi and σ_3=120 psi. For settlement estimation purposes, it is adequate to assign E_{50} corresponding to σ_3=30 psi to Stratums IIb and IIc, and E_{50} corresponding to σ_3=120 psi to Stratum III, where E50 is the secant modulus corresponding to 50 percent of the failure stress. If the pseudo-elastic model described in CCNPP3 FSAR Rev. 7 (MT2 model) is used with the properties from the triaxial test data, and the unload/reload modulus is not considered, the estimated maximum settlement at the NI is consistent with the observations made by the Staff in the question statement of RAI 268 Question 02.05.04-27. This reinforces the notion that the triaxial test data is faulty and not adequate for the purposes of linear or non-linear settlement calculations.

Thus, the settlement estimate for CCNPP3 depends more on the input parameters than on the selection of soil hardening, Cam Clay or Mohr-Coulomb constitutive models. The response to RAI 268 Question 02.05.04-27[1] included a comprehensive evaluation of test results and justification of the input parameters used in the analysis.

In addition, the following section provides comparison of the non-linear soil hardening and the Mohr-Coulomb models with similar modulus input.

The Non-Linear Soil Hardening (SH) Model is described in detail in the response to RAI 229 Question 02.05.04-17[4]. The response provides the features of a non-linear soil hardening model (SH Model). This model simulates the site-specific rolling topography of the ground surface. The parameters used for the SH Model are provided in Table 1. The descriptions of the input parameters are as follows:

E_{50}: Secant modulus corresponding to 50% of the failure stress,
E_{oed}: Oedometer modulus,
E_{ur}: Unload/reload modulus,
m: Factor accounting for the stress dependency of the modulus (0.5 for all layers).

The model requires all of the moduli to be provided for a given reference effective minor principal stress. E_{50} is assigned as the average E used for the MT2 (pseudo-elastic) model at the mid-elevation of each soil layer. This modulus changes with respect to both the confinement stress (depth) and the strain level. The results provided in Table 2 indicate that the non-linear SH model provides less settlement and tilt. Therefore, the MT2 Model is conservatively adopted. The comparison of the results from the SH Model (non-linear) to those from the MT2 Model (pseudo-linear) shows that the predicted settlement is not significantly different when using similar moduli as input.

The settlement analysis for the CCNPP3 site involves a pseudo-elastic approach with a Mohr-Coulomb model. Given the comprehensive evaluation of the field and laboratory testing data, the elastic moduli, the in situ stress conditions, the stress paths considered in the analysis, and the fact that similar results are obtained when using non-linear and linear models with equivalent input, it is concluded that the Mohr-Coulomb model used to predict the CCNPP3 settlement is adequate and provides the best settlement estimate.

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[4] UniStar Nuclear Energy Letter UN#10-207, from Greg Gibson to Document Control Desk, U.S. NRC, RAI 229, Stability of Subsurface Materials and Foundations dated July 23, 2010

Table 1, Soil Properties Used for Soil Hardening Model

LAYER	D/U	E_r^{ref} [psf]	E^{ref} [psf]	OCR	Reference Stress [psf]
Backfill	Drained	5.80E+06	1.92E+06	1.0	1440
IIb - Chesapeake Cem. Sand *(1)*	Drained	7.59E+06	2.53E+06	10.0	3568
IIb - Chesapeake Cem. Sand *(2)*	Drained	4.64E+06	1.03E+06	2.8	4580
IIb - Chesapeake Cem. Sand *(3)*	Drained	1.02E+07	2.62E+06	17.7	5213
IIc - Chesapeake Clay/Silt	Undrained	7.10E+06	2.37E+06	3.2	7941
IIc - Chesapeake Clay/Silt-Sand	Drained	7.10E+06	2.37E+06	4.7	7941
II - Nanjemoy Sand	Drained	9.80E+06	3.17E+06	1.9	16231

Table 2, Comparison of Settlement Results from Pseudo-Linear vs. Non-Linear Models

MODEL	ESTIMATED SETTLEMENT AT CENTER OF CONTAINMENT [in]	ESTIMATED MAXIMUM TILT FOR THE NUCLEAR ISLAND [in/50']
Pseudo-Linear (Mohr-Coulomb)	12.7	0.32
Non-Linear (Soil Hardening)	9.4	0.03

COLA Impact

FSAR Section 2.5.4.2.5.3 is being updated as follows:

Section 2.5.4.2.5.3 Performance Properties for Settlement Analysis under Static Conditions

The required performance properties under static conditions are the following:

- ~~Cr - Recompression index,~~
- ~~Cc - Compression index,~~
- ~~eo - Initial void ratio,~~
- p'c - Preconsolidation pressure,
- OCR - Overconsolidation ratio,
- ~~c_v - Coefficient of consolidation.~~
- ~~k - Permeability (hydraulic conductivity),~~
- k - Permeability (hydraulic conductivity).

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

Elastic modulus (E), unload/reload modulus (Eu/r), and field measured permeability (K) are used to determine the immediate and time-dependent settlement.

Sampling difficulties of the over-consolidated Stratum IIc and Stratum III justify the use of field in-situ data. The compromised triaxial test samples from deep sand in Stratum III and clay in Stratum IIc preclude the use of laboratory test data to estimate settlement. Furthermore, in situ tests have the advantage of maintaining the soil fabric and stress conditions in the field, which impact the behavior of the soils. Therefore, the field test results rather than consolidation and triaxial test results are used for estimating settlement at the CCNPP Unit 3. On the other hand, the over consolidation ratios (OCR) are conservatively assigned from the laboratory consolidation test results. For completion purposes and in order to satisfy the documentation requirements of RG 1.206, Table 2.5-53 provides the consolidation and permeability test results for each stratum, even though some of these properties are not used in the analysis.

Permeability is obtained from well field tests and development and calibration of hydrogeologic models. Details of the tests and models are provided in Sections 2.4.12 and 2.4.13. Hydraulic conductivity for backfill is based on laboratory results of tests performed on bulk samples.

FSAR Section 2.5.4.2.5.4 is being updated as follows:

Section 2.5.4.2.5.4 Strength Properties under~~Under~~ Static Conditions

The required strength properties under static conditions are the following:

- N - Standard Penetration Test (SPT) Resistance (N);
- c' - Cohesion under drained conditions;
- Φ - Friction angle under drained conditions;
- c - Cohesion under undrained conditions;
- Φ - Friction angle under undrained conditions;
- su - Undrained shear strength.

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

~~The shear strength parameters are based on laboratory testing data. Table 2.5-54 provides the strength properties for each stratum.~~ For completion purposes and in order to satisfy the documentation requirements of RG 1.206, Table 2.5-54 provides the strength properties according to the laboratory test results for each stratum.

FSAR Section 2.5.4.2.5.5 is being updated as follows:

Section 2.5.4.2.5.5 Elastic Properties ~~Under~~under Static Loading

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

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

The elastic moduli significantly impact settlement estimates and therefore numerous methods have been applied to estimate these parameters. They are determined based heavily on field tests as discussed in Section 2.5.5.2.5.3.The Shear modulus (G) and elastic modulus (E) are estimated for each soil strata using the following three criteria:

FSAR Section 2.5.4.10.2.2 is being updated as follows:

Section 2.5.4.10.2.2 Settlement and Heave Analysis in the CCNPP Powerblock Area

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

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

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

Two separate models were developed for the CCNPP Powerblock Area:

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

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

Heave Analysis: Excavation and Dewatering (ED Model)

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

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

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

- Zone I: low areas North East (Plant Local Coordinate System) with an average ground elevation of 60 ft;

- Zone II: South areas (Plant Local Coordinate System) with an average ground elevation of 80 ft;

- Zone III: high areas with an average ground elevation of 105 ft.

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

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

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

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

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

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

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

1. During Step 1, backfill is placed between El. 41.5 ft and El. 48 ft.

2. During Step 2, additional backfill is placed between El. 48 ft and El. 61 ft.

3. During Step 3, additional backfill is placed between El. 61 ft and El. 66 ft. An Excavation and Dewatering Model (ED Model).

4. During Step 4, additional backfill is placed between El. 66 ft and El. 76 ft.

5. During Step 5, additional and final backfill is placed between El. 76 ft and El. 83 ft.

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

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

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

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