# STABILITY CHECK FOR NI COMMON BASEMAT

## Technical Report

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## ABSTRACT

This technical report contains the nuclear island (NI) common basemat stability check, including an explanation of how the finite element models for the NI common basemat analysis are developed. The basemat uplift, settlement, and stability evaluation against overturning, sliding, and floatation are also described.

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## List of Acronyms

| | |
|---|---|
| AB | Auxiliary Building |
| FE | Finite Element |
| FOS | Factor of Safety |
| GDC | General Design Criteria |
| IBC | International Building Code |
| IRWST | In-containment Refueling Water Storage Tank |
| NI | Nuclear Island |
| PSW | Primary Shield Wall |
| RCB | Reactor Containment Building |
| SPT | Standard Penetration Test |
| SSI | Soil-Structure Interaction |
| SSW | Secondary Shield Wall |
| TGB | Turbine Generator Building |

# 1.0 INTRODUCTION

The purpose of this technical report is to present the stability check for the Nuclear Island (NI) common basemat.

The NI common basemat consists of the reactor containment building (RCB) base area and auxiliary building (AB) base area structures. The RCB is structurally separate from the AB with a seismic gap of 2 in above the common basemat. The RCB is a seismic Category I structure composed of a pre-stressed concrete cylindrical shell with a hemispherical type dome and reinforced concrete internal structures. The AB wraps around the RCB leaving a space for a seismic gap above the common basemat and is a seismic Category I structure. The AB consists of reinforced concrete shear walls and slabs that constitute a lateral load-resisting system.

The NI common basemat is a reinforced concrete mat foundation with an area of approximately 99,180 $ft^2$ (348 ft $\times$ 285 ft). The thickness of the AB basemat is 10 ft. The thickness of the RCB basemat varies from 10 ft at the center to 33 ft at the side, except for transient areas such as the tendon gallery and reactor cavity. The NI common basemat is embedded to a depth of 55 ft below the nominal plant grade of El. 100 ft 0 in. The bottom of the foundation is at El. 45 ft 0 in. Figure 1-1 is a plan view of the APR1400 basemat. Figures 1-2 and 1-3 show cross-sectional views at the containment centerline.

This technical report consists of five sections. Section 1 is an introduction with background information. Section 2 describes the site profiles for the APR1400 NI common basemat. Section 3 presents the modeling process of the finite element (FE) model for the NI common basemat analysis. Section 4 describes the stability evaluation of the NI common basemat. Section 5 presents the construction sequence analysis of the NI common basemat.

## 2.0 SITE PROFILES FOR THE APR1400 NUCLEAR ISLAND COMMON BASEMAT

This section describes the generic site profiles for the APR1400 NI common basemat.

### 2.1 Shear Wave Velocities of APR1400 Sites

The APR 1400 is designed with a standard design concept to enable construction on various foundation conditions enveloping rock and soil foundations. The generic sites for the APR1400 include nine site categories (S1~S9) that represent a variety of characteristics and configurations of rock and soil foundations as well as one fixed case. Figure 2-1 shows the profile of the shear wave velocities of the nine generic site categories. As shown in Table 2-1, unit weight and Poisson's Ratio corresponding to shear wave velocity are used to evaluate each site property (References [1] and [2]). Table 2-2 shows the soil and rock definition by shear wave velocity based on the International Building Code (IBC) (Reference [7]).

### 2.2 Review on the Elastic Modulus of Generic Sites

#### 2.2.1 Elastic Modulus of Soil Sites

In accordance with IBC, the $N$ value from the standard penetration test (SPT) in the ground with a shear wave velocity $V_s$ = 600~1,200 ft/sec is 15 < $N$ < 50. Therefore, where $V_s$ =1,000 ft/sec, the $N$ value can be interpolated as follows:

$$N = 15 + (1{,}000-600) / (1{,}200-600) \times (50-15) = 38$$

In addition, the relationship between the $N$ value and $V_s$ is described in Zen et al. (1987) (Reference [8]) as follows:

$$V_s = 89.1 \times (N)^{0.34} \text{ m/sec}$$

Where $V_s$ =1,000 ft/sec (= 304.8 m/sec), the $N$ value can be calculated as $N$ = 37. Based on the results from IBC and Zen et al. (1987), the range of $N$ values at $V_s$ =1,000 ft/sec is 37 ~ 38.

The relationship between the static elastic modulus ($E_{static}$) and the $N$ value is provided in Bowles (1982) (Reference [9]) as follows (Refer to Table 5-5 in Reference [9]):

$$E_{static} = 18{,}000 + 750 \times N \text{ (kPa)}$$
$$E_{static} = (15{,}200 \text{ to } 22{,}000) \times \ln N \text{ (kPa)}$$

Where $N$ = 37 ($V_s$ =1,000 ft/sec, minimum value), the static elastic modulus is obtained as $E_{static}$ = 45,750 kPa, 54,885 kPa, and 79,440 kPa from the relationship between $E_{static}$ and $N$, respectively. Therefore, the mean static elastic modulus can be determined as $E_{static}$ = 60,025 kPa = 60 MPa = 1,253 ksf.

In addition, the relationship between the maximum dynamic elastic modulus ($E_{dynamic}$) and $V_s$ is as follows:

$$E_{dynamic} = (\gamma / g) \times (V_s)^2 \times [2 \times (1+ \nu)]$$

where, $\gamma$ is unit weight, $\nu$ is Poisson's ratio, and g is gravity acceleration. Where $V_s$ =1,000 ft/sec, $\gamma$ = 125 pcf, and $\nu$ = 0.4, the dynamic elastic modulus is $E_{dynamic}$ = 10,860 ksf = 520 MPa. The relationship between $E_{static}$ and $E_{dynamic}$ at the soil site is $E_{static}/E_{dynamic}$ = 0.1153.

The APR 1400 generic sites are classified as the soil foundation where the shear wave velocity ($V_s$) is less than 1800 ft/sec, and the static elastic modulus ($E_{static}$) is obtained from shear wave velocity ($V_s$) using the relationships defined in this subsection.

### 2.2.2 Elastic Modulus of Rock Site

The dynamic elastic modulus ($E_{dynamic}$) of rock foundation is obtained from the relationship between $E_{dynamic}$ and $V_s$ in the same way as the soil foundation. In the rock foundation, the static elastic modulus can be calculated as the relationship between static and dynamic elastic moduli of rock.

Figure 2-2 shows the relationship between the static and dynamic elastic moduli of rock ($E_{static}/E_{dynamic}$). $V_P$ and $V_L$ in Figure 2-2 denote the compressional wave velocity in the field and the indoor sound test wave velocity, respectively (Deere, 1966) (Reference [10]). The reduction factor $\beta$ is the ratio of static elastic modulus to the dynamic elastic modulus ($E_{static}/E_{dynamic}$). According to Figure 2-2, at soft rock with a low rock quality ($[V_P / V_L]^2$ or rock-quality designation), $\beta$ is approximately 0.15 ~ 0.2. At relatively hard rock with a rock quality greater than 0.6, $\beta$ is approximately 0.3.

The relationship between the maximum dynamic elastic modulus and shear wave velocity at a rock site is identical to that provided for soil. The static and dynamic elastic moduli of soft rock ($V_s \leq$ 2,500 ft/sec) and relatively hard rock ($V_s >$ 2,500 ft/sec) are 0.15 and 0.3, respectively.

## 2.3 Material Properties and Subgrade Modulus of Site Profiles for the APR1400

The material properties according to depth from the ground level of the site profiles for the APR1400 are obtained from Sections 2.1 and 2.2 and can be found in the basemat analysis calculation (Reference [3]). In the tables in Section 1 and 2 (Reference [3]), the material properties are provided according to the depth from the ground level. Among the nine site categories and fixed case, S1, S4, and S8 are considered in the analysis. S1, S4, and S8 denote weak, moderate, and strong site properties, respectively. The analyses used to evaluate the site cases enveloped all of the site categories considered for the APR1400.

The subgrade moduli of three site profiles are obtained from an ANSYS analysis. The site properties used in the ANSYS ground model with 11 layers are based on the basemat analysis calculation and are shown in Table 2-3. For subgrade moduli, a unit pressure of 1 ksf is applied to the ground model and the maximum deformation is calculated. For the horizontal subgrade moduli of site, the two thirds of the maximum horizontal deformation is used. Figure 2-3 shows the deformation contour of ground models. Table 2-4 shows the subgrade moduli of site profiles that are obtained from the ANSYS analysis.

## 3.0 APR1400 NUCLEAR ISLAND COMMON BASEMAT ANALYSIS MODEL

This section presents the FE modeling process for the NI common basemat analysis.

### 3.1 General

The following considerations are made for the FE model development methodology for the APR1400 NI structures:

- Among the nine site profiles and the fixed case defined in the project, S1, S4, and S8 are considered in the NI common basemat analysis. S1, S4, and S8 denote weak, moderate, and strong site properties, respectively. The analyses used to evaluate the site cases enveloped all of the site categories considered for the APR1400.
- The NI common basemat is analyzed using a solid-element model for the entire mat including the tendon gallery using a FE method because of the non-symmetric features of the NI structures.
- The FE model for the superstructures including the RCB shell and dome, RCB internal structure, and AB structure are connected to the solid basemat model in order to simulate the stiffness effect of the superstructures to the basemat.
- The NI common basemat model considers the interaction of the basemat with the ground such as the possible basemat uplift from the ground. The LINK elements for the compression-only (gap) condition are used for the ground model.

The purpose of the model development is to create a three-dimensional FE model for the APR1400 NI common basemat analysis, which includes the RCB shell and dome, RCB internal structure, and AB structure. The three-dimensional FE analysis is carried out using the ANSYS program.

### 3.2 Development of Finite Element Models for Nuclear Island Structures

#### 3.2.1 Geometry and Coordinate System

The coordinate system for the model development of NI structures is the rectangular Cartesian coordinate system as follows.

- X (1): Along plant East-West, positive toward East side.
- Y (2): Along plant North-South, positive toward North side.
- Z (3): Positive vertical up.

The units used in the model are as follows:

- Length : ft
- Force : kip
- Stress : ksf
- Weight per unit volume : kcf

### 3.2.2 Material Properties

Linear-elastic material properties of concrete including modulus of elasticity, Poisson's Ratio and mass density are used in accordance with design criteria for the APR1400. The material properties of the NI structures are summarized in Table 3-1.

### 3.2.3 Finite Element Model

The NI structure is modeled using the following ANSYS program shell, solid, beam, and link elements:

- NI common basemat: SOLID185 elements
- RCB shell and dome: SOLID185 elements
- In-containment refueling water storage tank (IRWST) and fill concrete: SOLID185 elements
- Primary shield wall (PSW): SOLID185 elements
- Secondary shield wall (SSW): SHELL181 elements
- AB concrete wall and slab: SHELL181 elements
- AB steel column and girder: BEAM4
- Nonlinear ground (compression only): LINK180

The nominal element size in the NI common basemat is approximately 5 feet. Figure 3-1 shows the full FE model for the basemat structural analysis. In addition, the AB structure, RCB internal structure, RCB shell and dome, and basemat structure analysis models are shown in Figures 3-2 ~ 3-5, respectively.

### 3.2.4 Boundary Condition

Link (LINK180) elements are used for boundary conditions between the basemat structure and ground in order to consider the compressive behavior of the underlying subgrade. The LINK180 element is a uniaxial tension-compression element with three degrees of freedom for translation in the nodal x, y and z directions at each node. It is useful to describe the tension-only (cable) and/or compression-only (gap) condition.

Figure 3-6 shows the LINK180 element application as the boundary condition. The compression-only option is applied to the LINK180 elements connected direction with basemat structure and fixed boundary condition are applied to the other end side node of LINK180 element. Axial (tributary) areas of LINK180 elements are calculated by applying unit pressure to additional modeled shell element models that have the same geometry as the basemat model. Figure 3-7 shows the analysis model for the tributary area calculation.

### 3.2.5 Applied Loads

The applied loads analysis considers dead loads, live loads, post-tension loads for tendons embedded in the RCB shell and dome, containment pressure loads, pipe break load, seismic load, and buoyance load due to ground water.

The dead load of the NI common basemat is calculated by applying the vertical acceleration to the

basemat structure. Self-weight of the FE model is calculated in the ANSYS analysis automatically.

Reaction forces for dead loads, live loads, post-tension loads of tendon embedded RCB shell and dome, containment pressure loads, pipe break load, seismic load, and buoyance load calculated from the analysis results (References [4~6]) for each superstructure are applied as nodal force to the basemat structure model. Figures 3-8 ~ 3-10 show the application of typical nodal force for basemat structural analyses.

The reactions from seismic analyses of the RCB shell and dome, RCB internal structure, and AB building are applied as the seismic loads in the basemat model. The response spectrum analysis is used for the RCB shell and dome and RCB internal structure and the equivalent static analysis is used for the AB for seismic analyses of superstructures. In the response spectrum analysis, the maximum values of individual modes occur simultaneously; hence, the combined effect is obtained by using algebraic (considering signs) summation of the individual modal responses. Each directional reaction force at an arbitrary point corresponding to each seismic excitation from the seismic analysis reactions of superstructures is expressed as shown in Table 3-2. The responses due to the effect of the three directions of seismic motion are combined using the 100 percent ─ 40 percent ─ 40 percent (100-40-40) method in which one component is taken at 100 percent of its maximum value and the others are taken at 40 percent of their maximum values. In addition, assuming that each directional reaction has maximum value when the effect of the same directional seismic excitation is taken at 100 percent, the seismic load cases can be expressed as shown in Table 3-3. Finally, considering the interaction among the superstructures (e.g., RCB shell and dome, RCB internal structure, AB building), the seismic load combinations for the NI common basemat are expressed as shown in Table 3-4.

### 3.2.6 Load Combinations

The design base of RCB and AB base structures conform to the requirements of the ASME Code (Reference [11]) and ACI Code (Reference [12]), respectively. The boundary of jurisdiction between the ASME Code and ACI Code is shown in Figure 3-11. Because the design criteria are different, the application of load combinations for the NI common basemat is divided into two parts. Figure 3-12 represents the application of load combinations based on the code criteria. As shown in Figure 3-12, the load combinations provided by ASME and ACI are used for the analysis and design of the RCB and AB basemats, respectively. The division of the basemat by code jurisdiction at the thickness transition is a logical choice, and the boundary of the code jurisdiction is conservatively designed using the greater forces from the analysis results of ASME and ACI Codes.

Load combinations and load factors for the RCB and AB basemats are selected based on the relevant design code as shown in Figure 3-12. The five loading combinations (i.e., test, normal, severe, abnormal, and abnormal/extreme environmental conditions) are selected as the critical loading combinations in the NI common basemat analysis. The abnormal/extreme environmental loading combination includes the seismic load. Therefore, the load combinations are conservatively divided into the eight cases to account for the possibility of the phase behaviors as described in Section 3.2.5. Therefore, the load cases for the NI common basemat could be summarized as shown in Table 3-5. A total of 45 load combinations (15 combinations $\times$ 3 site profiles) are examined for the APR1400 NI common basemat.

# 4.0 STABILITY EVALUATION OF THE NUCLEAR ISLAND COMMON BASEMAT

This section presents the stability evaluation of the APR1400 NI common basemat against overturning, sliding, and flotation, and an evaluation of the settlement of NI common basemat.

## 4.1 Settlement of the Nuclear Island Common Basemat

### 4.1.1 Basemat Uplift

This section presents the uplift check of the NI common basemat during seismic excitation. According to SRP 3.7.2 (Reference [13]), the calculation of the ground contact ratio to provide reasonable assurance that the linear soil-structure interaction (SSI) analysis remains valid is required. The ground contact ratio is defined as the minimum ratio of the foundation area in contact with the ground to the total area of the foundation. The linear SSI analysis methods are acceptable if the ground contact ratio is equal to or greater than 80 percent.

Among the results from the NI common basemat analysis, the three load combination cases of LC08, LC10, and LC12, which are shown to have the uplift phenomenon, are considered for uplift check. Figures 4-1 ~ 4-3 show the deformation contour of the AB basemat in accordance with S1, S4, and S8. Table 4-1 shows the uplift area ratios of the NI common basemat. The APR1400 NI common basemat contact area during basemat uplift is 80 percent or greater.

### 4.1.2 Differential Settlement

Checks of the differential settlements of the NI common basemat are presented in this section. The differential settlements are divided by the differential settlement within the NI common basemat and the differential settlement between the NI basemat and other buildings.

For the differential settlements by static loading, the dead and live loads (D+L) are applied in the basemat. The node locations to check the settlement are determined based on the deformation results of the NI common basemat (see Figures 4-1 ~ 4-3). In addition, the nodes within a distance of approximately 50 feet are selected to check the differential settlement. Figure 4-4 shows the description and node location at the bottom of the NI common basemat for checking the settlement. Table 4-2 shows the differential settlements at S1, S4, and S8. The maximum differential settlements per 50 feet for S1, S4, and S8 are 0.176, 0.072, and 0.037 inches, respectively.

For the differential settlements by seismic loading, the displacement results from the seismic analysis calculation are used (Reference [2]). In the seismic analysis, the displacements of the basemat relative to the free-field are calculated at the 50 nodes shown in Figure 4-5. Figures 4-6 ~ 4-14 show the Z-displacement of the basemat relative to the free-field according to site profiles. These results are obtained from the analysis of seismic loading only; dead load is not included. These results are obtained as follows:

- Relative displacement time histories at the 50 selected basemat nodes are obtained using the SASSI RELDISP module.
- The average of the 50 relative displacement time histories is calculated.
- A snapshot of the relative displacements is obtained at the time of the minimum average time history and at the time of the maximum average time history.

From Figures 4-6 ~ 4-14, the maximum differential settlement by seismic loading is approximately 0.006 ft (0.072 in) which is less than 0.1 in. The differential settlement by seismic loading is calculated based on the maximum and minimum displacements of the basemat (not the differential settlements per 50 ft).

The maximum probable differential settlements of the APR1400 NI common basemat are 0.18 and 0.1 in for the static (D+L) and seismic ($E_s$) loading conditions. Therefore, it is concluded that the critical criteria for the differential settlement, which is 0.5 in differential settlement per 50 ft.

In addition, the differential settlement between the NI basemat and the other building is checked. Additional FE analyses for the turbine generator building (TGB), which is the building adjacent to the NI common basemat are performed for the differential settlement between the NI basemat and other buildings. The superstructure of the TGB consists of braced steel frames, and the basemat of the TGB is located at EL 73′-0″. The subgrade moduli for the TGB analysis corresponding to S1, S4, and S8 are 28.52 kcf, 121.37 kcf, and 877.20 kcf, respectively (see Table 2-4). The settlement analyses for the TGB basemat are carried out from using GTSTRUDL program. Figure 4-15 shows the FE model for the TGB basemat analysis. The maximum settlements of the NI and TGB basemats are used for calculating the differential settlement. Table 4-3 shows the differential settlement between the NI basemat and the TGB basemat. From the analysis results, it is concluded that the criteria for the differential settlement between the NI basemat and the other building, which is 0.5 in, is acceptable.

### 4.1.3 Site Interface for the Nuclear Island Common Basemat

The bearing pressures of the NI common basemat by static and seismic loadings are evaluated in this section.

For the bearing pressure, the D+L load (static) case and LC08 ~ LC15 (dynamic) cases are applied in the basemat and the maximum bearing pressures of the basemat are obtained from the ANSYS static analysis. Table 4-4 shows the bearing pressures by static and dynamic loadings. These bearing pressures are satisfied because the allowable bearing capacity is less than or equal to 15 ksf (static) and 60 ksf (dynamic).

## 4.2 Stability Check of the Nuclear Island Common Basemat

The NI common basemat structure is evaluated for stability against overturning, sliding, and floatation. The calculated factors of safety against overturning, sliding, and floatation for the applicable load combinations satisfy the criteria shown in Table 4-5.

The normal design ground water elevation for the APR1400 is 96.67 ft. The extreme ground water elevation (design basis flood level) is the same as the plant grade level (98.67 ft) for seismic Category I, II, and III structures considering the probable maximum flood level.

In the earthquake load, axial force, shear force, and moment due to horizontal and vertical excitation of the structure are obtained from seismic analys2s (References [2] and [3]). Table 4-6 shows the enveloped results of the seismic analysis corresponding to each site profile (S1~S9). Since the seismic load governs the wind load, a stability check is not considered for the wind load condition. In addition, the earth

pressure effect is neglected for a conservative stability check.

#### 4.2.1 Overturning Check

For the overturning check, the possible minimum resisting moment and maximum driving moment are conservatively calculated. In addition, when overturning is checked in combination with seismic forces ($E_s$), the hydrostatic force at the design water level ($H_e$) is used. Minimum resisting moment is obtained by multiplying the effective dead load ($D$-$H_e$) by the minimum distance ($d_{min}$). Maximum driving moment consists of the overturning moments due to horizontal moments ($M_x$ and $M_y$), seismic shear forces ($F_x$ and $F_y$), and upward seismic force (V). The 100-40-40 method is used for upward seismic force.

- Minimum resisting moment = 125,666,760 kips-ft
- Maximum driving moment = 92,086,763 kips-ft
- Factor of safety (FOS) for $D+H_e+E_s$ load combination
  - minimum resisting moment / maximum driving moment = 1.36 > 1.1

#### 4.2.2 Sliding Check

The resistance forces against sliding of the common basemat are checked for the driving shear forces generated from the seismic load. The basemat friction force is considered to resist the sliding of the common basemat. In the sliding check, the shear key and earth pressure effects are conservatively neglected. In addition, when sliding is checked in combination with seismic forces, the hydrostatic force at the design water level is used.

For the sliding check, the coefficient of friction ($\mu$) is 0.7 that the internal friction angle is 35°. The resistance force is calculated by multiplying the coefficient of friction by the effective dead load. The X-directional (E-W) seismic forces are selected as the maximum driving shear forces.

- Resisting force = 617,310 kips
- Maximum driving force = 408,146 kip
- Factor of safety (FOS) for $D+H_e+E_s$ load combination
  - resisting force / maximum driving force = 1.51 > 1.1

#### 4.2.3 Flotation Check

Flotation problems may be encountered during construction, operation, or flood condition. The dead weight of the structure is used to resist the hydrostatic uplift. For the flotation check, the hydrostatic force at flooding ground water level ($H_s$) is used. Any skin friction between the subgrade exterior walls and backfill is conservatively neglected.

- Resisting force = 1,232,270 kips
- Maximum driving force = 364,029.4 kip
- Factor of safety (FOS) for $D+H_e+E_s$ load combination
  - resisting force / maximum driving Force = 3.39 > 1.1

# 5.0 CONSTRUCTION SEQUENCE ANALYSIS

This section presents the construction sequence analysis of the APR1400 NI common basemat for the evaluation of the settlement of the NI common basemat during construction.

## 5.1 General

The construction sequence analysis accounts for the construction sequence and the associated varying loads and stiffnesses of the NI common basemat. The construction sequence analysis focuses on the response of the basemat in the early stages of construction when it could be susceptible to different loading and deformations. For the construction sequence analysis, 19 basemat concrete segments are constructed during the concrete placement and hardening stages. The construction sequence scenarios are based on construction techniques and experience from the Shin-Kori Unit 4 plant. In addition, it is assumed that there are no unscheduled delays and that the site is dewatered and excavated. Figure 5-1 shows the location of each basemat segment, and Table 5-1 represents the sequence of the construction stage. As shown in Figure 5-1 and Table 5-1, segments of concrete blocks are added to the construction site according to the prescribed concrete placement order.

## 5.2 Development of Finite Element Models for the Construction Sequence Analysis

### 5.2.1 Material Properties

The concrete used in the construction sequence analysis is normal weight concrete with the compressive strength of 5,000 psi at 91 days. However, the concrete strength is assumed for three hardening conditions to consider strength changes due to the concrete pouring sequence. The purpose of this assumption is to check the stress changes for the concrete according to the hardening (curing) time. If the hardening time has a significant effect on the concrete stresses, an analysis considering the hardening condition of the concrete due to the actual construction schedules is also performed.

In this report, the relationship between the age and strength of the concrete complies with the relationship for moist-cured concrete made with normal Portland cement. The modulus of elasticity for concrete is calculated using the equation, $57{,}000\sqrt{f_c'}$ as given in ACI-349 (References [12]). In addition, the compressive strength based on the hardening time is divided as follows:

Compressive strength according to the hardening time:

Hardening Step 1 (H1) : $0.67 \cdot f_c' = 3{,}350$ psi
Hardening Step 2 (H2) : $0.86 \cdot f_c' = 4{,}300$ psi
Hardening Step 3 (H3) : $1.00 \cdot f_c' = 5{,}000$ psi

The corresponding elastic moduli are:

Hardening Step 1 (H1) : $E = 3.30 \times 10^6$ psi
Hardening Step 2 (H2) : $E = 3.74 \times 10^6$ psi
Hardening Step 3 (H3) : $E = 4.03 \times 10^6$ psi

For the construction sequence analysis, the site cases with the strongest (S8) and weakest (S1) properties are considered to represent the limiting site conditions. Table 5-2 shows the material properties for construction sequence analysis.

### 5.2.2 Finite Element Model

The FE model for the construction sequence analysis consists of the following:

- Ground (El. -900 ft 0 in ~ El. 100 ft 0 in)
- Basemat concrete segment for concrete pouring

The SOLID185 elements in ANSYS program are used for the ground and basemat model. In addition, the fixed boundary condition is applied to the bottom and the roller boundary condition is applied to the sides of ground model. Figure 5-2 shows the FE model for construction sequence analysis.

## 5.3 Construction Sequence Analysis Results

For S1 and S8, the results of the settlement during the construction sequence are presented in Tables 5-3. Settlement distribution contours for each site profiles and construction stages are provided in Figures 5-3 and 5-4, respectively.

The maximum differential settlement is calculated as 0.39 and 0.029 inches at S1 and S8, respectively.

## 6.0 REFERENCES

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Table 2-1. Unit Weight and Poisson's Ratio According to Shear Wave Velocity

| Shear Wave Velocity, $V_s$ (ft/sec) | Unit Weight, $\gamma$(pcf) | Poisson's Ratio, $\nu$ | Remark |
|---|---|---|---|
| $V_s$ = 1,000~1,999 ft/sec | 125 | 0.40 | - |
| $V_s$ = 2,000~3,999 ft/sec | 130 | 0.38 | - |
| $V_s$ = 4,000~5,999 ft/sec | 135 | 0.35 | - |
| $V_s$ = 6,000~9,199 ft/sec | 145 | 0.33 | - |
| $V_s \geq$ 9,200 ft/sec | 155 | 0.33 | Fixed Condition |

Table 2-2. Ground Type Based on IBC

| Item | Shear Wave Velocity, $V_s$ (ft/sec) | *N* (Standard Penetration Test) |
|---|---|---|
| Hard Rock | $V_s$ > 5,000 ft/sec | - |
| Medium Hard Rock | $V_s$ = 2,500~5,000 ft/sec | - |
| Very Dense Soil or Soft Rock | $V_s$ = 1,200~2,500 ft/sec | > 50 |
| Dense Soil | $V_s$ = 600~1,200 ft/sec | 15~50 |
| Soft Soil | $V_s$ < 600 ft/sec | < 15 |

Table 2-3. Site Properties in ANSYS Ground Model

| Ground Level (ft) | S1 | | | S4 | | | S8 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Unit Weight (kcf) | Static Modulus $E_{static}$ (ksf) | Poisson ratio v | Unit Weight (kcf) | Static Modulus $E_{static}$ (ksf) | Poisson ratio v | Unit Weight (kcf) | Static Modulus $E_{static}$ (ksf) | Poisson ratio v |
| 0 ~ 25 | 0.125 | 1,861.6 | 0.40 | 0.125 | 5,638.8 | 0.40 | 0.145 | 146,888.2 | 0.33 |
| 25 ~ 55 | 0.125 | 2,256.0 | 0.40 | 0.126 | 6,333.0 | 0.40 | 0.145 | 152,585.5 | 0.33 |
| 55 ~ 75 | 0.125 | 2,635.5 | 0.40 | 0.130 | 7,099.5 | 0.38 | 0.145 | 157,719.8 | 0.33 |
| 75 ~ 100 | 0.125 | 2,993.4 | 0.40 | 0.130 | 7,682.4 | 0.38 | 0.145 | 162,291.4 | 0.33 |
| 100 ~ 150 | 0.125 | 3,620.3 | 0.40 | 0.132 | 53,228.5 | 0.37 | 0.145 | 169,792.8 | 0.33 |
| 150 ~ 200 | 0.125 | 5,850.9 | 0.40 | 0.135 | 58,398.8 | 0.35 | 0.145 | 179,557.2 | 0.33 |
| 200 ~ 300 | 0.130 | 31,805.4 | 0.38 | 0.145 | 146,335.8 | 0.33 | 0.155 | 325,128.0 | 0.33 |
| 300 ~ 400 | 0.130 | 38,754.3 | 0.38 | 0.145 | 159,403.6 | 0.33 | 0.155 | 325,128.0 | 0.33 |
| 400 ~ 500 | 0.130 | 42,004.7 | 0.38 | 0.145 | 165,223.2 | 0.33 | 0.155 | 325,128.0 | 0.33 |
| 500 ~ 750 | 0.140 | 126,333.1 | 0.34 | 0.155 | 325,128.0 | 0.33 | 0.155 | 325,128.0 | 0.33 |
| 750 ~ 1,000 | 0.145 | 141,829.7 | 0.33 | 0.155 | 325,128.0 | 0.33 | 0.155 | 325,128.0 | 0.33 |

Table 2-4. Equivalent Subgrade Moduli of Site Profiles

| Site Profile | Max. Displacement (ft) | Subgrade modulus (kcf) | Remark |
|---|---|---|---|
| NI Basemat | | | |
| S1 | 0.028046 (Z, Vertical) | $k_v$ = 35.66 | - |
| | 0.072731 (X, Horizontal) | $k_h$ = 20.62 | 2/3 of maximum value |
| | 0.073070 (Y, Horizontal) | $k_h$ = 20.53 | |
| S4 | 0.005769 (Z, Vertical) | $k_v$ = 173.34 | - |
| | 0.023239 (X, Horizontal) | $k_h$ = 64.55 | 2/3 of maximum value |
| | 0.023245 (Y, Horizontal) | $k_h$ = 64.53 | |
| S8 | 0.001162 (Z, Vertical) | $k_v$ = 860.59 | - |
| | 0.001099 (X, Horizontal) | $k_h$ = 1,364.88 | 2/3 of maximum value |
| | 0.001123 (Y, Horizontal) | $k_h$ = 1,335.71 | |
| TGB Basemat | | | |
| S1 | 0.035069 (Z, Vertical) | $k_v$ = 28.52 | - |
| | 0.041371 (X, Horizontal) | $k_h$ = 24.17 | |
| | 0.041708 (Y, Horizontal) | $k_h$ = 23.98 | |
| S4 | 0.008239 (Z, Vertical) | $k_v$ = 121.37 | |
| | 0.013406 (X, Horizontal) | $k_h$ = 74.59 | |
| | 0.013465 (Y, Horizontal) | $k_h$ = 74.27 | |
| S8 | 0.001140 (Z, Vertical) | $k_v$ = 877.20 | |
| | 0.000595 (X, Horizontal) | $k_h$ = 1,680.67 | |
| | 0.000608 (Y, Horizontal) | $k_h$ = 1,644.74 | |

∗ Subgrade modulus (kcf) = Pressure (1ksf) / Max. Displacement (ft)

Table 3-1. Material Properties of the NI Structures

| Property | NI Basemat | RCB Shell and Dome | RCB Internal Structure | AB Structure |
|---|---|---|---|---|
| Concrete strength (fc′, psi) | 5,000 | 6,000 | 6,000 | 5,000 |
| Elastic modulus (Ec, ksf) | 580,460 | 635,800 | 635,800 | 580,460 |
| Poisson’s Ratio | 0.17 | 0.17 | 0.17 | 0.17 |
| Weight per unit volume (w, kcf) | 0.15 | 0.15 | 0.15 | 0.15 |

Table 3-2. Reaction Force at Arbitrary Point Corresponding to Each Seismic Excitation

| Seismic Excitation | AB Building (A) | RCB Shell and Dome (C) | RCB Internal Structure (I) |
|---|---|---|---|
| X-direction | $(F_x)_x$, $(F_y)_x$, $(F_z)_x$ | $(F_x)_x$, $(F_y)_x$, $(F_z)_x$ | $(F_x)_x$, $(F_y)_x$, $(F_z)_x$ |
| Y-direction | $(F_x)_y$, $(F_y)_y$, $(F_z)_y$ | $(F_x)_y$, $(F_y)_y$, $(F_z)_y$ | $(F_x)_y$, $(F_y)_y$, $(F_z)_y$ |
| Z-direction | $(F_x)_z$, $(F_y)_z$, $(F_z)_z$ | $(F_x)_z$, $(F_y)_z$, $(F_z)_z$ | $(F_x)_z$, $(F_y)_z$, $(F_z)_z$ |

- $(F_x)_i$, $(F_y)_i$, and $(F_z)_i$ denote the three directional reaction forces corresponding to i (X, Y, and Z) directional seismic excitation.

Table 3-3. Seismic Load Cases for the Superstructures Using the 100-40-40 Method

| AB Building (A) | RCB Shell and Dome (C) | RCB Internal Structure (I) |
|---|---|---|
| $(Fx)_A = (Fx)_x+0.4(Fx)_y+0.4(Fx)_z$<br>$(Fy)_A = 0.4(Fy)_x+(Fy)_y+0.4(Fy)_z$<br>$(Fz)_A = 0.4(Fz)_x+0.4(Fz)_y+(Fz)_z$ | $(Fx)_C = (Fx)_x+0.4(Fx)_y+0.4(Fx)_z$<br>$(Fy)_C = 0.4(Fy)_x+(Fy)_y+0.4(Fy)_z$<br>$(Fz)_C = 0.4(Fz)_x+0.4(Fz)_y+(Fz)_z$ | $(Fx)_I = (Fx)_x+0.4(Fx)_y+0.4(Fx)_z$<br>$(Fy)_I = 0.4(Fy)_x+(Fy)_y+0.4(Fy)_z$<br>$(Fz)_I = 0.4(Fz)_x+0.4(Fz)_y+(Fz)_z$ |

Table 3-4. Seismic Load Combinations for the NI Common Basemat

| Case | X-directional Reaction | Y-directional Reaction | Z-directional Reaction |
|---|---|---|---|
| $E_S01$ | $(Fx)_A + (Fx)_C + (Fx)_I$ | $(Fy)_A + (Fy)_C + (Fy)_I$ | $(Fz)_A + (Fz)_C + (Fz)_I$ |
| $E_S02$ | $(Fx)_A + (Fx)_C + (Fx)_I$ | $(Fy)_A + (Fy)_C + (Fy)_I$ | $-(Fz)_A - (Fz)_C - (Fz)_I$ |
| $E_S03$ | $(Fx)_A + (Fx)_C + (Fx)_I$ | $-(Fy)_A - (Fy)_C - (Fy)_I$ | $(Fz)_A + (Fz)_C + (Fz)_I$ |
| $E_S04$ | $(Fx)_A + (Fx)_C + (Fx)_I$ | $-(Fy)_A - (Fy)_C - (Fy)_I$ | $-(Fz)_A - (Fz)_C - (Fz)_I$ |
| $E_S05$ | $-(Fx)_A - (Fx)_C - (Fx)_I$ | $(Fy)_A + (Fy)_C + (Fy)_I$ | $(Fz)_A + (Fz)_C + (Fz)_I$ |
| $E_S06$ | $-(Fx)_A - (Fx)_C - (Fx)_I$ | $(Fy)_A + (Fy)_C + (Fy)_I$ | $-(Fz)_A - (Fz)_C - (Fz)_I$ |
| $E_S07$ | $-(Fx)_A - (Fx)_C - (Fx)_I$ | $-(Fy)_A - (Fy)_C - (Fy)_I$ | $(Fz)_A + (Fz)_C + (Fz)_I$ |
| $E_S08$ | $-(Fx)_A - (Fx)_C - (Fx)_I$ | $-(Fy)_A - (Fy)_C - (Fy)_I$ | $-(Fz)_A - (Fz)_C - (Fz)_I$ |

Table 3-5. Load Combinations for the NI Common Basemat Analysis

| Position | Condition | Load Case | Load Combination |
|---|---|---|---|
| RCB Basemat | Test | LC01 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_t$ |
| | Normal | LC02 | $1.0D+1.0L+1.0L_h+1.0F$ |
| | Severe | LC03 | $1.0D+1.3L+1.3L_h+1.0F$ |
| | Abnormal | LC04 | $1.0D+1.0L+1.0Lh+1.0F+1.5P_a$ |
| AB Basemat | Test | LC05 | $1.1D+1.3L+1.1L_h+1.0F+1.0P_t$ |
| | Normal | LC06 | $1.4D+1.7L+1.4L_h+1.0F$ |
| | Abnormal | LC07 | $1.0D+1.0L+1.0Lh+1.0F+1.4P_a$ |
| RCB and AB Basemat | Abnormal /Extreme | LC08 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s01$ |
| | | LC09 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s02$ |
| | | LC10 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s03$ |
| | | LC11 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s04$ |
| | | LC12 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s05$ |
| | | LC13 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s06$ |
| | | LC14 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s07$ |
| | | LC15 | $1.0D+1.0L+1.0L_h+1.0F+1.0P_a+1.0Y_r+1.0E_s08$ |

Where:

D = Dead load
L = Live load
F = Post-tension load of tendon embedded RCB shell and dome
$P_a$ = Design internal pressure of RCB shell and dome
$P_t$ = Internal pressure of RCB shell and dome at testing phase
$Y_r$ = Pipe break load
$E_s$ = Seismic load

Table 4-1. Uplift Area Ratios for the NI Common Basemat

| Site Profile | Load Combinations | Area at Bottom of Basemat ($ft^2$) | Uplift Area ($ft^2$) | Uplift Area Ratios (%) |
|---|---|---|---|---|
| S1 | LC08 | 113,590 | 20,530.86 | 18.07 % |
| | LC10 | | 3,976.67 | 3.50 % |
| | LC12 | | 10,393.17 | 9.15 % |
| S4 | LC08 | | 22,540.91 | 19.81 % |
| | LC10 | | 2,455.38 | 2.16 % |
| | LC12 | | 9,933.7 | 8.75 % |
| S8 | LC08 | | 23,353.56 | 20.56% |
| | LC10 | | 8,470.57 | 7.46 % |
| | LC12 | | 17,032.33 | 14.99 % |

Table 4-2. Differential Settlements According to Site Profiles (Static Loading Case)

| Section | Node Number | | Distance (ft) | Differential Settlement (in) | | |
|---|---|---|---|---|---|---|
| | Start | End | | S1 | S4 | S8 |
| AB1 | 26810 | 27829 | 48.58 | 0.103 | 0.013 | 0.010 |
| AB2 | 27829 | 29466 | 46.26 | 0.149 | 0.056 | 0.022 |
| AB3 | 29466 | 28901 | 46.59 | 0.145 | 0.045 | 0.006 |
| AB4 | 28901 | 1367 | 44.70 | 0.176 | 0.072 | 0.026 |
| AB5 | 26811 | 27246 | 48.73 | 0.059 | 0.023 | 0.020 |
| AB6 | 27246 | 26610 | 44.08 | 0.011 | 0.013 | 0.007 |
| AB7 | 26610 | 27669 | 41.54 | 0.005 | 0.019 | 0.005 |
| AB8 | 27669 | 790 | 39.68 | 0.046 | 0.053 | 0.025 |
| AB9 | 26620 | 28027 | 48.73 | 0.152 | 0.059 | 0.037 |
| AB10 | 28027 | 26667 | 44.08 | 0.079 | 0.002 | 0.014 |
| AB11 | 26667 | 27610 | 41.54 | 0.084 | 0.011 | 0.004 |
| AB12 | 27610 | 822 | 39.68 | 0.036 | 0.023 | 0.014 |
| AB13 | 26826 | 27117 | 48.58 | 0.054 | 0.011 | 0.010 |
| AB14 | 27117 | 29708 | 46.26 | 0.106 | 0.057 | 0.024 |
| AB15 | 29708 | 30238 | 46.59 | 0.101 | 0.044 | 0.007 |
| AB16 | 30238 | 1466 | 44.70 | 0.126 | 0.071 | 0.029 |
| RCB1 | 5929 | 18822 | 46.06 | 0.087 | 0.012 | 0.005 |
| RCB2 | 15931 | 15467 | 47.09 | 0.009 | 0.009 | 0.003 |
| RCB3 | 6135 | 14571 | 46.06 | 0.028 | 0.002 | 0.003 |
| RCB4 | 16131 | 15368 | 47.09 | 0.082 | 0.012 | 0.004 |
| Total Max. Differential Settlement | | | | 0.176 | 0.072 | 0.037 |

Table 4-3. Differential Settlements between NI Basemat and TGB Basemat (Static Loading Case)

| Basemat / Differential Settlement | Max. Settlement (in) | | |
|---|---|---|---|
| | S1 | S4 | S8 |
| NI Basemat | 4.063 | 0.853 | 0.187 |
| TGB Basemat | 3.972 | 1.103 | 0.205 |
| Differential Settlement | 0.091 | 0.250 | 0.018 |

Table 4-4. Bearing Pressure of the NI Common Basemat

| Case | Max. Bearing Pressure (ksf) | | |
|---|---|---|---|
| | S1 | S4 | S8 |
| Static Case | 12.074 | 12.321 | 13.397 |
| Dynamic Case | 18.084 | 19.357 | 29.572 |

∗ Bearing pressure (ksf) = Maximum soil spring reaction (kips) / Tributary area ($ft^2$)

Table 4-5. Required Factor of Safety (FOS) for the Stability Check

| Load Combination | Minimum Factor of Safety (FOS) | | |
|---|---|---|---|
| | Overturning | Sliding | Flotation |
| $D + H_e + W$ | 1.5 | 1.5 | - |
| $D + H_e + E_s$ | 1.1 | 1.1 | - |
| $D + H_e + W_t$ | 1.1 | 1.1 | - |
| $D + H_s$ | - | - | 1.1 |

Where:

D = Dead load
$H_e$ = Static and dynamic lateral and vertical earth pressure including buoyant effect of normal design ground water level
$H_s$ = Buoyant force of the design basis flood
W = Wind Load
$W_t$ = Tornado Load
$E_s$ = Safe Shutdown Earthquake

Table 4-6. Enveloped Results of the Seismic Analysis Corresponding to Site Profiles

| Superstructure | Axial (V) (kips) | Shear X ($F_x$) (kips) | Shear Y ($F_y$) (kips) | Moment X ($M_x$) (kips-ft) | Moment Y ($M_y$) (kips-ft) |
|---|---|---|---|---|---|
| Containment Shell and Dome (El. 78′-0″) | 59,620 | 67,610 | 73,820 | 12,230,000 | 11,540,000 |
| Primary Shield Wall (El. 66′-0″) | 13,480 | 11,550 | 11,520 | 684,600 | 839,700 |
| Secondary Shield Wall (El. 78′-0″) | 23,430 | 22,290 | 19,970 | 936,200 | 1,348,000 |
| Auxiliary Building (El. 55′-0″) | 210,400 | 239,100 | 220,800 | 19,790,000 | 20,770,000 |
| Total | 306,930 | 340,550 | 326,110 | 33,640,800 | 34,497,700 |

∗ A value of 0.3 times the basemat self-weight is conservatively used for axial and shear forces of the basemat corresponding to each directional seismic force with regard to a 0.3g acceleration of SSE.

Table 5-1. Sequence of Basemat Segments due to Concrete Pouring

| Stage | A-S001 | C-S001 | C-S002 | A-S004 | C-S003 | A-S005 | C-S004 | A-S006 | A-S002 | A-S003 | A-S007 | A-S008 | C-W007 | A-S010 | C-S005 | A-S009 | C-S006 | C-S008 | C-S009 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | P | | | | | | | | | | | | | | | | | | |
| 2 | H1 | P | | | | | | | | | | | | | | | | | |
| 3 | H2 | H1 | P | | | | | | | | | | | | | | | | |
| 4 | H3 | H2 | H1 | P | | | | | | | | | | | | | | | |
| 5 | H3 | H3 | H2 | H1 | P | | | | | | | | | | | | | | |
| 6 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | | | | | | |
| 7 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | | | | | |
| 8 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | | | | |
| 9 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | | | |
| 10 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | | |
| 11 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | | |
| 12 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | | |
| 13 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | | |
| 14 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | | |
| 15 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | | |
| 16 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | | |
| 17 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | P | | |
| 18 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 | | |
| 19 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | | |
| 20 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | P | |
| 21 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H1 | P |
| 22 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 | H1 |
| 23 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H2 |
| 24 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 | H3 |

Note:

1) 'P' indicates the segment where concrete is being poured in the stage
2) 'H1' indicates the segment where concrete has hardened to 'Hardening Step1' which has the compressive strength of 3,350psi, in the previous stage
3) 'H2' indicates the segment where concrete has hardened to 'Hardening Step2' which has the compressive strength of 4,300psi, in the previous stage
4) 'H3' indicates the segment where concrete has hardened to 'Hardening Step3' which has the compressive strength of 5,000psi, in the previous stage

Table 5-2. Material Properties for the Construction Sequence Analysis

| Material | | Elastic Modulus (ksf) | Poisson Ratio | Weight Density (pcf) | Remarks |
|---|---|---|---|---|---|
| Concrete | H1 | 4.7506E+05 | 0.17 | 150 | at 0.67·$f'_c$ |
| | H2 | 5.3827E+05 | 0.17 | 150 | at 0.86·$f'_c$ |
| | H3 | 5.8032E+05 | 0.17 | 150 | at 1.00·$f'_c$ |
| S1 | Layer 1 | 1.8970E+03 | 0.4 | 125 | El. 70′-0″ ~ El. 100′-0″ |
| | Layer 2 | 2.3680E+03 | 0.4 | 125 | El. 35′-0″ ~ El. 70′-0″ |
| | Layer 3 | 2.9140E+03 | 0.4 | 125 | El. 0′-0″ ~ El. 35′-0″ |
| | Layer 4 | 3.6200E+03 | 0.4 | 125 | El. -50′-0″ ~ El. 0′-0″ |
| | Layer 5 | 5.8510E+03 | 0.4 | 125 | El. -100′-0″ ~ El. -50′-0″ |
| | Layer 6 | 3.1805E+04 | 0.38 | 130 | El. -200′-0″ ~ El. -100′-0″ |
| | Layer 7 | 3.8754E+04 | 0.38 | 130 | El. -300′-0″ ~ El. -200′-0″ |
| | Layer 8 | 4.5255E+04 | 0.38 | 130 | El. -400′-0″ ~ El. -300′-0″ |
| | Layer 9 | 1.1751E+05 | 0.35 | 135 | El. -520′-0″ ~ El. -400′-0″ |
| | Layer 10 | 1.2999E+05 | 0.33 | 145 | El. -1,000′-0″ ~ El. -520′-0″ |
| S8 | Layer 1 | 1.4741E+05 | 0.33 | 145 | El. 70′-0″ ~ El. 100′-0″ |
| | Layer 2 | 1.5413E+05 | 0.33 | 145 | El. 35′-0″ ~ El. 70′-0″ |
| | Layer 3 | 1.6128E+05 | 0.33 | 145 | El. 0′-0″ ~ El. 35′-0″ |
| | Layer 4 | 1.6979E+05 | 0.33 | 145 | El. -50′-0″ ~ El. 0′-0″ |
| | Layer 5 | 1.7956E+05 | 0.33 | 145 | El. -100′-0″ ~ El. -50′-0″ |
| | Layer 6 | 3.2513E+05 | 0.33 | 155 | El. -1,000′-0″ ~ El. -100′-0″ |

Table 5-3. Results of the Construction Sequence Analysis

| Stage | Settlement (in) | | | | | |
|---|---|---|---|---|---|---|
| | S1 | | | S8 | | |
| | Max | Min | Diff | Max | Min | Diff |
| 1 | - | - | - | - | - | - |
| 2 | -0.1644 | -0.1044 | -0.06 | -0.0096 | -0.0048 | -0.0048 |
| 3 | -0.2124 | -0.0756 | -0.1368 | -0.012 | -0.0036 | -0.0072 |
| 4 | -0.2448 | -0.0792 | -0.1656 | -0.012 | -0.0036 | -0.0084 |
| 5 | -0.2496 | -0.1296 | -0.1212 | -0.0132 | -0.0048 | -0.0084 |
| 6 | -0.2964 | -0.0672 | -0.2304 | -0.0144 | -0.0048 | -0.0096 |
| 7 | -0.306 | -0.096 | -0.21 | -0.0144 | -0.0048 | -0.0096 |
| 8 | -0.3432 | -0.0924 | -0.2508 | -0.0156 | -0.0048 | -0.0108 |
| 9 | -0.3516 | -0.1716 | -0.18 | -0.0168 | -0.006 | -0.0096 |
| 10 | -0.3648 | -0.1608 | -0.204 | -0.0168 | -0.0072 | -0.0096 |
| 11 | -0.378 | -0.162 | -0.216 | -0.0168 | -0.006 | -0.0108 |
| 12 | -0.3804 | -0.168 | -0.2136 | -0.018 | -0.006 | -0.0108 |
| 13 | -0.3948 | -0.1728 | -0.222 | -0.018 | -0.006 | -0.012 |
| 14 | -0.3972 | -0.2016 | -0.1956 | -0.0192 | -0.0072 | -0.012 |
| 15 | -0.4752 | -0.2028 | -0.2724 | -0.024 | -0.0072 | -0.0168 |
| 16 | -0.4944 | -0.2688 | -0.2256 | -0.0252 | -0.0084 | -0.018 |
| 17 | -0.546 | -0.2712 | -0.2748 | -0.0276 | -0.0084 | -0.0204 |
| 18 | -0.546 | -0.276 | -0.2688 | -0.0276 | -0.0084 | -0.0204 |
| 19 | -0.5448 | -0.2796 | -0.2652 | -0.0276 | -0.0084 | -0.0204 |
| 20 | -0.6228 | -0.2844 | -0.3384 | -0.0348 | -0.0084 | -0.0264 |
| 21 | -0.6876 | -0.2976 | -0.3888 | -0.0372 | -0.0084 | -0.0288 |
| 22 | -0.6876 | -0.2976 | -0.39 | -0.0372 | -0.0084 | -0.0288 |
| 23 | -0.6864 | -0.2988 | -0.3876 | -0.0372 | -0.0084 | -0.0288 |
| 24 | -0.6852 | -0.2988 | -0.3864 | -0.0372 | -0.0084 | -0.0288 |