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1 PURPOSE

The objective of this calculation is to estimate the three dimensional distribution of the vertical subsurface displacement at the Calvert Cliffs Nuclear Power Project Unit 3 (CCNP3) Power Block Area, including:

- Settlement after dewatering,
- Heave after excavation,
- Settlement during and after construction.

2 INTRODUCTION

PLAXIS 3D Foundation v2 is a Finite Element Method (FEM) based computer code designed for the geotechnical analyses.

PLAXIS 3D provides a FEM solution of the virtual work equation defining equilibrium conditions and natural boundary conditions in a differential equation form. The program calculates displacements with the use of numerical integration methods. In addition to the typical capabilities of a general FEM application for elastic solids, PLAXIS3D incorporates advanced constitutive models, (stress vs. strain relationships) that are capable of simulating the response of soils to external loading. Such response includes both elastic/elastoplastic displacement and consolidation. This feature makes PLAXIS3D a unique application for the analysis of foundation systems. The computer code also incorporates the capability of simulating the anisotropic behavior of soils.

Several soil constitutive models are incorporated within PLAXIS 3D. Three common models are summarized by Table 1.

This calculation utilizes the Mohr-Coulomb model. Two separate models were used in the analysis:

1. An excavation and dewatering (ED Model);
2. Construction and post-construction (PC Model).

The revision 1 for this calculation includes the following additions:

1. Update of hydraulic conducitivities according to the updated groundwater models conducted (Ref. 7). Input hydraulic conducitivities were updated for both ED and PC models.
2. Modification of ED model: Dewatering was not conducted as a separate stage in ED model. The Chesapeake aquifer is not connected to the surficial aquifer because of the upper Chesapeake aquitard in between the two layers. Therefore dewatering of the surficial aquifer is not expected to impact the groundwater conditions in the Chesapeake aquifer. ED model was updated to include only the excavation stage which also includes dewatering of the surficial aquifer but not the Chesapeake aquifer. The groundwater elevation before and after excavation was modified so that both aquifers have the corresponding groundwater tables.
3. The stiffness of soil layers were modified to account for the effect of unloading/reloading on the elastic modulus (will be explained in Section 3.3). Previous calculation version assumed similar unloading/reloading and primary moduli to be conservative.

4. For the ED model, unloading/reloading modulus , E_r, was used for all soil layers. For the PC model, E_r was used only up to a certain load step.
5. The stiffness of the Nanjemoy sand was increased linearly by depth. Nanjemoy sand was assumed to be about 550 ft thick (to meet the U.S. Nuclear Regulatory Commission Regulatory Guide (1.132) Appendix D requirement for the depth as discussed in Sections 3, 4.1 and 4.2), and the constant modulus assumed throughout the entire layer was conservative, since the strength and stiffness of sandy materials increase with the effective mean stress, i.e., depth.
6. The surface topography in ED model was modified to account for the variations in elevations in the immediate vicinity of the power block area.

Resulting figures for the two models were also updated based on these revisions.

3 DESIGN INPUT / METHODOLOGY

The area considered in the model is 2500 ft by 2500 ft. The area occupied by the buildings is 1100 ft by 1100 ft. The site geometry was obtained from Ref. 1 and is shown in Figure 1. Figure 2 provides a detailed geometry of the nuclear island common base mat. 15-noded wedge elements were used in the analysis. Total number of nodes was 39862 and 42130 for ED and PC models, respectively. 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 implemented 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 that the consolidation behavior is not impacted by the free-drainage conditions implemented on the sides of the model.

For the ED model, the size of the triangular finite element is about 90 ft in the finely meshed area, and 130 ft in the coarsely meshed area on the horizontal plane. In the vertical plane, the element length varies between 10 ft and 50 ft (approximate numbers). The aspect ratio with these elements sizes ranges between 1.5 and 13. The finely meshed areas have aspect ratio between 1.5 and 9.0.

For the PC model, the size of the triangular finite element is about 25 ft in the finely meshed area, and 150 ft in the coarsely meshed area on the horizontal plane. In the vertical plane, the element length varies between 10 ft and 50 ft (approximate numbers). The aspect ratio with these elements sizes ranges between 0.5 and 15. The finely meshed areas have aspect ratio between 0.5 and 2.5.

Soil profiles , such as the one shown in Figure 3 and Figure 4, in the north-south and east-west directions of the Nuclear Island (A-A' and B-B') respectively were taken as the basis for the geotechnical input of the FEM model. In addition, data from boreholes B311, B313, B334, B335, B344, and B357A was included to adequately represent the three-dimensional nature of the model (Ref. 2). In total, data from 20 boreholes was used. PLAXIS3D interpolates information between borehole locations to obtain the three-dimensional representation of the subsurface conditions, as shown in Figure 5.

U.S. Nuclear Regulatory Commission Regulatory Guide (1.132) Appendix D states that:

"Where soils are very thick, the maximum required depth for engineering purposes, denoted d_{max}, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loadings is less than 10% of the effective in situ overburden stress".

The analysis depth of 2B, where B is the least dimension of the foundation, was assumed to be adequate to meet this criterion. The exploration depth of the borehole program extended down to El. -310 ft. However, given the dimensions of the NI common base mat, the model depth was extended to El. -760 ft to have a model with a depth of at least 2B. It is necessary to assume that the deepest Nanjemoy sand (the continuous soil layer deeper than -208 ft elevation) extends down to the bottom end of the model based on the best information available.

3.1 Excavation and Dewatering (ED) Model

The ED model was designed to evaluate settlements in the CCNPP Unit 3 power block area due to excavation before construction.

In the CCNPP Unit 3 power block area, the current ground elevations range from approximately El. 50 ft to El. 120 ft, with an approximate average value of 86 ft (Ref. 3). For the ED model, the CCNPP Unit 3 area was subdivided 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 6. The zones are:

- Zone I, on the North-East area of Unit 3; the average ground elevation for this zone is 60 ft.
- Zone II, at the North-West area of Unit 3; the average ground elevation for this zone is 80 ft.
- Zone III, at the South area of Unit 3; the average ground elevation for this zone is 105 ft.

For Zone III, the ground surface is modeled at El. 80 ft, and the overburden effect between El. 80 ft and El. 105 ft is modeled with a loading of 2963 psf. This loading was calculated based on the average moist unit weight (118.5 pcf for CCNPP Unit 3 area) for Terrace Sand and Chesapeake Clay IIa layers (Ref. 4). The terrace Sand and Chesapeake Clay IIa layers exist on the site between the ground surface and El. 41.5 ft (Ref. 2). These two soil layers will be excavated and therefore only the unit weight is of relevance to the calculation. In the analysis, an equivalent soil layer (named as surficial soil layer) was used with the unit weight of 118.5 pcf. Since the model does not consist of only the power block area, the surrounding area of the power block area was also divided into three separate zones.

Initial configuration for the ED model includes surface elevations of 60 ft for Zone I, and 80 ft for Zones II and III. Soil layers encountered from top to bottom were:

- Surface soil layer
- Chesapeake Stratum IIb Sand Layer 1,
- Chesapeake Stratum IIb Sand Layer 2,
- Chesapeake Stratum IIb Sand Layer 3,
- Chesapeake Stratum IIc Clay/Silt, and
- Nanjemoy Sand.

The Chesapeake Stratum IIb consists mainly of sand and was divided into three sub-layers based on SPT blow count and shear wave velocity (Ref. 2). The layer Chesapeake Stratum IIc Clay/Silt has also two sand layers interbedded within the clay. As shown in Figures 3 and 4, there is a considerable change in the thicknesses of Chesapeake Stratum IIb Layer 1, Chesapeake Stratum IIb Layer 2, Chesapeake Stratum IIb Layer 3, Chesapeake Stratum IIc Clay/Silt, and Chesapeake Stratum IIc sand layers. The ED model can take into account the variations in soil layer thicknesses in three dimensions. Following assumptions were made while generating the soil profile:

1) Boreholes along sections A-A and C-C were assumed to lie on straight lines.
2) For 17 borings out of 20, the soil layer right below El. 41.5 is Chesapeake Stratum IIb Sand Layer 1, only three borings have a clay layer between El. 41.5 ft and the top of Chesapeake Stratum IIb Sand Layer 1. For these borings, these clay layers with thicknesses 7.8 ft (B313), 2.2 ft (B314), and 0.8 ft (B335) were assumed to be Chesapeake Stratum IIb Sand Layer 1 (for practical purposes).
3) Thin clay layers between Chesapeake Stratum IIc Sand Layer and Chesapeake Stratum IIb Sand Layer 3 were occasionally replaced with the Chesapeake Stratum IIc Sand Layer. The comparison of profiles obtained from bore holes and the one used in Plaxis analysis is given in Figures 7 and 8 for profiles AA' and BB', respectively. The soil profile used in analysis and the one obtained from boreholes are in good agreement.

Based on available information, the ground water level in CCNPP Unit 3 power block and construction laydown areas (CLA) ranges from approximately El. 60 ft to El. 85.7 ft with an average elevation of El. 80 ft (Ref. 3). For this analysis, the groundwater table was considered to be at El. 60 ft in Zone I, and at elevatation El. 80 ft in Zones II and III. This groundwater table is associated with the surficial aquifier. The surficial aquifer is not connected to the Chesapeake aquifer due to the presence of the upper Chesapeake aquitard. Therefore, two separate groundwater tables exist on site, one associated with the surficial aquifer, and the second one associated with the Chesapeake aquifer. The deeper groundwater table for the Chesapeake aquifer varies between El. 34 ft and El. 38 ft (mostly around El. 38 ft). The groundwater table for the Chesapeake aquifer was assumed to be at El. 38 ft (Ref. 3).
The ED model simulated the excavation process down to the El. 41.5 ft, the elevation of the Nuclear Island Common mat,. Therefore, a total of 18.5 ft, 38.5 ft and 63.5 ft of soil will be excavated in zones I, II and III, respectively, resulting in the removal of approximately $5.3x10^7$ ft^3 (2 million cyd) of material. Retaining walls were placed at the line of intersection of El. 60 and El. 80 in order to eliminate slope failure problems.

3.2 THE POST CONSTRUCTION (PC) MODEL

The PC model was designed to evaluate the settlements during and after construction. This model was not intended to be linked to the ED model. For the PC 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 was fully dissipated. Also, the ground surface was assumed to be re-leveled after the immediate settlement and heave (will be shown that heave due to consolidation is very small compared to the immediate heave) as calculated by ED model. The initial stage for PC model had the ground surface at El. 41.5 ft. The three dimensional PC model and soil profiles used in the Plaxis 3D input file is shown in Figure 9 and Figure 10 (Ref. 2). Initial effective stress conditions for the PC model was in accordance with the post-excavation overburden geometry. In other words, PC and ED models were separate, and the initial stress configuration of PC model did not use the results of ED model.

The building loads were applied in a sequential manner (8 steps). Information regarding building loads is provided in Table 2 and Table 3 (Ref. 5). The loading sequence is also shown in Figure 11 for all buildings considered. During the application of each load, consolidation analysis was conducted simultaneously, considering both immediate and consolidation settlements. After the last load increment was applied, dewatering was stopped.

Backfill between El. 41.5 ft and El 83 ft was placed in five steps as follows:

1) During Step 1 as given in Table 3, backfill was placed between El. 41.5 ft and El. 48 ft (60 days).

2) During Step 2 as given in Table 3, backfill was placed between El. 48 ft and El. 61 ft (80 days).
3) During Step 3 as given in Table 3, backfill was placed between El. 61 ft and El. 66.5 ft (160 days).
4) During Step 4 as given in Table 3, backfill was placed between El. 66.5 ft and El. 76 ft (200 days).
5) During Step 5 as given Table 3, backfill was placed between El. 76 ft and El. 83 ft (300 days).

Backfill placement schedule was assigned in such a way that before a foundation was placed, backfill was already in place below the foundation level (except the nuclear island common mat which does not have backfill underneath). In cases where the backfill elevation was higher than the foundation elevation on the foundation perimeter, the backfill was supported by retaining walls in the model so that the backfill does not experience a slope failure. More detailed description of backfill and load placement sequence are given in Figures 12, 13, 14, and 15. Figure 12 shows three profiles examined. Figures 13, 14, and 15 depict the schematic representation of load and backfill placement for load sections AA, BB, and CC, respectively.

The groundwater elevation in the power block area was at El. 38 ft during construction. Outside the powerblock area, the groundwater elevation was at El. 69 ft. For the post-construction conditions, groundwater elevation in the power block area was considered to increase up to El. 55 ft in 20 years and remained constant at that level, while the groundwater elevation outside the power block area remained at El. 69 ft.

3.3 Properties of Soil and Structural Elements

As mentioned in previous section, retaining walls were placed in the model around the buildings to eliminate the slope stability problems. The effect of having these walls on the vertical settlement around the site was assumed to be negligible. The properties for the walls used are given in Table 4.

The vertical displacement pattern observed underneath the buildings heavily depended on the stiffness of the foundation. As the construction proceeds, the displacement pattern of the foundations is expected to be closer to the rigid body motion due to the installation of structural walls and beams above. Therefore, in this analysis, the foundation stiffness was increased as a function of time in correspondence with the loading sequence (reflecting the construction sequence). Three-step increase in stiffness was incorporated into the analysis (some building foundations have only two-step stiffness increase). Table 5 shows the corresponding stiffness parameters for each step. Table 5 indicates that stiffness configuration C includes support walls, while configurations A and B do not. Support walls were considered only for the nuclear island common mat, and consist of four walls: two extending in north-south direction and two extending in east-west direction as shown in Figure 16. Structural properties of these walls are shown in Table 4.

Incorporation of foundation stiffness increase in the analysis is shown in Table 6. Most buildings have three-step increase, while Radioactive Waste, Emergency Power Generating, and Emergency Service Water Building Foundations have two-step foundation stiffness increases.

The soil properties are provided in Table 7 (Ref. 4, 6, 7, 8 and 9). Hydraulic conductivity was obtained from Ref. 7. The hydraulic conductivity for various layers were determined based on a groundwater model described in Ref. 7. Interbedded sand layers in Stratum IIc were assumed to have the same hydraulic conductivity as the Nanjemoy sand layer. The hydraulic conductivity of sandy materials were assumed to be ten times higher in the horizontal direcetion, whereas the hydraulic conductivity of clay layers was the same in vertical and horizontal directions. Effective Poisson's ratio for sandy and clayey or silty materials was assumed to be 0.3 and 0.35, respectively (Ref. 8). All soil properties were

determined based on test results conducted for materials in CCNPP Unit 3 power block area. If no information is available for the CCNPP Unit 3 power block area, results for the entire site were used. In both models, the displacement analysis for the sandy and clayey/silty materials was conducted for drained and undrained conditions, respectively. The analysis was conducted based on effective stress failure parameters. Mohr Coulomb failure model was adopted for all soils. During the consolidation analysis, a failure condition for the Mohr-Coulomb model was simultaneously checked. In other words, consolidation analysis and Mohr Coulomb failure model were simultaneously adapted (or coupled) in the model.

The unloading and reloading deformation moduli of soils are usually similar, E_r, while the primary loading modulus, E, is lower compared to unloading and reloading moduli (Ref. 9). Primary loading refers to a load higher than the initial overburden load.

In ED model, E_r was incorporated in the entire model. On the other hand, the modulus used was E_r only up to the certain loading step in the PC model, after which the modulus was reduced to E. Pre-excavation topography on the power block area suggests that the overburden stress is lower on the north-east part of the power block area. Lower loads would be needed to reach the initial overburden stress in the the north-east part of the power block area (around Fuel Building and Nuclear Auxiliary Building) compared to the rest of the power block area. Ideally, E_r should be used up to the end of the reloading, and, E should be used after the end of the reloading. The initial ground surface elevations vary between El. 50 ft and El. 120 ft in the power block area. Therefore, the reloading may last until the end of 2nd loading step (as given in Table 3) or the end of 5th loading step depending on the location of interest in the power block area. To adequately simulate the effect of initial topography four different runs were conducted (this will further be discussed in Section 4.2).

Run 1: E_r was used until the end of the second loading step, when the reloading for the north-east part of the power block area (the lowest topography initially) is expected to be completed (low topography model, LT model)

Run 2: E_r was used until the end of the third loading step, when all site is expected to undergo primary loading (medium topography model 1, MT1).

Run 3: E_r was used until the end of the fourth loading step (medium topography 2 model, MT2).

Run 4: E_r was used until the end of the fifth loading step. At the end of fifth step, all the power block area is expected to undergo primary loading (high topography model, HT).

The stiffness of the deepest Nanjemoy sand layer was increased linearly. The stiffness of dense sands is expected to increase by depth. SPT test results as shown in Ref.8 show such a trend; however, a rate of stiffness increase by depth is hard to deduce based solely on SPT test results. Determination of such a rate was based on soil-hardening model, which describes the stiffness change as a function of the confining stress. The rate of stiffness increase was determined to be 2589 psf / ft (Appendix 1). The stiffness of Nanjemoy sand shown in Table 7 was the value at the El. -207.5 ft, and increases linearly by depth according to the rate 2589 psf/ft.

4 RESULTS

The following sections provide the results of the simulations with the ED and PC models.

4.1 ED MODEL:

The effective vertical stress at the bottom of the model for the initial configuration (with different elevations and surface loading) is 54870 psf, and becomes 54320 psf right after the excavation. The change in effective vertical stress is less than 1.0 % of the initial effective overburden stress. Therefore, the depth selected in this model satisfies the requirement stated by U.S. Nuclear Regulatory Commission Regulatory Guide (1.132).

Shown in Figures 17 and 18 are the immediate heave and the heave due to de-consolidation (dissipation of negative excess pore pressures, Figure 19) when the power block area is excavated down to El. 41.5 ft. The maximum immediate heave due to excavation is 5.5 inches. Also shown in Figure 17, is the uneven heave pattern around the safety related buildings. The maximum de-consolidation related heave is 1.2 in as shown in Figure 18. If enough time is allowed to dissipate this excess pore pressure (i.e. for the de-consolidation to take place) after excavation, the final heave including both the immediate and de-consolidation induced is as shown in Figure 20. The condition shown in Figure 20 takes place in approximately 806 days after which no more heave is expected, since the de-consolidation is completed. The immediate, de-consolidation related, and total heave are given in Table 8 for Points A, B, C, and D around the power block area. Point C falls around the center of nuclear island common mat, as shown in Figures 17, 18, 20 and 21. These points are at 41.5 ft elevation. The maximum difference in heave due excavation (including the consolidation component) is 2.9 in between Points A and C. The total heave after excavation/dewatering is between 2.4 in and 5.3 in. The maximum difference is between Points A and C. For all points shown in Table 8, the heave due to consolidation is less than 13 % of the total heave. Table 8 also includes heave results 6 months and 1 year after the end of the excavation. According to this, more than 99 % of the expected total heave already takes place 6 months after the end of the excavation.

4.2 PC MODEL:

The maximum effective vertical stress at the bottom of the model for the initial configuration is 51190 psf, and becomes 52380 psf at the end of the excavation. The change in effective vertical stress is less than 2.3 % of the initial effective overburden stress. Therefore, the depth selected in this model satisfies the requirement stated by U.S. Nuclear Regulatory Commission Regulatory Guide (1.132).

The building geometries, foundation elevations and foundation contact pressures are given in Figure 22. Sections analyzed in terms of settlement and tilts are shown in Figure 22. In total 15 sections were analyzed. 88 locations in plan view shown in Figure 23 are included for the settlement analysis. Settlements at these locations but at the corresponding foundation elevations were used to compute tilts.

The tilt shown in Figures 23 through 27 may be induced by one or more of the three following factors:

1) Non-uniform load distribution throughout the power block area.

2) The non-uniform subsurface soil profile.

3) Initial surface topography effect.

The loading step, at which the initial overburden stress is equal to the stresses induced due to structural loads, controls the time of reduction in deformation modulus from E_r (for loads beyond the initial overburden load) to E. Since the initial

surface topography is variable across the site, the time step for E reduction may be different at different points across the power block area, and an additional tilt may be induced due to the initial surface topography effect. i.e. variable E for the same soil at different location. Aforementioned four models, HT, MT1, MT2 and LT models, can capture the tilt due to non-uniform load distribution and non-uniform subsurface soil profile when each one of them is considered alone. However, each model alone is not sufficient to address the initial surface topography effect.The approach taken was to determine which model would best describe the stiffness variation by time for a point underneath each one of the 88 locations (Figure 23) at El. 41.5 ft. This elevation was chosen as a reference elevation, since the excavation on the power block area was down to El. 41.5 ft. To match each location with a particular model, the results from the four models were combined in the following manner:

1) Surface elevations for 88 points analyzed were determined from initial site topography.

2) Initial overburden stresses were determined for 88 locations at El. 41.5 ft using an average overburden unit weight of 118 pcf and groundwater table elevation of;

- 80 ft if the point of interest has an elevation between 60 ft and 105 ft.
- 60 ft if the point of interest has an elevation less than 60 ft.

3) Using the results of LT model, the step at which the initial overburden stress matches the stresses applied due to structural loads was determined for each point.

4) Each point is matched with one of the models based on the load step determined in Item 3. The four models represent the deformation modulus reduction at the end of 2nd, 3rd, 4th, and 5th loading steps. Only 5 points corresponded to the end of 6th loading step for ESWB buildings, and they were matched with HT model, which is not expected to significantly alter the conclusions. This matching for 88 nodes analyzed is shown in Table 9, which was conducted based on the results of LT model.

This approach, however, may result in conservative estimates of tilt, since the foundation elements may seem to deform unrealistically. This unrealistic deformation pattern may emerge since two or more points across the same foundation element are analyzed using different models. In reality, the foundation stiffness is expected to compensate some of this additional tilt due to initial surface topography effect. Another implication of this approach is that it may not always give higher estimates of tilt. For example, if the tilt is in one direction due to load distribution and subsurface profile, the initial topography effect may increase or decrease this tilt, which can be determined using the aforementioned approach.

Settlements computed with MT2 model across 15 sections analyzed are given in Figures 23 through 27, which also include node numbers as given in Figure 22. The results from MT2 model were used, since this model represents an average case for the timing of deformation modulus in the CCNPP power block area, and this model describes the behavior of more points in the NI common mat than the other three models (Table 9). According to sections AA and BB going through the nuclear island common mat as shown in Figure 23, settlement at the center of the nuclear island mat is more than the settlement at the edges. Also shown in Figure 23 is the amount of tilt for sections AA and BB. The final tilt observed along section BB (0.27 in/50ft) is more than the tilt AA (0.10 in /50ft). This is mainly due to the high load difference between the buildings FB and SGB23. If TB and NI are considered together in the tilt analysis, as shown in section BB', the tilt reaches up to 0.42 in/ 50 ft. Figure 23 shows that as the building constructions come closer to the

completion, a distinct settlement pattern between the turbine building and nuclear island common mat emerges. According to this pattern, Point 29 (Figure 22) settles more than Point 87 (Figure 22). In other words, turbine building and nuclear island common mat tilts towards fuel building (CCNPP East). Settlements along diagonal sections CC and DD are shown in Figure 24. The lines indicating the settlement patterns for sections CC and DD have abrupt changes at about 140 ft distance for section CC and 110 ft distance for section DD. This is due to the change in foundation elevation at these points. For section CC, the foundation elevation changes from El. 48 ft to El. 41.5 ft (Figure 22, building NAB and the nuclear island common mat) at the distance of 140 ft in Figure 24, and similarly, for section DD, elevation change is again between El. 48 ft and El. 41.5 ft (Figure 22, building AB and the nuclear island common mat). Absolute magnitude of final tilt is similar for CC and DD sections. Both AB and NAB buildings tilt toward the nuclear island common mat (CCNPP south west and CCNPP south east, respectively). The tilt of the NI is towards NAB (CCNPP north east) along section CC, and towards 1ESWB and 2 ESWB (CCNPP south east) along section DD.

Figure 25 show the settlement patterns for 1ESWB and 2ESWB buildings. Both buildings tilt towards the nuclear island according to HH (towards CCNPP north west) and EE (towards CCNPP north west) sections. The magnitudes of tilt along GG and FF sections are significantly less than tilt along HH and EE sections indicating the effect of the nuclear island loading. Settlement along FF section for 1ESWB is close to uniform settlement. Settlement behavior of 3ESWB and 4ESWB are depicted in Figure 26. Sections EE, FF, GG, HH in Figure 25 are comparable to sections LL, KK, II, JJ in Figure 26, respectively. Qualitatively, settlement behavior along the sections EE, FF, GG, HH is very similar to the behavior along the sections LL, KK, II, JJ respectively. In other words, all ESWB buildings tilt towards NI. However, quantitatively, 1ESWB and 2ESWB experience more settlement and tilt compared to 3ESWB and 4ESWB. This is attributed to their closer location to NI.

Shown in Figure 27 is the settlement behavior of 1EPBG and 2EPGB. Settlement observed under both 1EPBG and 2EPGB indicate a rigid foundation behavior with tilt towards the nuclear island. In terms of overall settlement behavior, sections NN and OO, and sections MM and PP show similarity (symmetry). However, 1EPBG experiences slightly more settlement probably due to presence of AB nearby. Also, nodes 56 and 60 experience slightly more settlement compared to nodes 57 and 63, respectively. This is attributed to the heavier NI and FB being closer to nodes 56 and 60 than nodes 57 and 63.

Figure 28 shows the variation of tilt across the nuclear island common mat as a function of time (MT2 model). For three sections considered (AA, BB, DD), tilt observed increases in one direction, drops down to zero, and then increases again in the other direction. This variation in tilt is due to the loading sequence. As expected, the loading sequence significantly impacts the tilt observed. The largest tilt according to MT2 model is along the section CC, and towards NAB.

Tilt of ESWBs as a function of time is given in Figure 29 (MT2 model). As mentioned before, the tilt observed in 1ESWB and 2ESWB is more than the tilt observed in 3ESWB and 4ESWB, since the former two are closer to the highly loaded nuclear island area. Table 3 shows that the construction of ESWBs start at the fifth loading step. The tilt observed before the fifth loading step is expected to be eliminated during the foundation works of ESWBs, i.e., aforementiond maximum tilts for ESWBs are uncorrected. Corrected tilts were obtained by subtracting the tilt observed at the end of the fourth stage (just before the foundation construction for ESWBs starts) from the later stages (fifth, sixth, etc.). Uncorrected tilts are reported in Figure 29, since they are conservative estimates. The corrected maximum tilts are reported in Table 10-14 for the four models and the combined results.

Shown in Figure 30 is the uncorrected tilt for EPGBs (MT2 model). Both buildings have increasing tilt by time. Maximum uncorrected tilt for 1EPGB and 2EPGB according to MT2 model are 0.68 in / 50 ft and 0.72 in / 50 ft, respectively. The corrected tilts for EPGBs are 0.50 in / 50 ft and 0.43 in / 50 ft for EPGB1 and EPGB2, respectively.

Maximum corrected and uncorrected tilts are shown in Tables 10-14 for all buildings as determined by the results of the four models considered, and the combinations of these four models. The corrected and unccorected tilts in these tables are also shown in Figures 31 – 34. Figure 31 shows the maximum tilt in nuclear island as obtained by the four models and also the combination of the models. The combination of results represents the effect of surface topography. Sections AA and BB are influenced by the surface topography effect significantly, which have maximum tilts of 0.60 in / 50 ft and 1.16 in / 50 ft, respectively. As mentioned before, these numbers were obtained using the results from different models, and the effect of foundation stiffness is expected to be lower than this tilt. However, these numbers can still represent the upper bound for the tilt. Section CC has a tilt towards the SGB23 according to MT2 results. When the surface topography effect is considered, this tilt direction is reversed through NAB. In other words, the loads and subsurface profile induce a tilt in the opposite direction compared to the tilt induced by the surface topography effect for section CC. For both sections CC and DD, the estimated tilt is below 0.5 in / 50 ft. Section CC in Figure 31 represents the combination of NI and NAB, while Section DD represents the combination of NI and AB. If Sections CC and DD are considered for only NAB and AB buildings as shown in Figure 32, then, the tilt is well above 1 in independent from the model used, even after the construction sequence correction is applied.

The tilt for ESWBs are shown in Tables 10-14 and Figures 33 and 34. Sections FF, GG, and LL are significantly influenced by the surface topography effect. The tilt in 1ESWB and 2ESWB is higher than the tilt in 3ESWB and 4ESWB as stated previously. The construction sequence correction, as described before, significantly reduces the tilt estimate. The maximum corrected tilt estimate is 0.60 in / 50 ft for 1ESWB, 0.73 in / 50 ft for 2ESWB, and 0.18 in / 50 ft for 3ESWB. The effect of non-uniform loading and surface topography act in alignment on the tilt aling LL section, and the tilt is 0.46 in / 50 ft (corrected for construction sequence).

The tilt for EPGBs are shown in Tables 10-14 and Figures 35. The surface topography effect on the tilt of these buildings is small. The maximum corrected tilt estimates are similar for all models and combined results. The estimated maximum corrected tilt is 0.50 in / 50 ft and 0.43 in / 50 ft for 1EPGB and 2EPGB.

The settlement at the center of each building as a function of time is shown in Figure 36 for the MT2 model. Even though, LT model can result in larger settlements than those shown in Figure 36, MT2 model is proposed to be a better approximation to the site topography. However, to be more comprehensive, the settlements at the end of 8th loading step as obtained from all models and combined results are shown in Figure 37 (variation of settlement as a function of time is given in Tables 15-19). As expected, MT2 results provide an average settlement around the entire site for almost all buildings. According to MT2 model, NI, FB, SGB4 and NAB experience the highest settlements between 12 in and 13 in. The highest settlement occurs beneath FB. The lowest settlements occur beneath ESWBs. Uniform heave (rebound) of 1.2 in is expected due to groundwater recharge in the powerblock area from El. 38 ft to El. 55ft, but this is not shown in Figure 36 or Figure 37. Since rewatering was conducted in 20 years, no more additional consolidation is expected. The creep, or the secondary compression, is expected to be negligable based on the overconsolidated nature of the clay layer and the magnitude of loads applied. Appendix 2 shows that the ratio of final applied stress to the preconsolidation pressure mostly remains below 0.7 for Chesapeake Stratum IIb Sand and Chesapeake Stratum IIc Clay/Silt layers. For this stress condition, Ref.12 proposes that the secondary compression is not significant. The higher ratio deep in the

Nanjemoy sand layer (depth > 350 ft) may be due to the lower preconsolidation pressure measured at relatively shallow depth in that layer (only limited number of tests area available) and assigned to the entire layer.
The heave due to rewatering and the settlement due to creep may counterbalance in the long run, resulting in no additional settlement after the end of the construction during the lifetime of structures.
The amount of settlement at the center of nuclear island is shown in Figure 38 for all four models. The final settlement for all four models range between 10 in and 16.4 in. The MT2 model (average or the best estimate model) predicts 12.7 in of settlement under the center of NI at the end of load step 8.

Figure 39 shows the progression of settlement throughout the 8-step loading and rewatering stages. Up to the end of fourth stage, the effect of backfill placement is distinctly reflected and settlements are more uniform throughout the power block area. Applied loads do not induce a distinguishable settlement pattern up to the end of Step 4 (500 days). However, after Step 5 (800 days), the effect of building loads on settlement pattern becomes more pronounced compared to the effect of backfill load. Tilt becomes evident in the direction of SGB23 and SGB1. After Step 7 (1400 days), tilt pattern changes, and the nuclear island common mat starts tilting towards NAB. After Step 8, tilt is still towards NAB, but the settlement throughout the nuclear island common mat becomes more uniform. Also seen in Figure 39, is the more significant effect of nuclear island loading on the 1EPGB compared to 2EPGB, and on the 2ESWB compared to 3ESWB, confirming the discussions made based upon Figures 25 through 27.

5 CONCLUSIONS

- PLAXIS3D provides a realistic 3D representation of the subsurface conditions and the building geometries. The model is capable of capturing variation of soil properties below the footprints of the foundation and therefore it is possible to better assess differential settlement.
- There is a direct correlation with the load schedule and the amount of settlement beneath different points throughout the foundation footprint. Settlement does not occur uniformly throughout the footprint.
- Tilt is the main indicator of the Design Certification of the US EPR. The current revision of the US EPR FSAR provides a limit of 0.5 inches per 50 feet:

  > "A COL applicant that references the U.S. EPR design certification will verify that the differential settlement value of ½ inch per 50 ft in any direction across the foundation basemat of a Seismic Category I structure is not exceeded. Settlement values larger than this may be demonstrated acceptable by performing additional site-specific evaluations."

- The maximum tilt for the sections going through NI is below 1.16 in / 50 ft (Table 14), and it is the tilt between NI common mat and TB. For the other considered cross sections going through NI, the tilt is below 0.6 in / 50 ft (Table 14). For EPGBs, 3ESWB, and 4ESWB the tilt is below 0.5 in / 50 ft (Tables 10-14).
- For 1ESWB and 2ESWB the tilt is below 0.75 in / 50 ft (Tables 10-14).
- The most critical buildings in terms of tilt are NAB and AB, which may experience tilt between 1.2 in/ 50 ft and 1.5 in/ 50 ft (Tables 10-14).
- The settlement estimates under the center of the buildings ranges between 10 in and 16 in for the power block area (Tables 15 - 19). The settlement under the center of the nuclear reactor is expected to be about 13 in (Tables 17).

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Table 1. Plaxis 3D main soil constitutive models

Model	Feature	Application
Linear Elastic	Isotropic, linear elastic behavior	Common engineering materials such as steel and concrete
Mohr-Coulomb	Traditional soil behavior model: Cohesion Friction Young's Modulus Poisson Ratio Dilatancy Angle	Geotechnical engineering applications for estimation of shear strength and settlement
Hardening Soil	Elastoplastic model	Friction hardening and irreversible compaction under compression

Table 2- Building Loads

Building	Elevation (ft msl)	Depth (ft) (1)	Area (ft^2) (2)	Total load (DL+25%LL) (kips) (2)	Uniformly distributed load (ksf)
Reactor Building	41.5	41.5	26266	313,477	11.9
Fuel Building, FB	41.5	41.5	14547	216,806	14.9
Safeguard 1, SGB1	41.5	41.5	9,198	108,064	11.7
Safeguard 2&3, SGB23	41.5	41.5	20,910	200,814	9.6
Safeguard 4, SGB4	41.5	41.5	9,247	104,079	11.3
Total of NI	-	-	80,167	943,240	11.8
Nuclear Aux. Building, NAB	48.0	35.0	12,559	122,000	9.7
Rad. Waste Building, RWPB	47.0	36.0	16,970	109,700	6.5
Access Building, AB	48.0	35.0	7,620	49,300	6.5
Turbine Building	60.5	22.5	101,305	446,600	4.4
Turbine Building Extension	66.5	16.5	28320	214,213	4.4
Emergency Power Generating Building, EPGB	76.0	7.0	12,611	40,200	3.2
Emergency Service Water Building, ESWB	61.0	22.0	16,284	88,700	5.4

(1) Depth from surface elevation of 83 ft.

(2) Ref. 8. Areas shown here are not exactly the same with the FEM model of the nuclear island common mat, due to idealization of the circular geometry of the nuclear reactor. However, the deviation is less than 5 %.

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Table 3 - Time-dependant uniformly distributed loads for CCNPP Unit 3 buildings, all pressures are in kips/ft^2 (Ref. 8)

Building	Step =	0	1	2	3	4	5	6	7	8
	Day =	0	60	140	300	500	800	1000	1400	2000
	Week =	0	9	20	43	71	114	143	200	286
Reactor Building		0.00	0.14	0.21	2.07	3.82	7.49	9.61	9.82	11.93
Fuel Building, FB		0.00	0.00	0.98	1.66	1.99	5.68	8.97	14.91	14.91
Safeguard 1, SGB1		0.00	0.00	0.83	3.77	5.86	8.56	11.75	11.75	11.75
Safeguard 2&3, SGB23		0.00	0.00	1.03	1.68	2.55	5.36	8.24	9.58	9.58
Safeguard 4, SGB4		0.00	0.00	0.80	3.61	5.61	8.20	11.26	11.26	11.26
Nuclear Aux. Building, NAB		0.00	0.00	0.88	1.77	3.53	5.30	7.06	9.71	9.71
Access Building, AB		0.00	0.00	0.92	1.85	2.77	4.62	5.55	6.47	6.47
Rad. Waste Building, RWPB		0.00	0.00	0.00	0.00	0.00	0.72	2.15	6.46	6.46
Emergency Power Generating Building, EPGB		0.00	0.00	0.00	0.00	0.00	0.00	0.53	3.19	3.19
Emergency Service Water Building, ESWB		0.00	0.00	0.00	0.00	0.00	1.60	5.45	5.45	5.45
Turbine Building,		0.00	0.00	0.00	0.59	1.76	3.23	4.41	4.41	4.41
Turbine Building Extension		0.00	0.00	0.00	0.00	1.76	3.23	4.41	4.41	4.41

Table 4 - Properties of wall elements

Element	Thickness [ft]	E [psf]	v
Wall	5	1.00E+09	0.15

Table 5 – Increments in foundation stiffness

Step	Young's Modulus (psf)	Thickness (ft)	Reinforcing Walls ?
A (Flexible)	5.8×10^8	6	NO
B (Stiff)	2.0×10^{10}	10	NO
C (Rigid)	2.0×10^{10}	10	YES

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Table 6 – Variation of foundation stiffness for different buildings

Building	Step =	0	1	2	3	4	5	6	7	8
	Day =	0	60	140	300	500	800	1000	1400	2000
	Week =	0	9	20	43	71	114	143	200	286
Reactor Building,		A	A	A	B	C	C	C	C	C
Fuel Building, FB		A	A	A	B	C	C	C	C	C
Safeguard 1, SGB1		A	A	A	B	C	C	C	C	C
Safeguard 2&3, SGB23		A	A	A	B	C	C	C	C	C
Safeguard 4, SGB4		A	A	A	B	C	C	C	C	C
Nuclear Aux. Building, NAB		A	A	A	B	B	B	B	B	B
Access Building, AB		A	A	A	B	B	B	B	B	B
Rad. Waste Building, RWPB		A	A	A	B	B	B	B	B	B
Emergency Power Generating Building, EPGB		A	A	A	B	B	B	B	B	B
Emergency Service Water Building, ESWB		A	A	A	B	B	B	B	B	B
Turbine Building,		A	A	A	B	B	B	B	B	B
Turbine Building Extension		A	A	A	B	B	B	B	B	B

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Table 7- Soil Properties (Mohr-Coulomb constitutive model)

LAYER	Unit Weight [pcf] (1)	$K_{x\ or\ z}$ [ft/day] (2)	K_y [ft/day] (2)	c' [psf] (3)	ϕ' [°] (3)	E_r [psf] (4)	E [psf] (5)	ν'	Loading Conditions (7)
Backfill(6)	141	820.021	82.021	0	44.0	1.92E+06	1.92E+06	0.30	Drained
IIb - Chesapeake Cemented Sand *(1)*	122	0.851	0.0851	600	33.5	7.59E+06	2.53E+06	0.30	Drained
IIb - Chesapeake Cemented Sand *(2)*	123	0.851	0.0851	520	32.3	4.60E+06	1.03E+06	0.30	Drained
IIb - Chesapeake Cemented Sand *(3)*	123	0.851	0.0851	220	31.7	1.02E+07	2.62E+06	0.30	Drained
IIc - Chesapeake Clay/Silt	104	1.4E-04	1.4E-04	800	32.0	7.11E+06	2.37E+06	0.35	Undrained
IIc - Chesapeake Clay/Silt - Sand	104	0.085	0.0085	800	32.0	7.11E+06	2.37E+06	0.30	Drained
II - Nanjemoy Sand	127	0.085	0.0085	0	40.0(8)	9.70E+06	3.17E+06	0.30	Drained

(1) Ref. 4

(2) $K_{x\ or\ z}$ = hydraulic conductivity in horizontal planes, K_y = vertical hydraulic conducitivity. Hydraulic conductivity for the backfill was obtained from Ref. 10 whereas hydraulic conductivities of all other layers were obtained from Ref. 7

(3) Ref. 6

(4) Ref. 9

(5) Ref. 8

(6) Ref. 10 (Backfill identified as GAB was chosen since its both hdyraulic conductivity and strength properties were high).

(7) Conditions indicating pore pressure evaluation during analysis.

(8) Estimated based on SPT values.

Table 8 - Variations of immediate settlement and heave for ED model

Node ID	Vertical Displacement [in]			
	Excavation/Dewatering			
	Immediate	End of Consolidation	6 Months after Excavation	1 Year After Excavation
A	-2.18	-2.40	-2.38	-2.39
B	-3.04	-3.49	-3.47	-3.49
C	-4.68	-5.32	-5.30	-5.32
D	-3.14	-3.52	-3.49	-3.50

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Table 9 – Model selection for 88 Nodes Analyzed

Node Number	Original Ground Surface Elevation, (ft)	Building	Elevation (ft)	Initial Effective Stress at El. 41.5 ft (psf)	Corresponding Model[1]
1	95	NI	41.5	3910.6	HT
2	95	NI	41.5	3910.6	HT
3	105	NI	41.5	5090.6	HT
4	85	NI	41.5	2730.6	MT2
5	90	NI	41.5	3320.6	HT
6	90	NI	41.5	3320.6	MT2
7	110	NI	41.5	5680.6	HT
8	80	NI	41.5	2140.6	MT2
9	80	NI	41.5	2140.6	MT2
10	95	SGB1	41.5	3910.6	MT2
11	88	SGB1	41.5	3084.6	MT2
12	98	SGB1	41.5	4264.6	MT2
13	105	SGB1	41.5	5090.6	MT2
14	105	SGB1	41.5	5090.6	MT2
15	100	SGB1	41.5	4500.6	MT2
16	100	SGB1	41.5	4500.6	MT2
17	110	SGB2	41.5	5680.6	HT
18	100	SGB2	41.5	4500.6	MT2
19	108	SGB2	41.5	5444.6	HT
20	110	SGB2	41.5	5680.6	HT
21	110	SGB2	41.5	5680.6	HT
22	60	SGB4	41.5	1028.6	LT
23	66	SGB4	41.5	1362.2	MT1
24	75	SGB4	41.5	1862.6	MT1
25	100	SGB4	41.5	4500.6	HT
26	70	SGB4	41.5	1584.6	LT
27	68	SGB4	41.5	1473.4	LT
28	70	FB	41.5	1584.6	LT
29	70	FB	41.5	1584.6	LT
30	80	FB	41.5	2140.6	MT1
31	85	FB	41.5	2730.6	MT1
32	70	FB	41.5	1584.6	MT1
33	66	FB	41.5	1362.2	MT1

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Table 9 continued– Model selection for 88 Nodes Analyzed

Node Number	Original Ground Surface Elevation, (ft)	Building	Elevation (ft)	Initial Effective Stress at El. 41.5 ft (psf)	Corresponding Model[1]
34	66	FB	41.5	1362.2	MT1
35	65	NAB	48	1306.6	LT
36	70	NAB	48	1584.6	LT
37	66	NAB	48	1362.2	LT
38	66	NAB	48	1362.2	LT
39	66	NAB	48	1362.2	LT
40	58	NAB	48	917.4	LT
41	58	NAB	48	917.4	LT
42	60	NAB	48	1028.6	LT
43	75	AB	48	1862.6	LT
44	95	AB	48	3910.6	MT2
45	95	AB	48	3910.6	MT2
46	105	AB	48	5090.6	HT
47	110	AB	48	5680.6	HT
48	85	AB	48	2730.6	LT
49	55	RWPB	47	750.6	LT
50	50	RWPB	47	472.6	LT
51	60	RWPB	47	1028.6	LT
52	58	RWPB	47	917.4	LT
53	50	RWPB	47	472.6	LT
54	65	1EPBG	76	1306.6	LT
55	58	1EPBG	76	917.4	LT
56	65	1EPBG	76	1306.6	LT
57	70	1EPBG	76	1584.6	LT
58	58	1EPBG	76	917.4	LT
59	110	2EPBG	76	5680.6	HT
60	110	2EPBG	76	5680.6	HT
61	110	2EPBG	76	5680.6	HT
62	110	2EPBG	76	5680.6	HT
63	110	2EPBG	76	5680.6	HT

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Table 9 continued– Model selection for 88 Nodes Analyzed

Node Number	Original Ground Surface Elevation, (ft)	Building	Elevation (ft)	Initial Effective Stress at El. 41.5 ft (psf)	Corresponding Model[1]
64	84	4ESWB	61	2612.6	LT
65	75	4ESWB	61	1862.6	LT
66	72	4ESWB	61	1695.8	LT
67	88	4ESWB	61	3084.6	MT1
68	94	4ESWB	61	3792.6	MT2
69	75	3ESWB	61	1862.6	LT
70	70	3ESWB	61	1584.6	LT
71	60	3ESWB	61	1028.6	LT
72	74	3ESWB	61	1807	LT
73	82	3ESWB	61	2376.6	LT
74	95	1ESWB	61	3910.6	MT2
75	90	1ESWB	61	3320.6	MT2
76	95	1ESWB	61	3910.6	MT2
77	100	1ESWB	61	4500.6	HT
78	86	1ESWB	61	2848.6	MT2
79	105	2ESWB	61	5090.6	HT
80	95	2ESWB	61	3910.6	MT2
81	110	2ESWB	61	5680.6	HT
82	110	2ESWB	61	5680.6	HT
83	105	2ESWB	61	5090.6	HT
84	85	TB	60.5	2730.6	LT
85	95	TB	60.5	3910.6	MT2
86	108	TB	60.5	5444.6	HT
87	84	TB	60.5	2612.6	LT
88	90	TB	60.5	3320.6	MT1

(1) LT = Low topography model
MT1 = Medium topography model 1
MT2 = Medium topography model 2
HT = High topography model

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Table 10. Tilts observed at the end of 8th loading step for different buildings, LT Model

Building Name	Section Label	Tilt (in / 50 ft)		Tilt Direction
		Uncorrected (1)	Corrected(1)	
Nuclear Island Common Mat	AA	0.07	0.07	North
	BB	0.25	0.25	East
	CC	0.34	0.34	South West
	DD	0.36	0.36	South East
Nuclear Auxiliary Building	CC	1.38	1.42	South West
Access Building	DD	1.18	1.24	South East
Emergency Service Water Building 1	EE	0.95	0.60	North West
	FF	0.15	0.18	North East
Emergency Service Water Building 2	GG	0.32	0.24	South West
	HH	1.12	0.73	North West
Emergency Service Water Building 3	II	0.05	0.17	North East
	JJ	0.28	0.13	South East
Emergency Service Water Building 4	KK	0.42	0.28	South West
	LL	0.58	0.42	South East
Emergency Power Generating Building 1	MM	0.41	0.16	South West
	NN	0.77	0.49	South East
Emergency Power Generating Building 2	OO	0.84	0.42	North East
	PP	0.50	0.14	North West

(1) Correction to subtract the observed tilt before the construction of the building.

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Table 11. Tilts observed at the end of 8th loading step for different buildings, MT1 Model

Building Name	Section Label	Tilt (in / 50 ft)		Tilt Direction
		Uncorrected (1)	Corrected(1)	
Nuclear Island Common Mat	AA	0.09	0.09	North
	BB	0.25	0.25	East
	CC	0.27	0.27	South West
	DD	0.33	0.33	South East
Nuclear Auxiliary Building	CC	1.23	1.26	South West
Access Building	DD	1.09	1.16	South East
Emergency Service Water Building 1	EE	0.88	0.60	North West
	FF	0.15	0.18	North East
Emergency Service Water Building 2	GG	0.31	0.23	South West
	HH	1.01	0.73	North West
Emergency Service Water Building 3	II	0.08	0.17	North East
	JJ	0.24	0.13	South East
Emergency Service Water Building 4	KK	0.40	0.28	South West
	LL	0.56	0.42	South East
Emergency Power Generating Building 1	MM	0.38	0.16	South West
	NN	0.71	0.49	South East
Emergency Power Generating Building 2	OO	0.77	0.42	North East
	PP	0.45	0.14	North West

(1) Correction to subtract the observed tilt before the construction of the building.

Table 12. Tilts observed at the end of 8th loading step for different buildings, MT2 Model

Building Name	Section Label	Tilt (in / 50 ft)		Tilt Direction
		Uncorrected (1)	Corrected(1)	
Nuclear Island Common Mat	AA	0.10	0.10	North
	BB	0.27	0.27	East
	CC	0.21	0.21	South West
	DD	0.32	0.32	South East
Nuclear Auxiliary Building	CC	1.10	1.13	South West
Access Building	DD	1.06	1.13	South East
Emergency Service Water Building 1	EE	0.80	0.59	North West
	FF	0.17	0.18	North East
Emergency Service Water Building 2	GG	0.29	0.23	South West
	HH	0.88	0.72	North West
Emergency Service Water Building 3	II	0.13	0.17	North East
	JJ	0.19	0.13	South East
Emergency Service Water Building 4	KK	0.36	0.28	South West
	LL	0.53	0.42	South East
Emergency Power Generating Building 1	MM	0.35	0.16	South West
	NN	0.68	0.49	South East
Emergency Power Generating Building 2	OO	0.72	0.42	North East
	PP	0.37	0.14	North West

(1) Correction to subtract the observed tilt before the construction of the building.

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Table 13. Tilts observed at the end of 8th loading step for different buildings, HT Model

Building Name	Section Label	Tilt (in / 50 ft)		Tilt Direction
		Uncorrected (1)	Corrected(1)	
Nuclear Island Common Mat	AA	0.12	0.12	North
	BB	0.26	0.26	East
	CC	0.11	0.11	South West
	DD	0.25	0.25	South East
Nuclear Auxiliary Building	CC	0.82	0.86	South West
Access Building	DD	0.83	0.90	South East
Emergency Service Water Building 1	EE	0.65	0.44	North West
	FF	0.14	0.15	North East
Emergency Service Water Building 2	GG	0.23	0.17	South West
	HH	0.67	0.52	North West
Emergency Service Water Building 3	II	0.13	0.18	North East
	JJ	0.15	0.09	South East
Emergency Service Water Building 4	KK	0.30	0.21	South West
	LL	0.44	0.33	South East
Emergency Power Generating Building 1	MM	0.25	0.15	South West
	NN	0.54	0.49	South East
Emergency Power Generating Building 2	OO	0.55	0.42	North East
	PP	0.28	0.14	North West

(1) Correction to subtract the observed tilt before the construction of the building.

Table 14. Tilts observed at the end of 8th loading step for different buildings, combined results

Building Name	Section Label	Tilt (in / 50 ft)		Tilt Direction
		Uncorrected (1)	Corrected(1)	
Nuclear Island Common Mat	AA	0.59	0.59	North
	BB	1.13	1.13	East
	CC	0.17	0.17	South West
	DD	0.12	0.12	North West
Nuclear Auxiliary Building	CC	1.38	1.42	South West
Access Building	DD	1.77	1.71	South East
Emergency Service Water Building 1	EE	0.65	0.44	North West
	FF	0.56	0.57	North East
Emergency Service Water Building 2	GG	0.25	0.31	North East
	HH	0.88	0.72	North West
Emergency Service Water Building 3	II	0.05	0.17	South West
	JJ	0.28	0.13	South East
Emergency Service Water Building 4	KK	0.14	0.28	North East
	LL	1.23	0.46	South East
Emergency Power Generating Building 1	MM	0.41	0.16	South West
	NN	0.77	0.49	South East
Emergency Power Generating Building 2	OO	0.55	0.42	North East
	PP	0.28	0.14	North West

(1) Correction to subtract the observed tilt before the construction of the building.

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Table 15. Final settlements under the centers of each building foundation, LT Model

Building Name		Settlement (in)							
		Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Reactor Building	REACTOR	0.3	1.1	3.4	6.5	10.7	13.8	15.8	16.4
Fuel Building	FB	0.3	1.3	3.4	6.2	10.2	13.2	15.8	16.4
Safe Guard Building 1	SGB1	0.3	1.4	3.7	6.8	10.7	13.8	15.1	15.7
Safe Guard Building 2 and 3	SGB23	0.3	1.4	3.5	6.6	10.5	13.5	14.7	15.1
Safe Guard Building 4	SGB4	0.3	1.3	3.6	6.6	10.4	13.4	15.6	16.1
Nuclear Auxiliary Building	NAB	0.4	1.4	3.4	6.2	9.7	12.4	15.2	15.6
Access Building	AB	0.4	1.6	3.7	6.8	10.5	13.3	14.9	15.3
Radioactive Waste Building	RWB	0.5	1.3	2.9	5.2	7.8	9.6	12.1	12.3
Emergency Service Water Building 1	ESWB1	0.0	1.6	2.6	4.1	6.3	8.7	9.1	9.1
Emergency Service Water Building 2	ESWB2	0.0	1.7	2.9	4.8	7.5	10.3	11.0	11.1
Emergency Service Water Building 3	ESWB3	0.0	2.0	3.2	5.4	8.2	11.0	11.4	11.5
Emergency Service Water Building 4	ESWB4	0.0	1.9	2.9	4.8	7.3	9.8	10.1	10.1
Emergency Power Generating Building 1	EPGB1	0.0	0.0	0.0	6.8	9.0	10.7	12.6	12.7
Emergency Power Generating Building 2	EPGB2	0.0	0.0	0.0	6.6	8.7	10.1	11.6	11.7

(1) Rewatering takes place in 20 years.

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Table 16. Final settlements under the centers of each building foundation, MT1 Model

Building Name		Settlement (in)							
		Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Reactor Building	REACTOR	0.3	1.1	2.0	5.0	9.1	12.2	14.1	14.8
Fuel Building	FB	0.3	1.3	2.0	4.8	8.7	11.7	14.3	14.9
Safe Guard Building 1	SGB1	0.3	1.4	2.3	5.2	9.1	12.1	13.5	14.0
Safe Guard Building 2 and 3	SGB23	0.3	1.4	2.2	5.1	9.0	12.0	13.2	13.6
Safe Guard Building 4	SGB4	0.3	1.3	2.2	5.1	8.9	11.8	14.1	14.5
Nuclear Auxiliary Building	NAB	0.4	1.4	2.2	4.9	8.3	11.0	13.8	14.2
Access Building	AB	0.4	1.6	2.4	5.3	9.0	11.8	13.4	13.8
Radioactive Waste Building	RWB	0.5	1.3	1.9	4.2	6.7	8.5	11.0	11.2
Emergency Service Water Building 1	ESWB1	0.0	1.6	2.0	3.5	5.7	8.1	8.4	8.4
Emergency Service Water Building 2	ESWB2	0.0	1.7	2.2	3.9	6.6	9.5	10.1	10.3
Emergency Service Water Building 3	ESWB3	0.0	2.0	2.5	4.5	7.3	10.1	10.5	10.6
Emergency Service Water Building 4	ESWB4	0.0	1.9	2.3	4.0	6.5	9.1	9.3	9.3
Emergency Power Generating Building 1	EPGB1	0.0	0.0	0.0	5.5	7.8	9.4	11.4	11.5
Emergency Power Generating Building 2	EPGB2	0.0	0.0	0.0	5.4	7.5	8.9	10.4	10.5

(1) Rewatering takes place in 20 years.

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Table 17. Final settlements under the centers of each building foundation, MT2 Model

Building Name		Settlement (in)							
		Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Reactor Building	REACTOR	0.3	1.1	2.0	3.1	7.1	10.2	12.1	12.7
Fuel Building	FB	0.3	1.3	2.0	3.0	6.8	9.8	12.4	13.0
Safe Guard Building 1	SGB1	0.3	1.4	2.3	3.3	7.1	10.1	11.4	12.0
Safe Guard Building 2 and 3	SGB23	0.3	1.4	2.2	3.2	7.0	9.9	11.1	11.6
Safe Guard Building 4	SGB4	0.3	1.3	2.2	3.2	6.9	9.8	12.1	12.5
Nuclear Auxiliary Building	NAB	0.4	1.4	2.2	3.2	6.5	9.1	12.0	12.3
Access Building	AB	0.4	1.6	2.4	3.5	7.0	9.8	11.4	11.7
Radioactive Waste Building	RWB	0.5	1.3	1.9	2.7	5.1	6.9	9.4	9.6
Emergency Service Water Building 1	ESWB1	0.0	1.6	2.0	2.6	4.7	7.0	7.4	7.4
Emergency Service Water Building 2	ESWB2	0.0	1.7	2.2	2.9	5.5	8.3	8.9	9.1
Emergency Service Water Building 3	ESWB3	0.0	2.0	2.5	3.3	6.0	8.8	9.1	9.2
Emergency Service Water Building 4	ESWB4	0.0	1.9	2.3	3.0	5.3	7.9	8.1	8.2
Emergency Power Generating Building 1	EPGB1	0.0	0.0	0.0	3.8	5.9	7.6	9.5	9.6
Emergency Power Generating Building 2	EPGB2	0.0	0.0	0.0	3.7	5.7	7.1	8.5	8.7

(1) Rewatering takes place in 20 years.

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Table 18. Final settlements under the centers of each building foundation, HT Model

Building Name		Settlement (in)							
		Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Reactor Building	REACTOR	0.3	1.1	2.0	3.1	4.6	7.5	9.4	10.0
Fuel Building	FB	0.3	1.3	2.0	3.0	4.4	7.2	9.8	10.3
Safe Guard Building 1	SGB1	0.3	1.4	2.3	3.3	4.7	7.5	8.8	9.3
Safe Guard Building 2 and 3	SGB23	0.3	1.4	2.2	3.2	4.6	7.4	8.5	9.0
Safe Guard Building 4	SGB4	0.3	1.3	2.2	3.2	4.6	7.3	9.5	10.0
Nuclear Auxiliary Building	NAB	0.4	1.4	2.2	3.2	4.3	6.8	9.6	10.0
Access Building	AB	0.4	1.6	2.4	3.5	4.8	7.3	8.9	9.3
Radioactive Waste Building	RWB	0.5	1.3	1.9	2.7	3.6	5.2	7.7	7.9
Emergency Service Water Building 1	ESWB1	0.0	1.6	2.0	2.6	3.4	5.7	6.0	6.0
Emergency Service Water Building 2	ESWB2	0.0	1.7	2.2	2.9	3.9	6.5	7.2	7.3
Emergency Service Water Building 3	ESWB3	0.0	2.0	2.5	3.3	4.3	7.0	7.4	7.4
Emergency Service Water Building 4	ESWB4	0.0	1.9	2.3	3.0	3.9	6.3	6.5	6.6
Emergency Power Generating Building 1	EPGB1	0.0	0.0	0.0	3.8	4.6	6.0	7.9	8.1
Emergency Power Generating Building 2	EPGB2	0.0	0.0	0.0	3.7	4.4	5.7	7.1	7.2

(1) Rewatering takes place in 20 years.

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Table 19. Final settlements under the centers of each building foundation, combined results

Building Name		Settlement (in)							
		Step 1	Step 2	Step 3	Step 4	Step 5	Step 6	Step 7	Step 8
Reactor Building	REACTOR	0.3	1.1	2.0	3.1	4.6	7.5	9.4	10.0
Fuel Building	FB	0.3	1.3	2.0	4.8	8.7	11.7	14.3	14.9
Safe Guard Building 1	SGB1	0.3	1.4	2.3	3.3	7.1	10.1	11.4	12.0
Safe Guard Building 2 and 3	SGB23	0.3	1.4	2.2	3.2	4.6	7.4	8.5	9.0
Safe Guard Building 4	SGB4	0.3	1.3	2.2	5.1	8.9	11.8	14.1	14.5
Nuclear Auxiliary Building	NAB	0.4	1.4	3.4	6.2	9.7	12.4	15.2	15.6
Access Building	AB	0.4	1.6	2.4	3.5	7.0	9.8	11.4	11.7
Radioactive Waste Building	RWB	0.5	1.3	2.9	5.2	7.8	9.6	12.1	12.3
Emergency Service Water Building 1	ESWB1	0.0	1.6	2.0	2.6	3.4	5.7	6.0	6.0
Emergency Service Water Building 2	ESWB2	0.0	1.7	2.2	2.9	5.5	8.3	8.9	9.1
Emergency Service Water Building 3	ESWB3	0.0	2.0	3.2	5.4	8.2	11.0	11.4	11.5
Emergency Service Water Building 4	ESWB4	0.0	1.9	2.9	4.8	7.3	9.8	10.1	10.1
Emergency Power Generating Building 1	EPGB1	0.0	0.0	0.0	6.8	9.0	10.7	12.6	12.7
Emergency Power Generating Building 2	EPGB2	0.0	0.0	0.0	3.7	4.4	5.7	7.1	7.2

(1) Rewatering takes place in 20 years.

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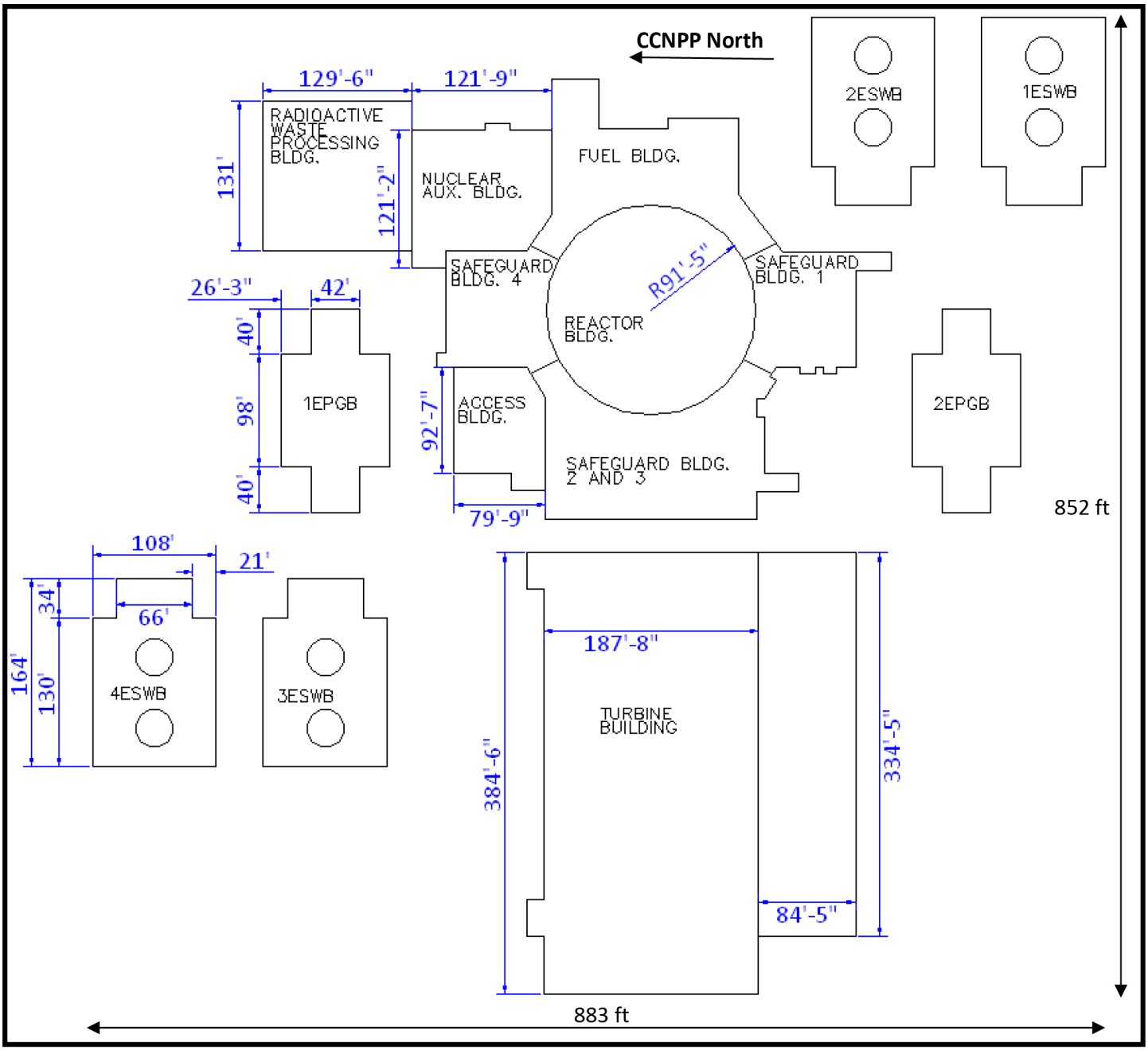

Figure 1 – Power Block Area

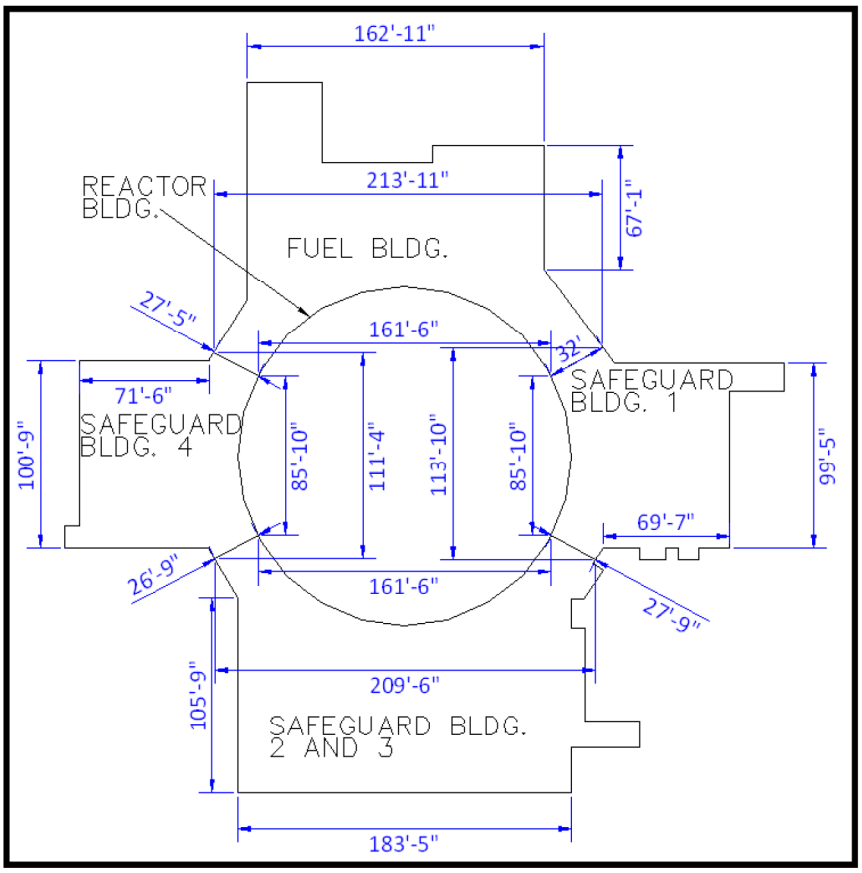

Figure 2 – NI Common Mat Dimensions

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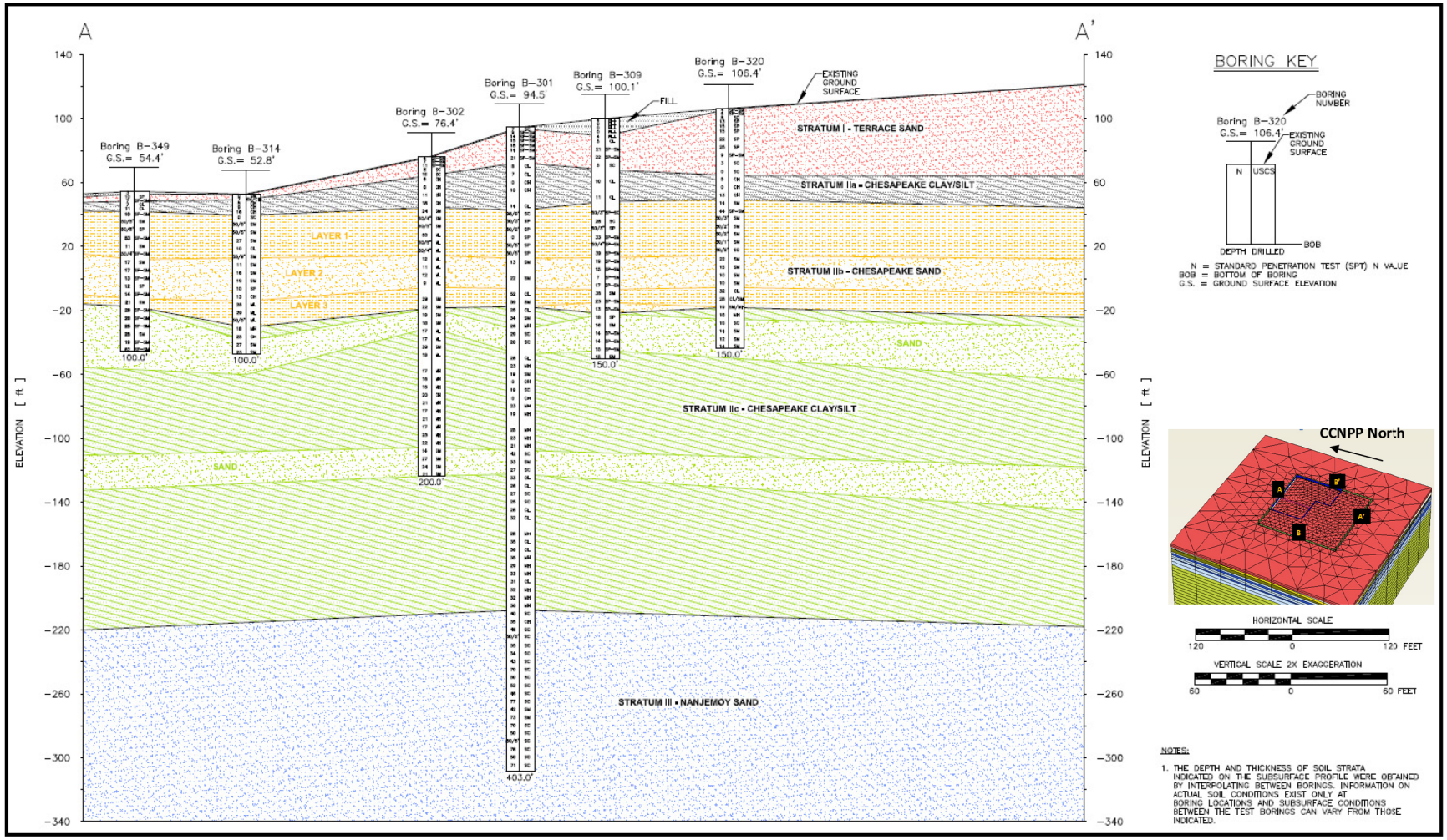

Figure 3- Section AA soil profile

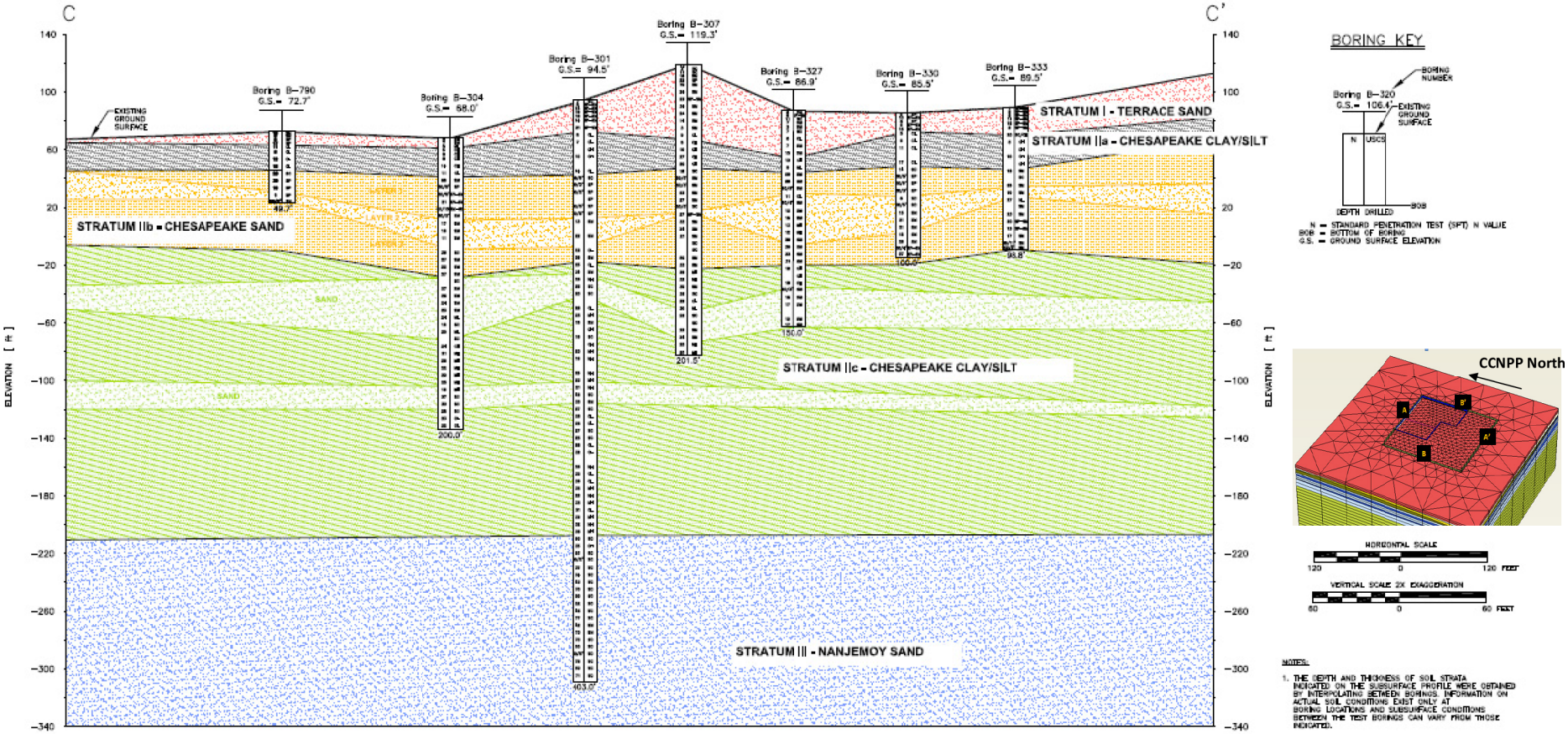

Figure 4 - Section BB soil profile

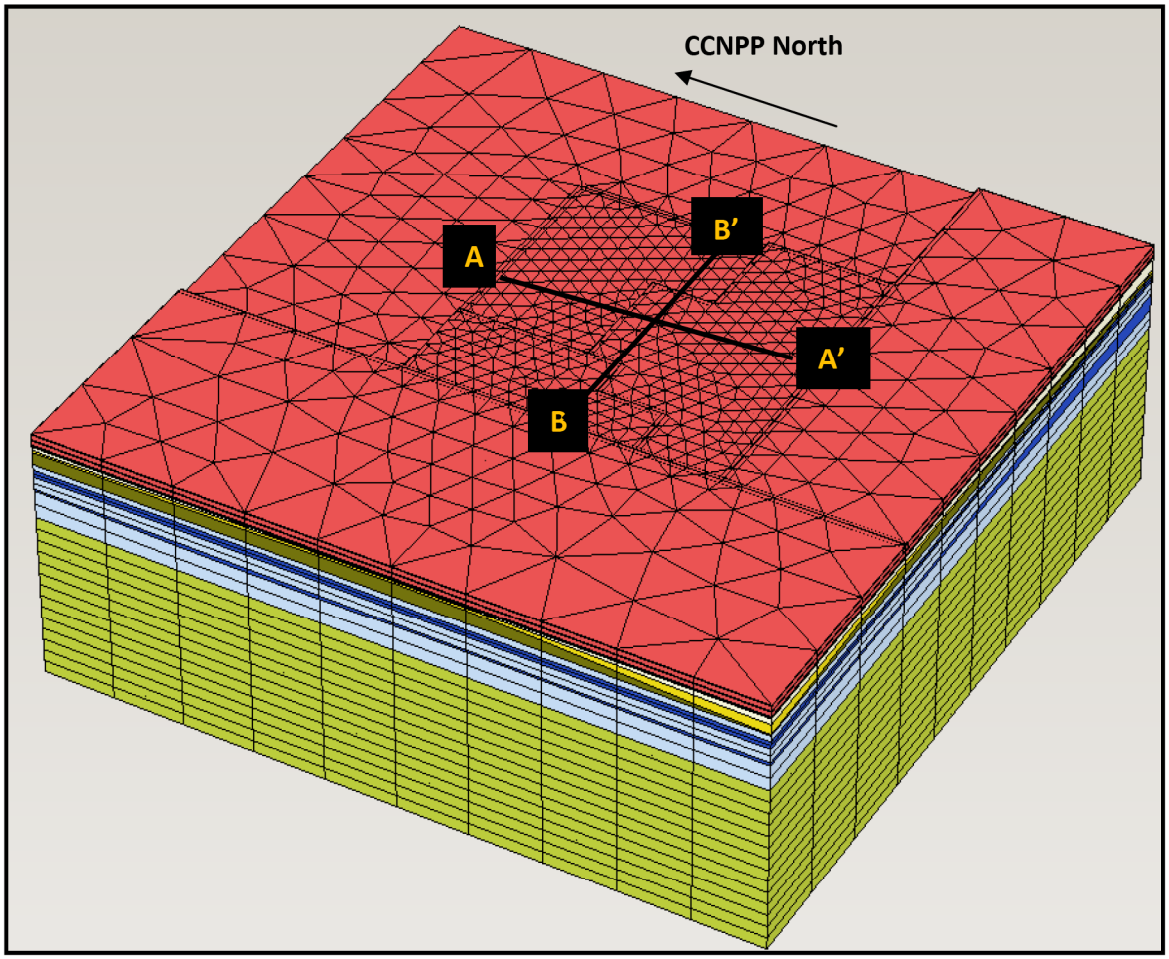

Figure 5 – Three dimensional subsurface model

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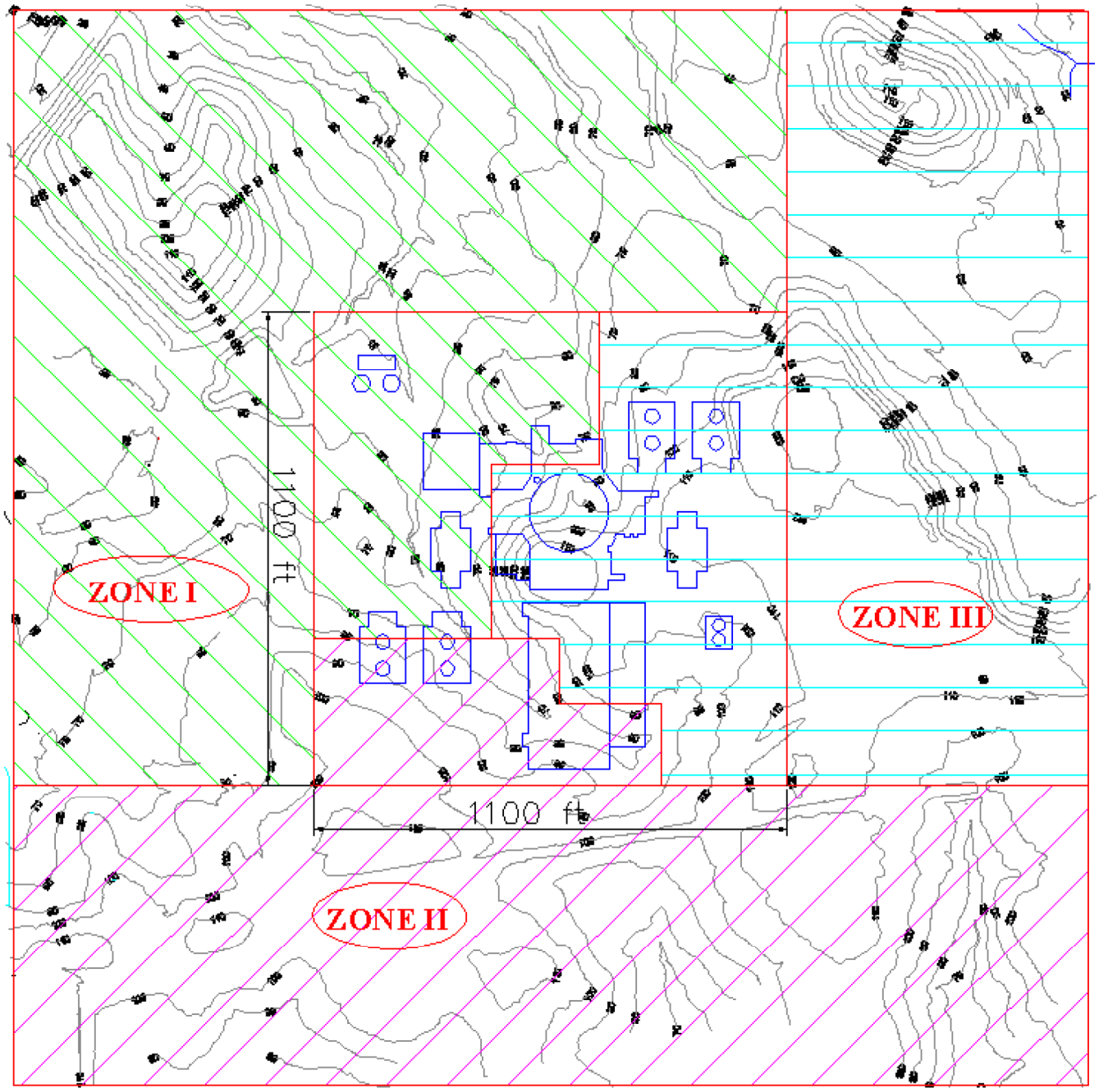

Figure 6 - Current ground surface elevations and zones considered in the excavation analysis (ED model) (Ref. 3).

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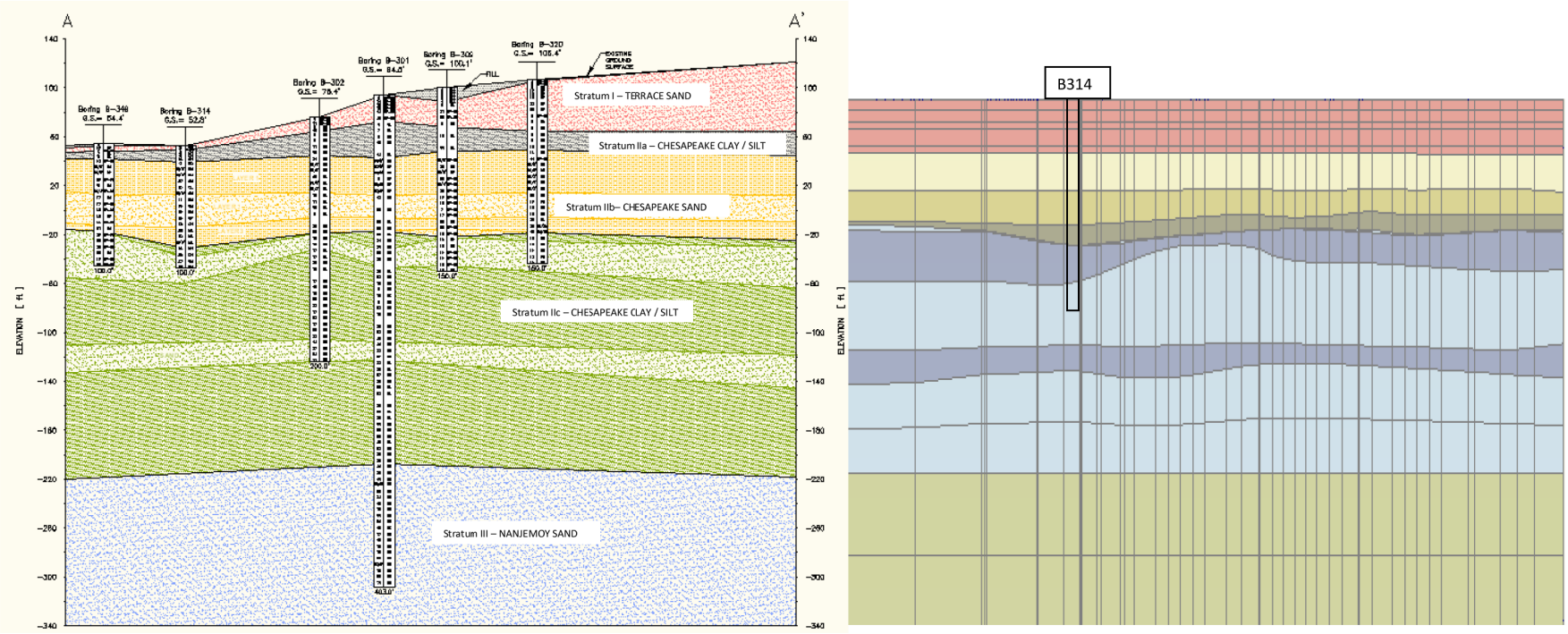

(a) (b)

Figure 7 – Soil cross-section A-A (a) according to boring logs, and (b) modeled in Plaxis

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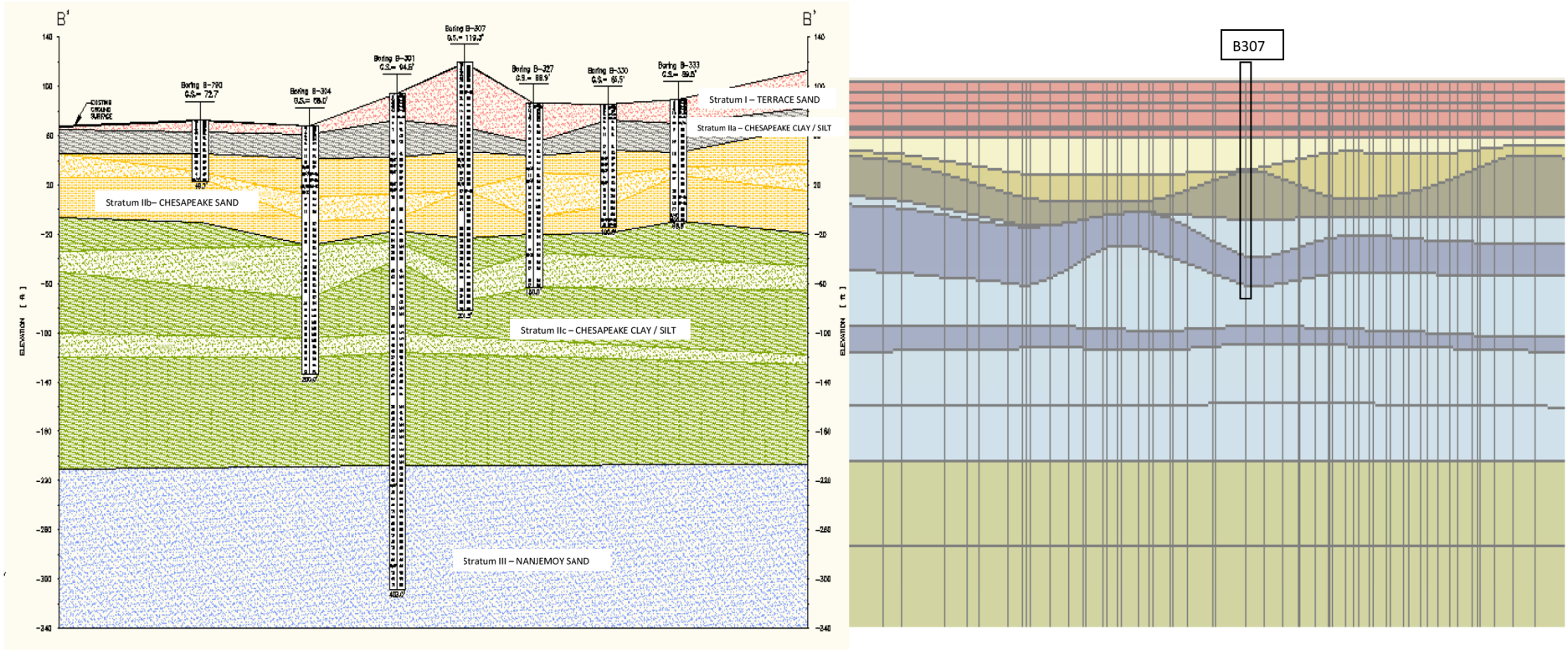

(a) (b)

Figure 8 – Soil cross-section B-B (a) according to boring logs, and (b) modeled in Plaxis

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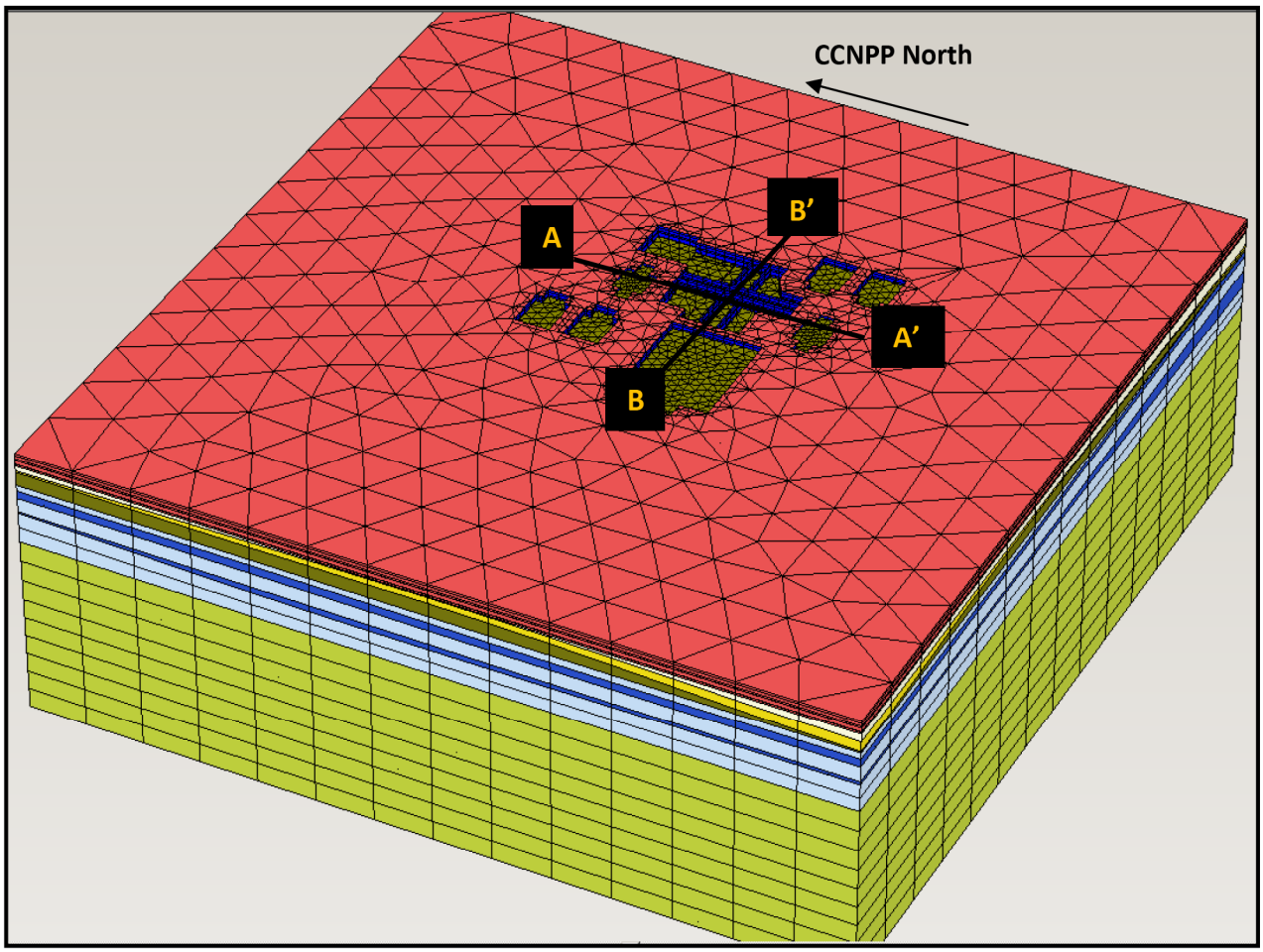

Figure 9 - Three dimensional model with power block area building locations (PC Model)

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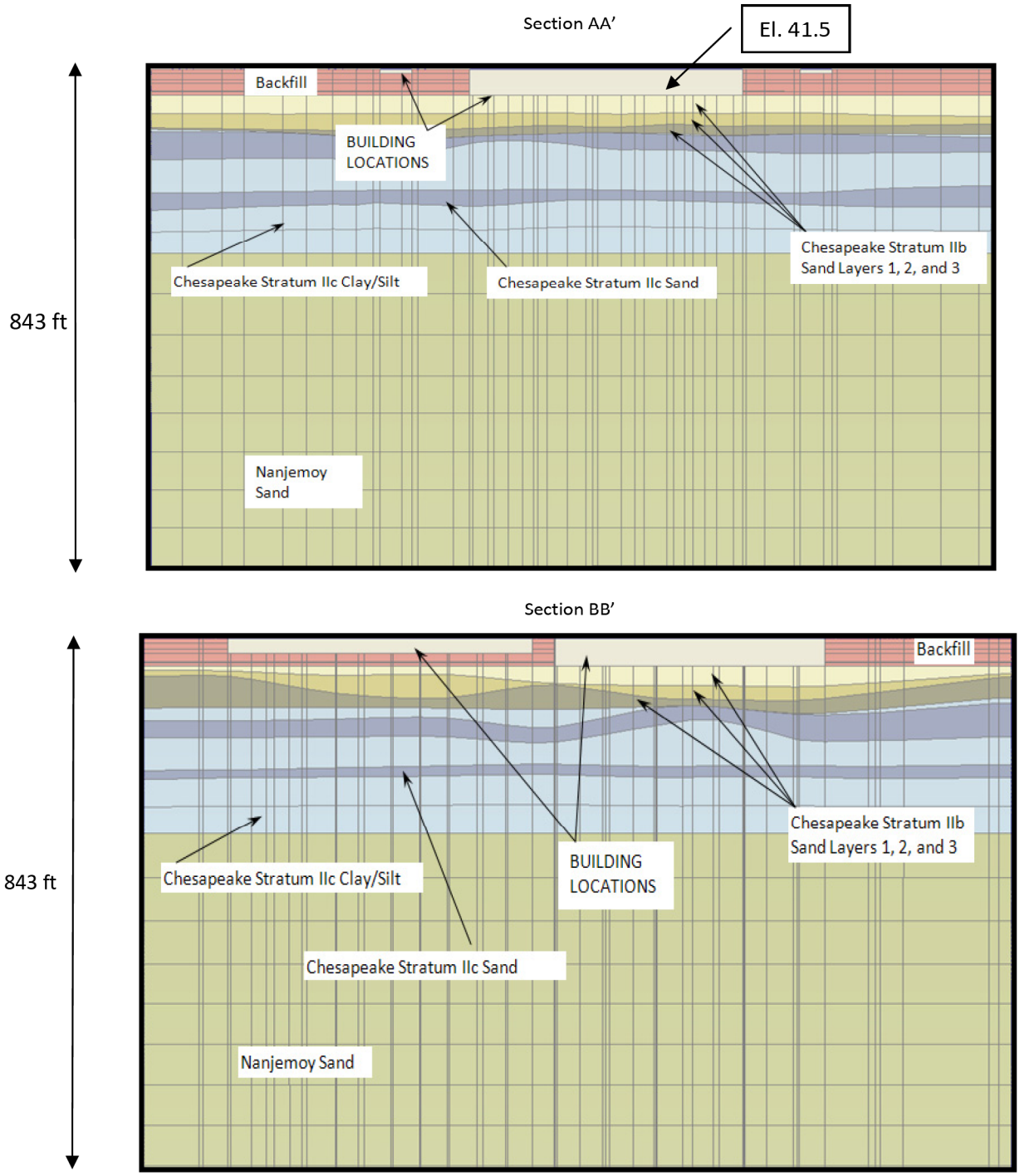

Figure 10- PC model cross sections

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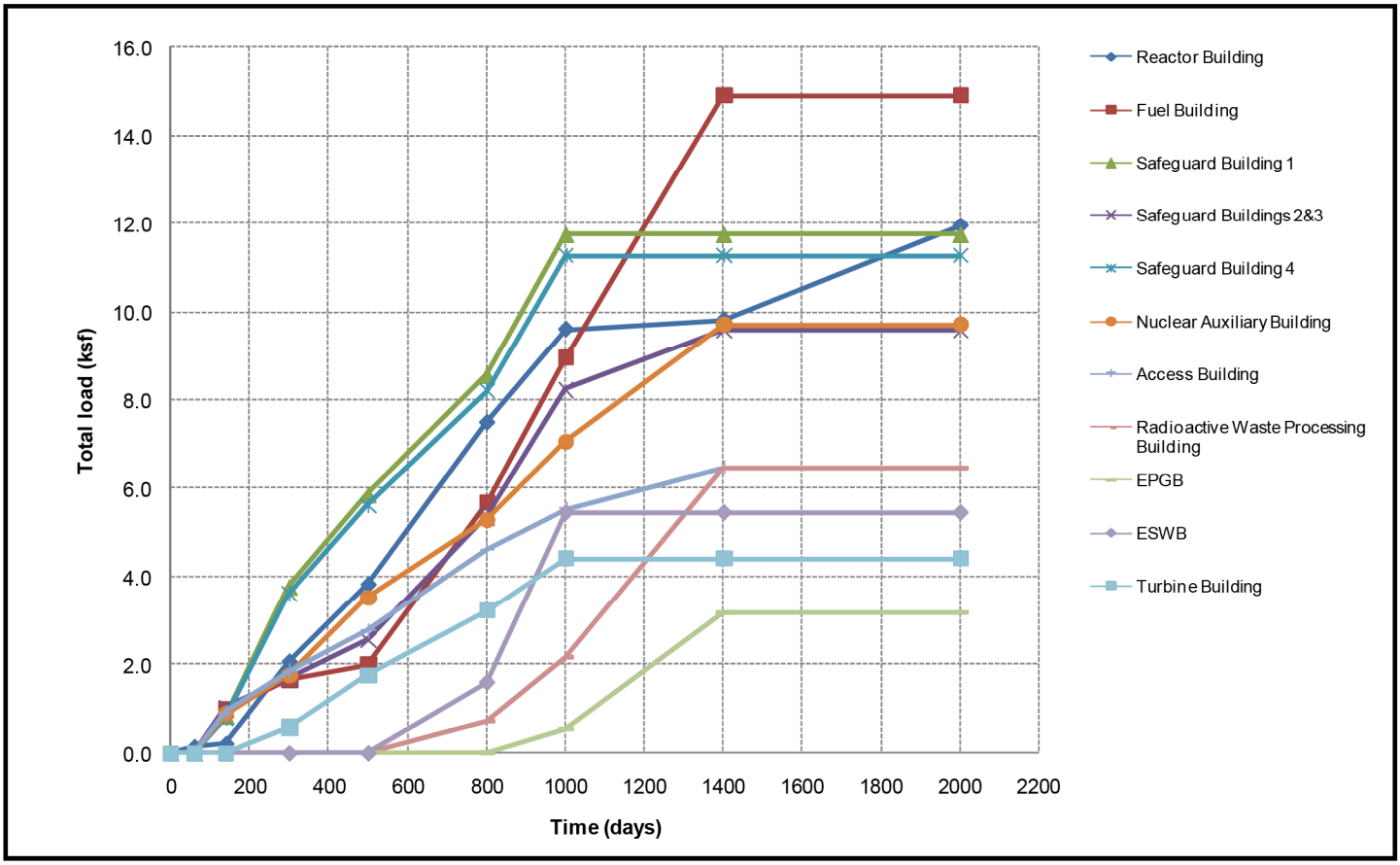

Figure 11 - Loading sequence (PC model)

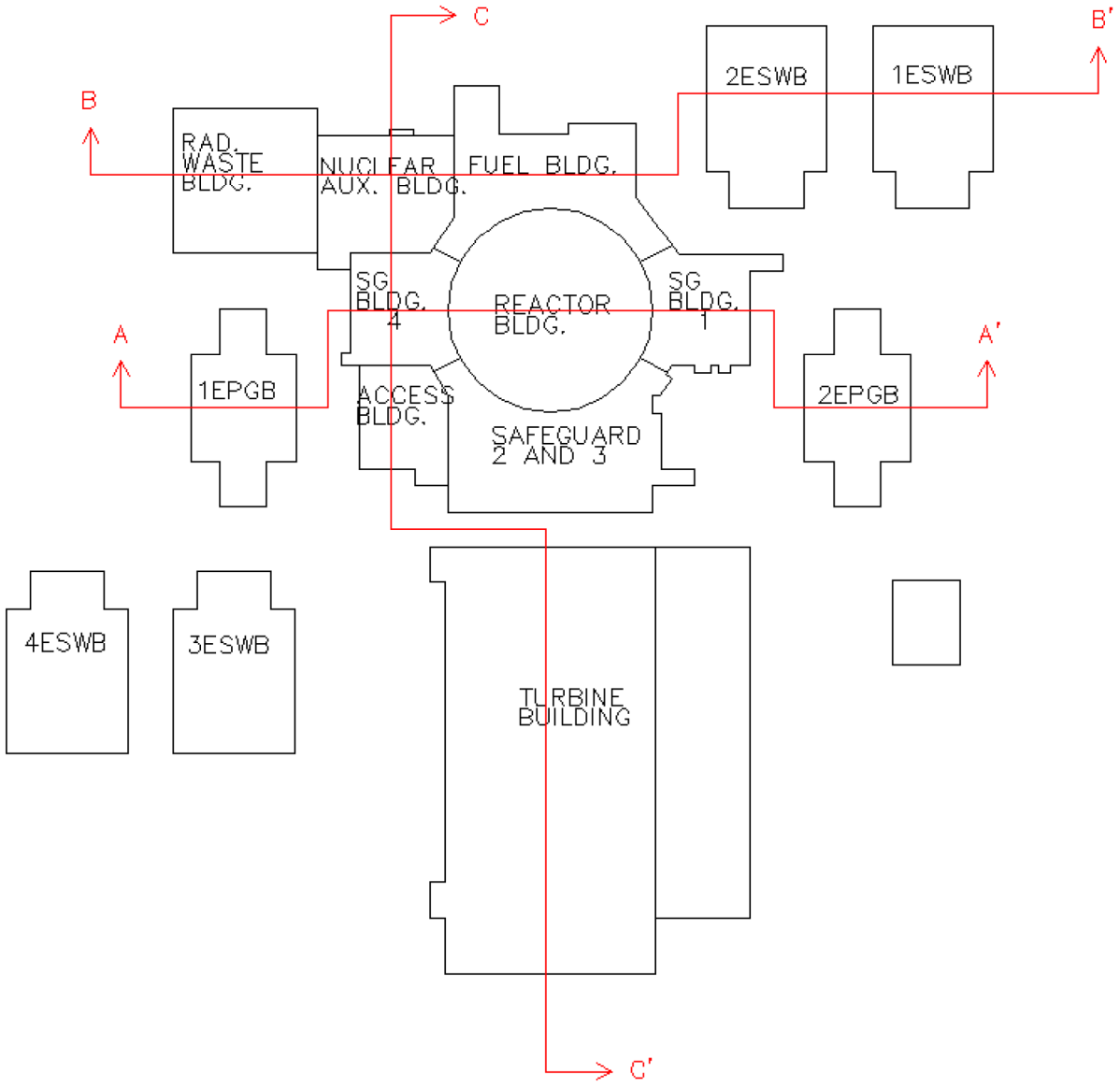

Figure 12 – Schematic representation of construction sequence, definition of sections

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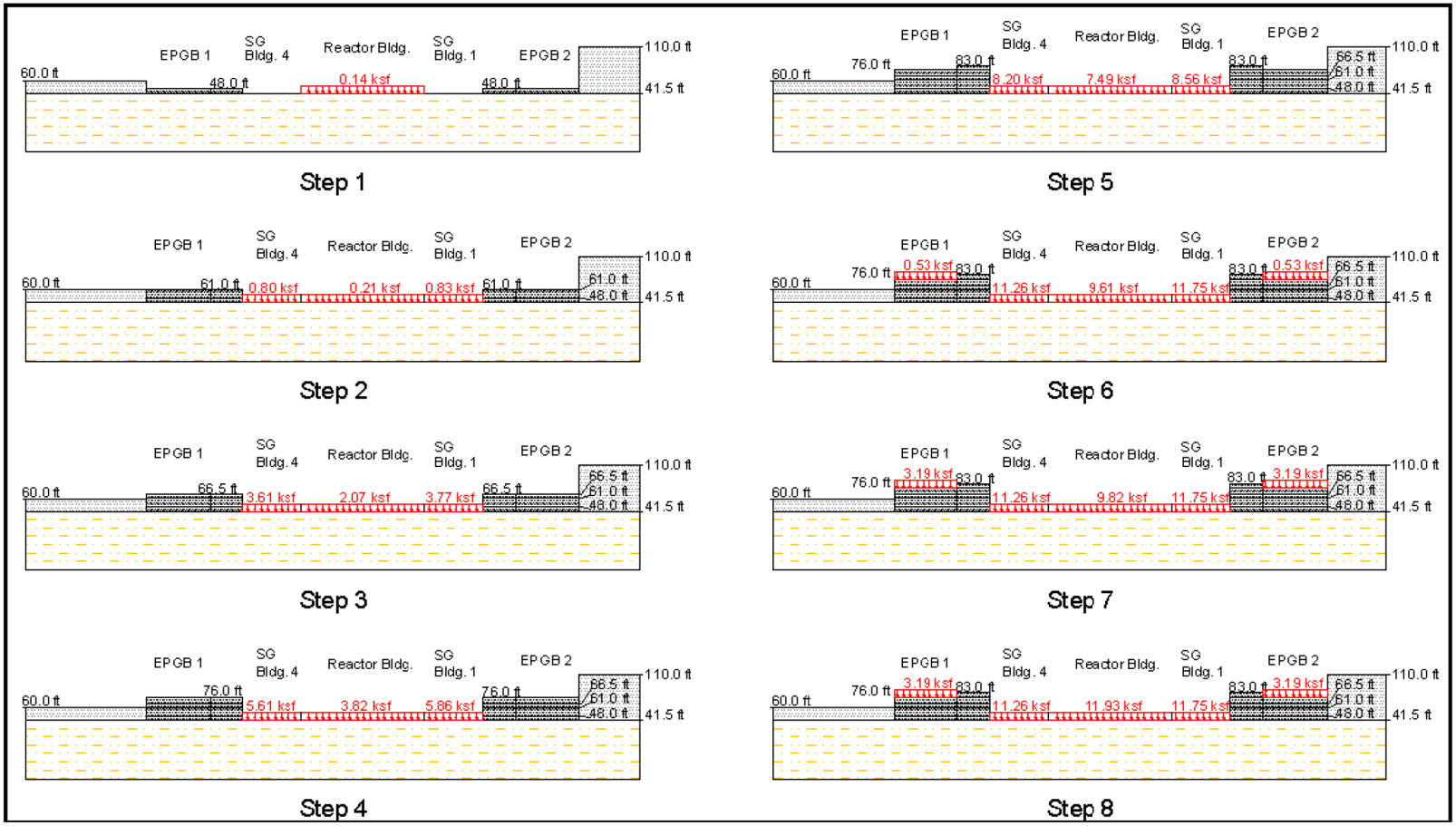

Figure 13 – Schematic representation of construction sequence, Section AA'

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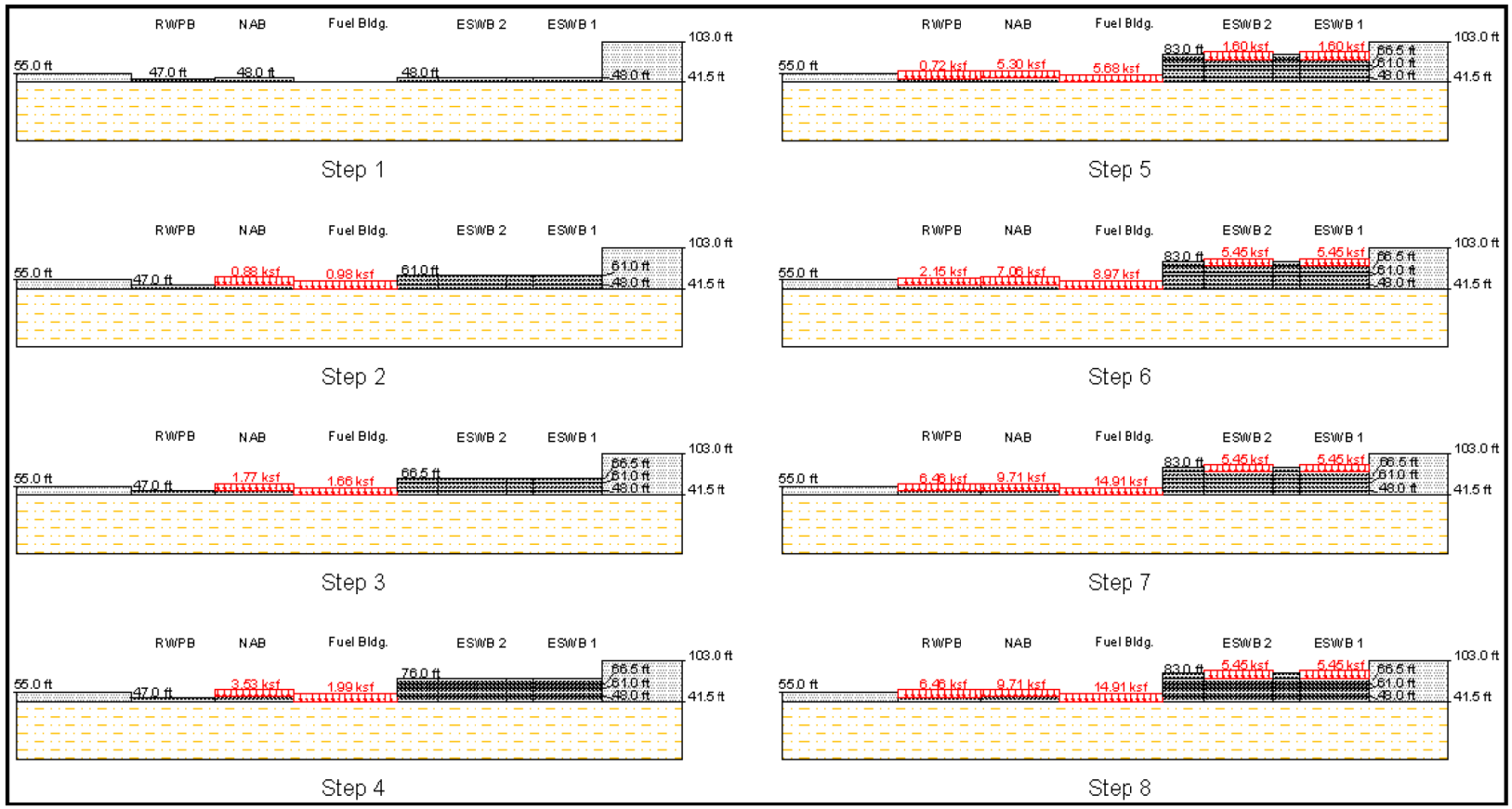

Figure 14 – Schematic representation of construction sequence, Section BB'

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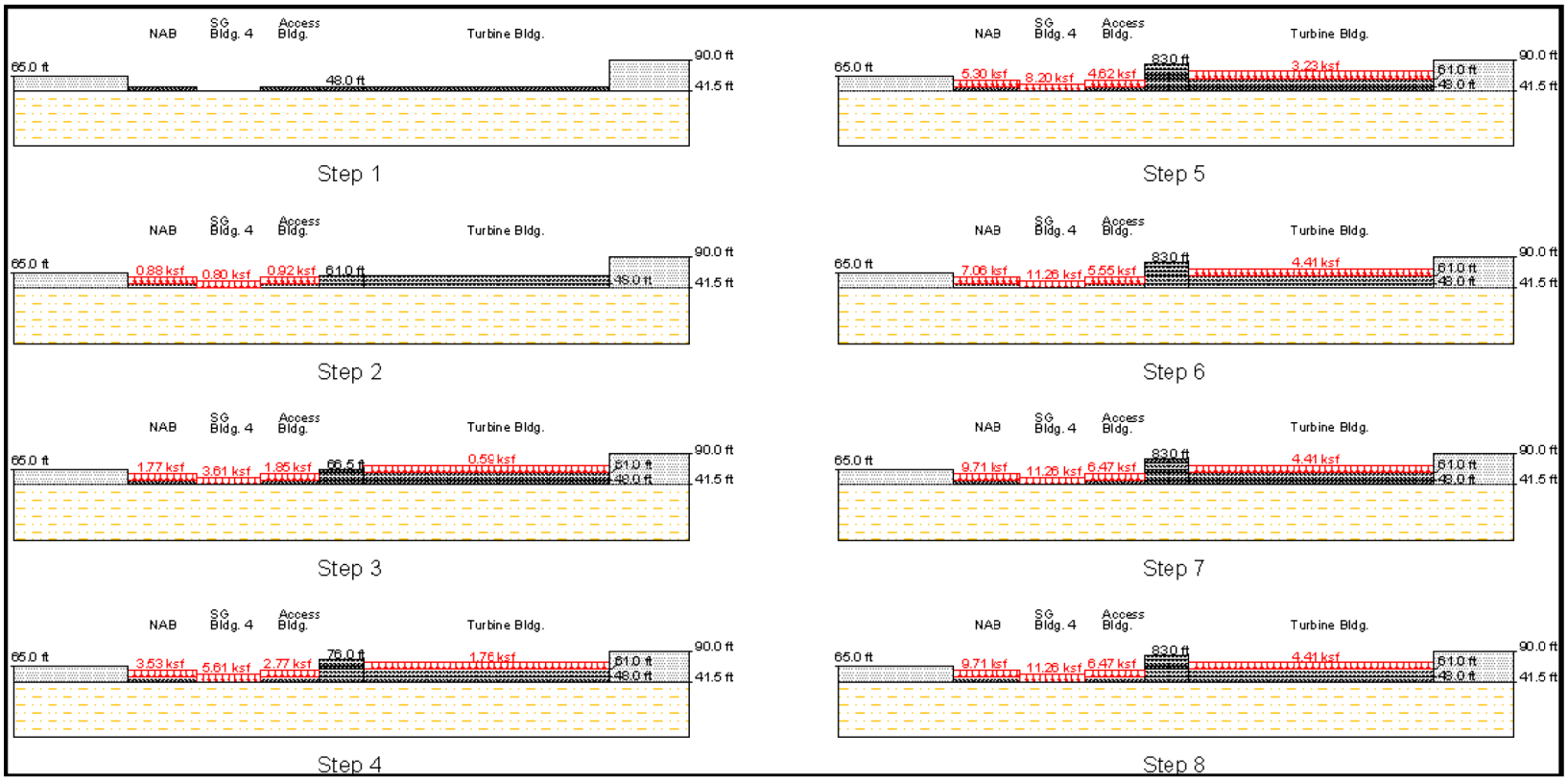

Figure 15 – Schematic representation of construction sequence, Section CC'

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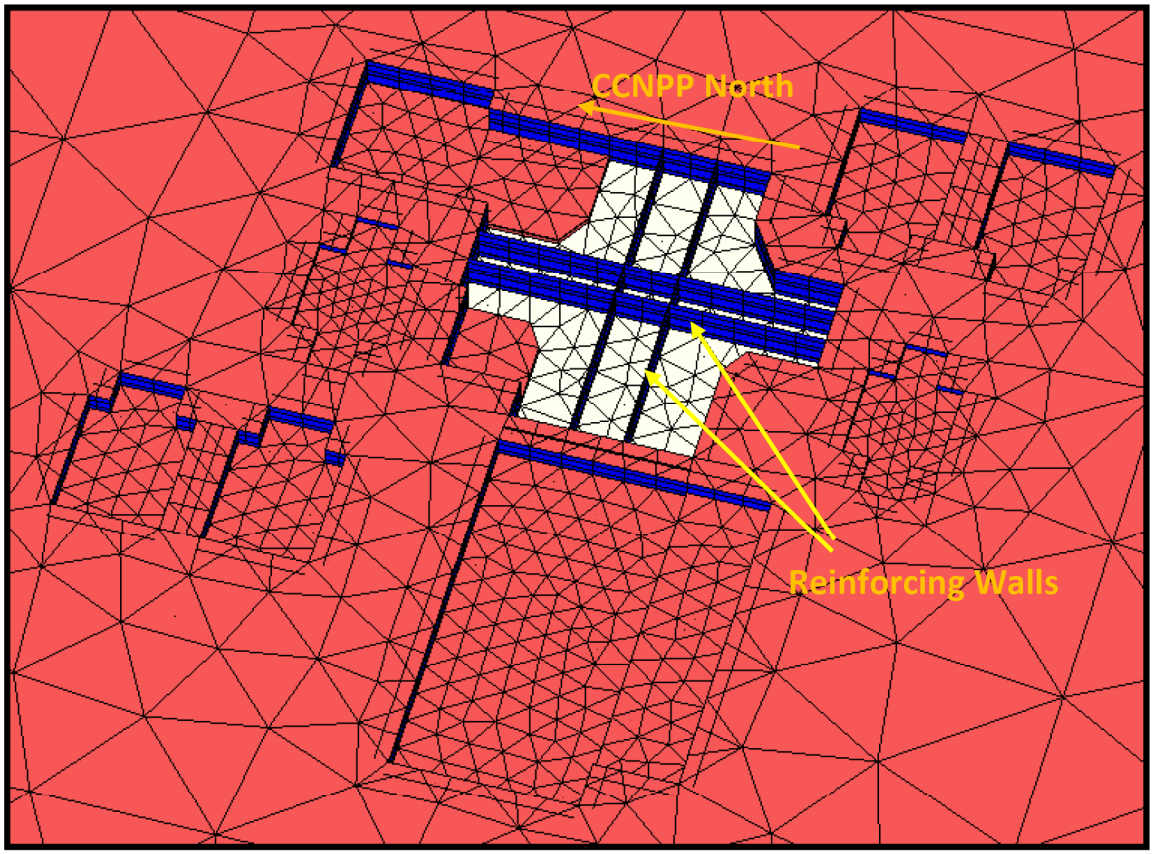

Figure 16 – Reinforcing walls in PC model.

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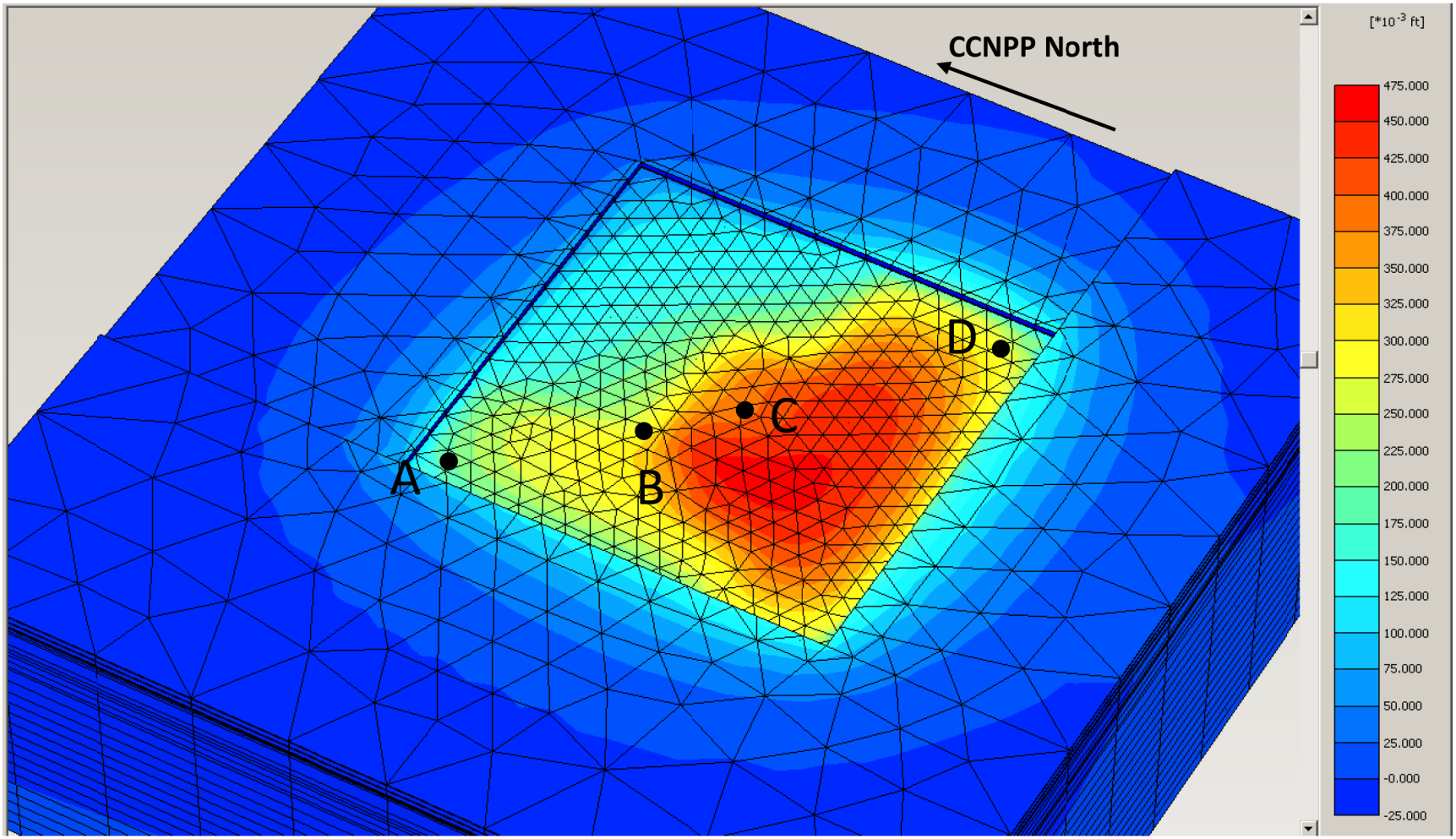

Figure 17 – Immediate heave due to excavation (negative sign indicates settlement), ED model.

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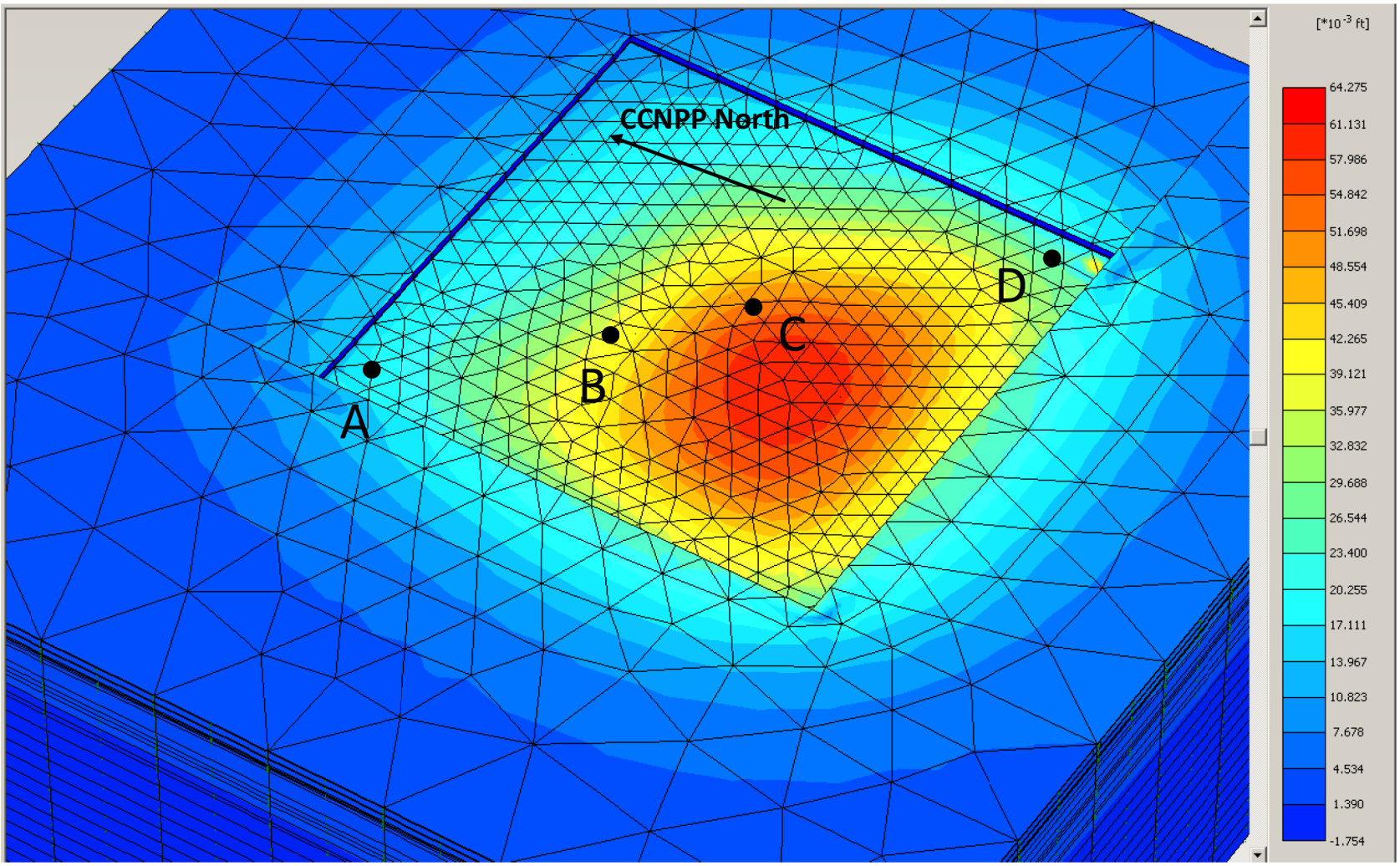

Figure 18- Heave due to de-consolidation induced by excavation, ED model

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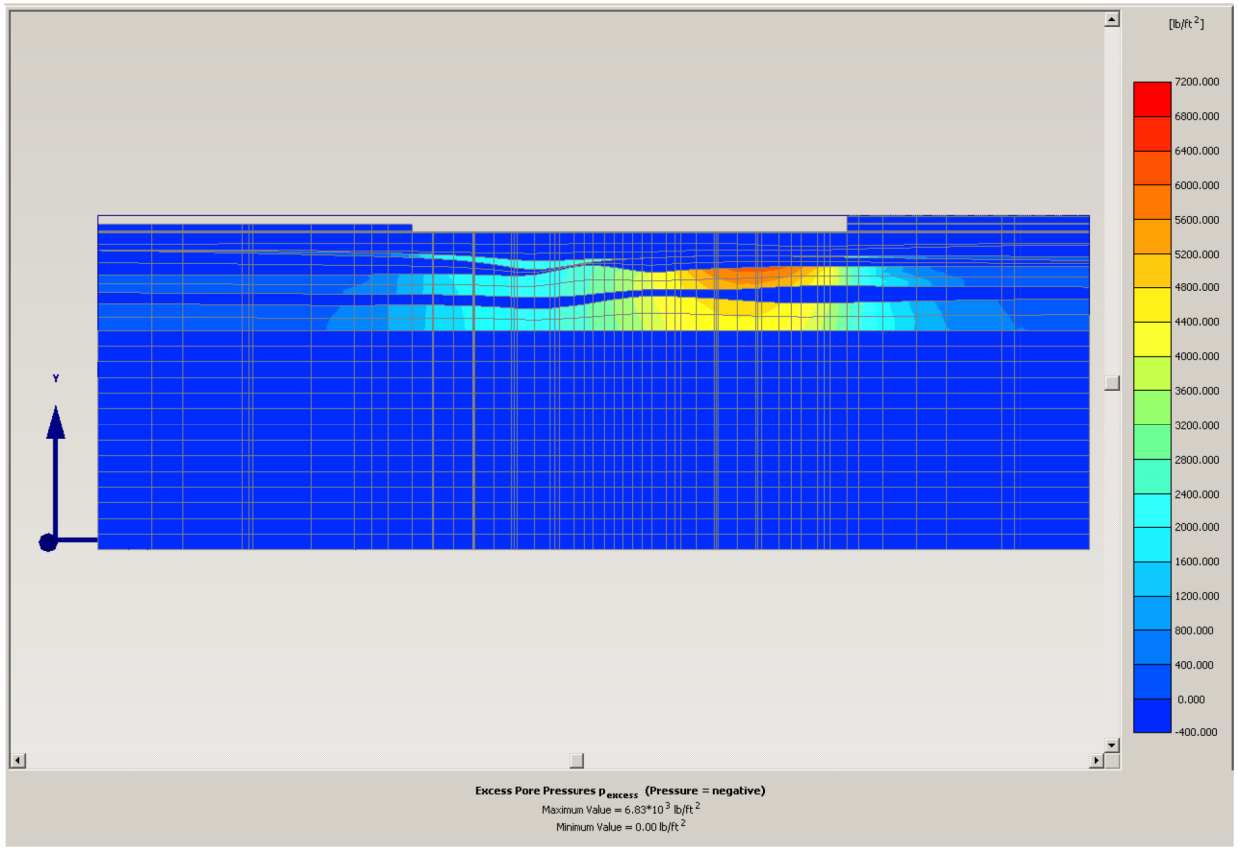

Figure 19 - Excess pore pressure generated on the cross-section A-A of Figure 5 due to excavation, ED model

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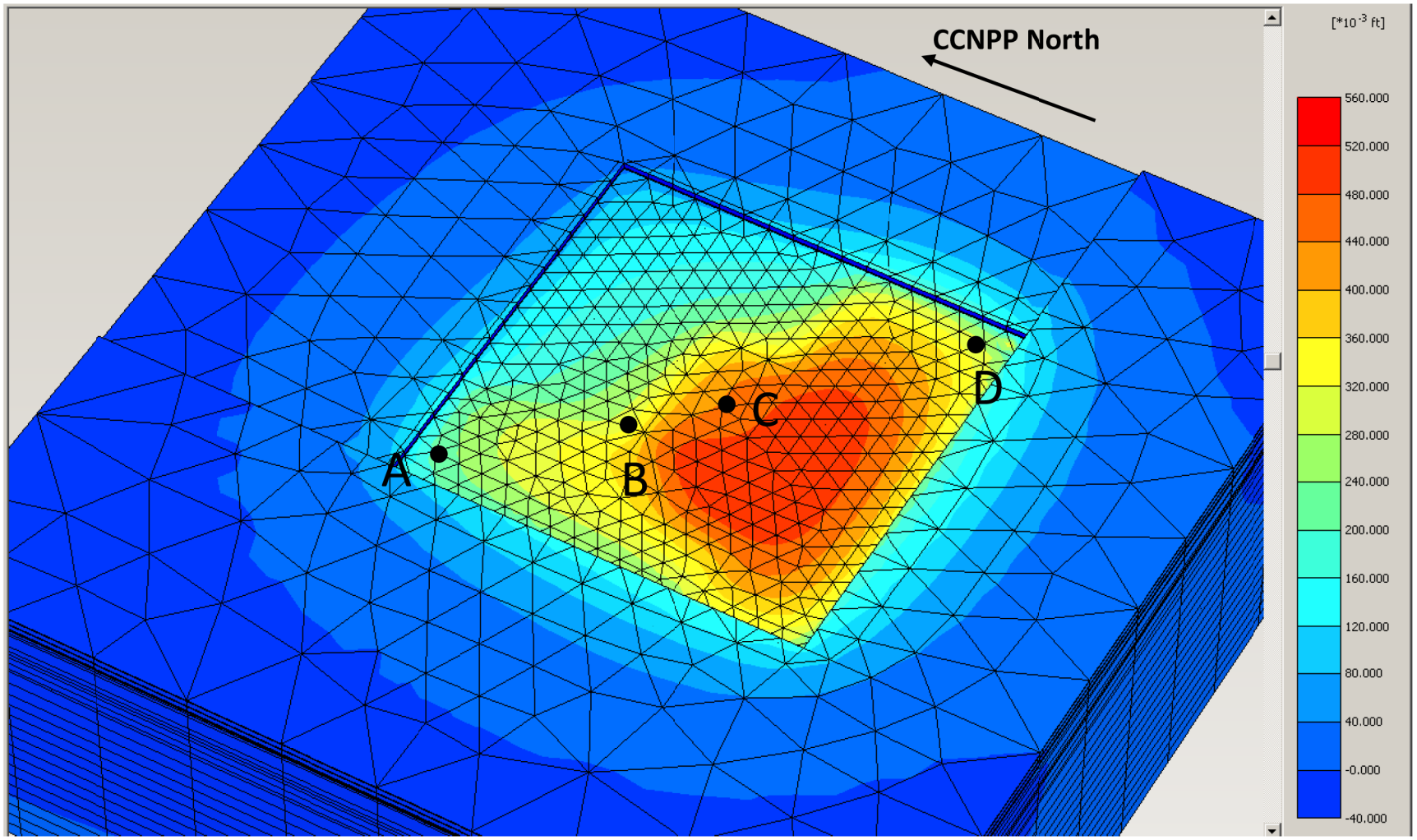

Figure 20- Ultimate heave due to excavation in long term, ED model

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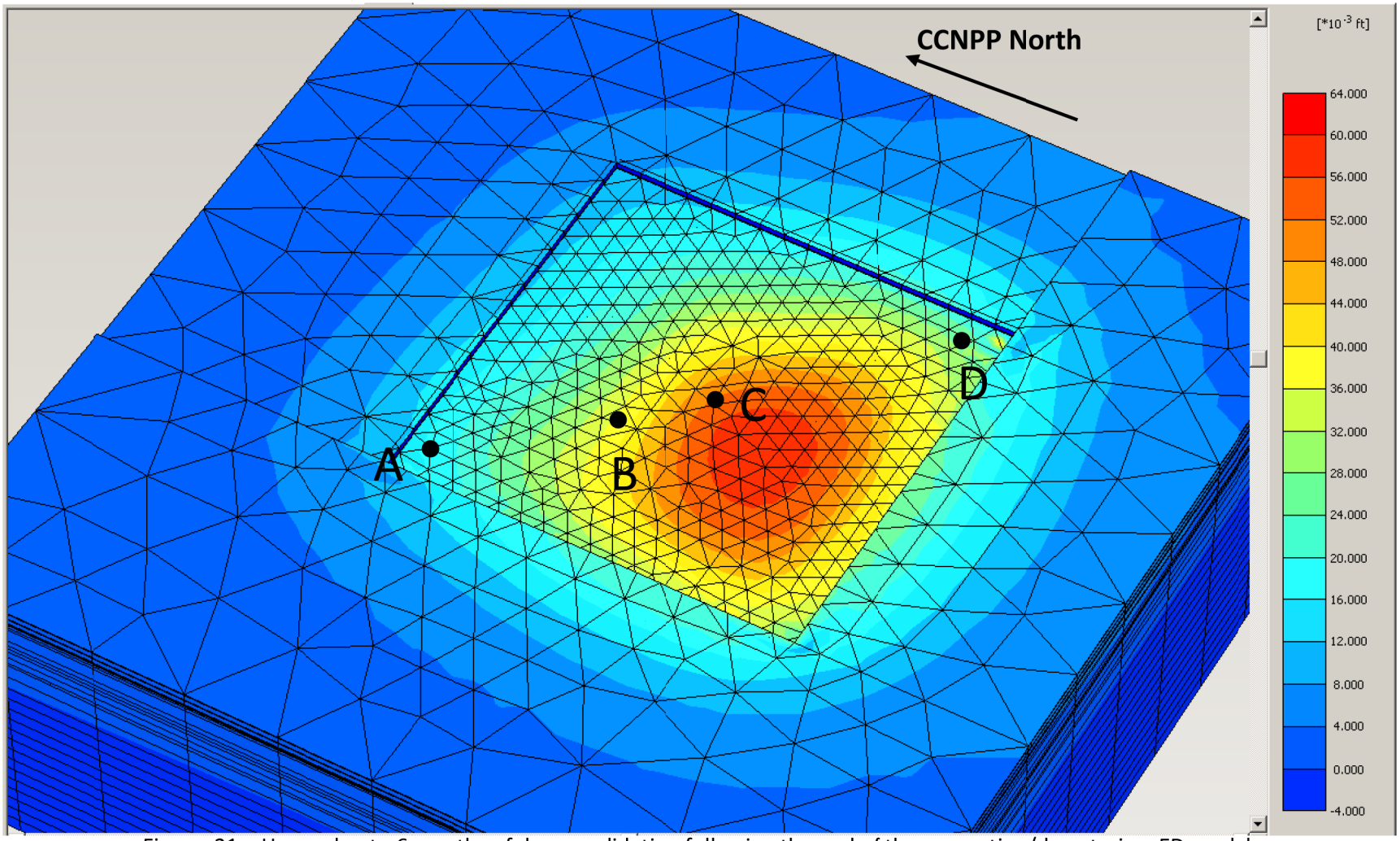

Figure 21 – Heave due to 6 months of de-consolidation following the end of the excavation/dewatering, ED model

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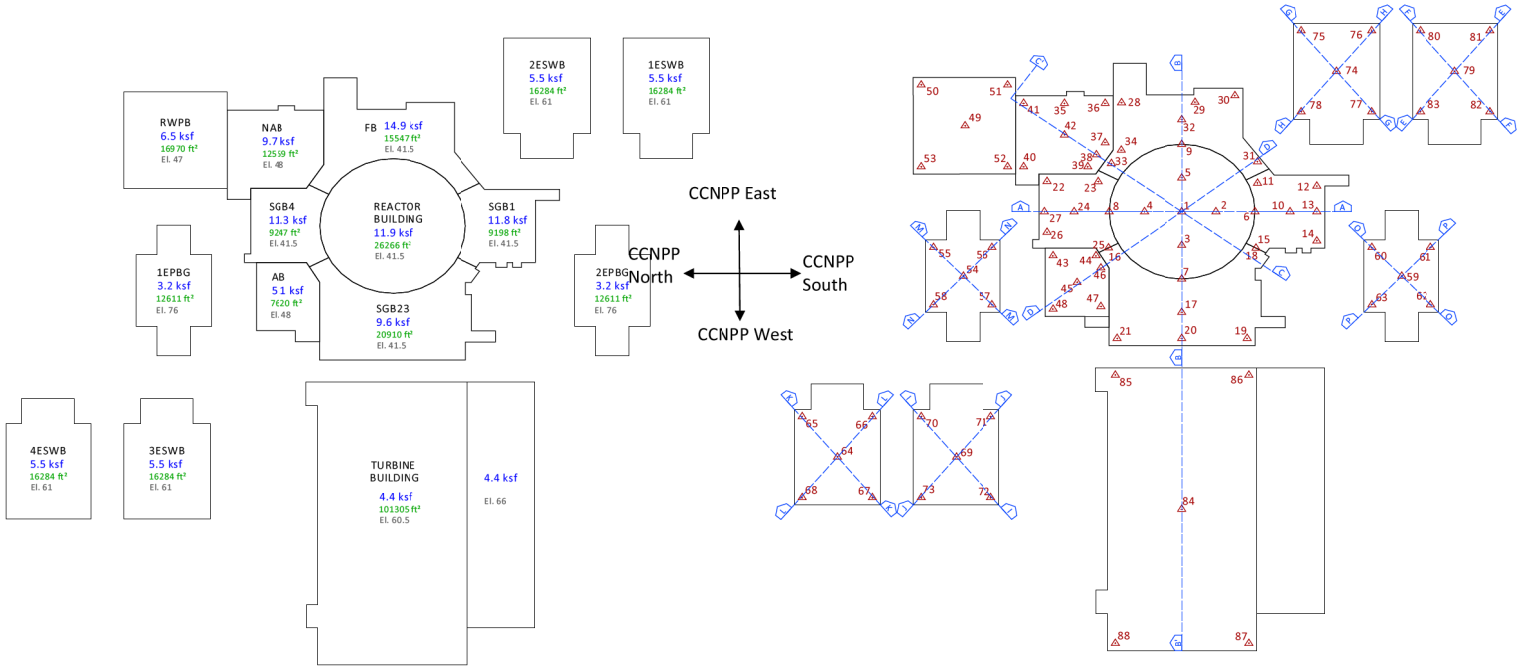

Figure 22 –Building layout and ultimate loads and sections analyzed for settlement and/or tilt

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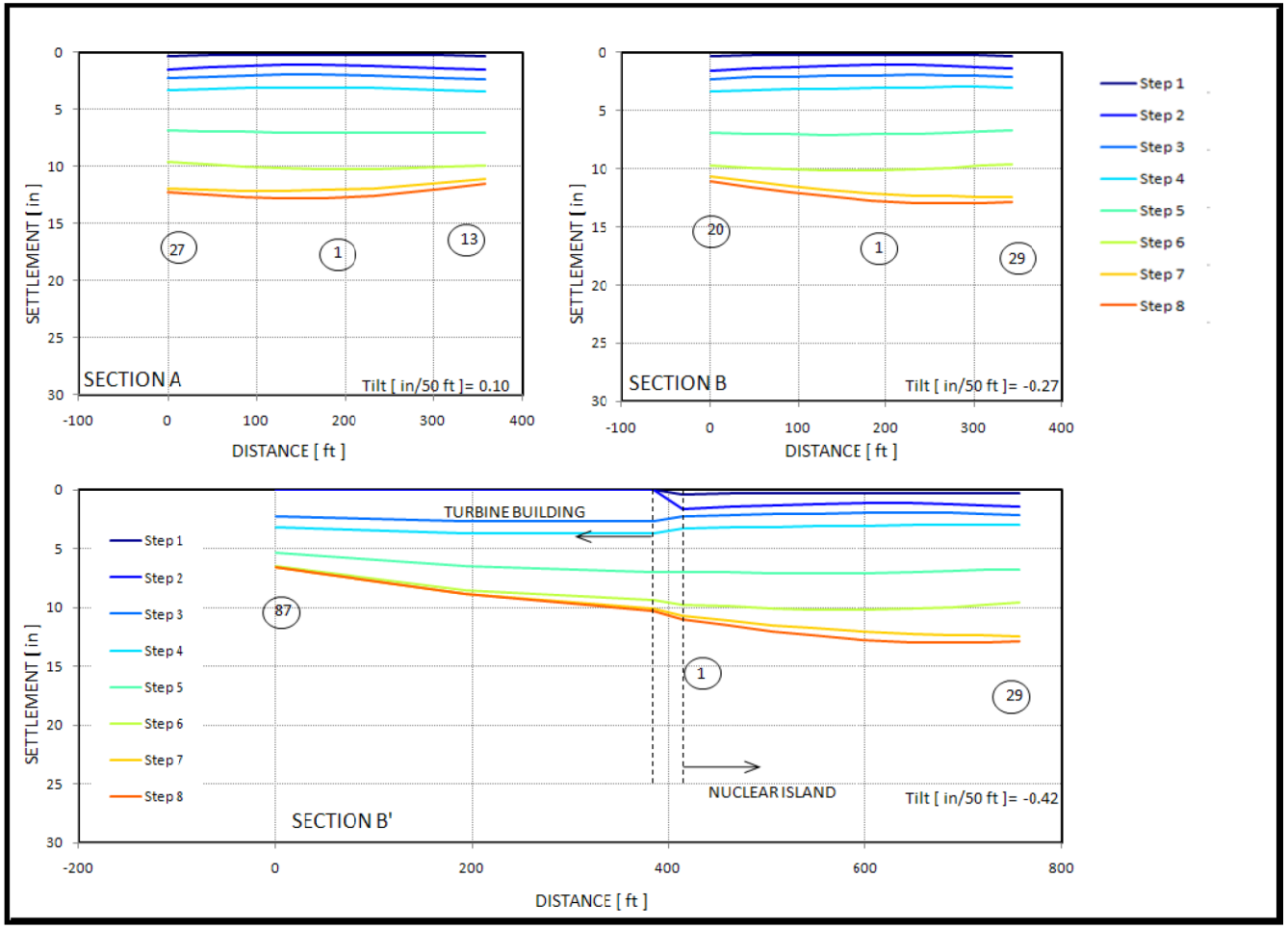

Figure 23 – Settlement of nuclear island common mat and turbine building, MT2 Model (Sections AA, BB, BB')

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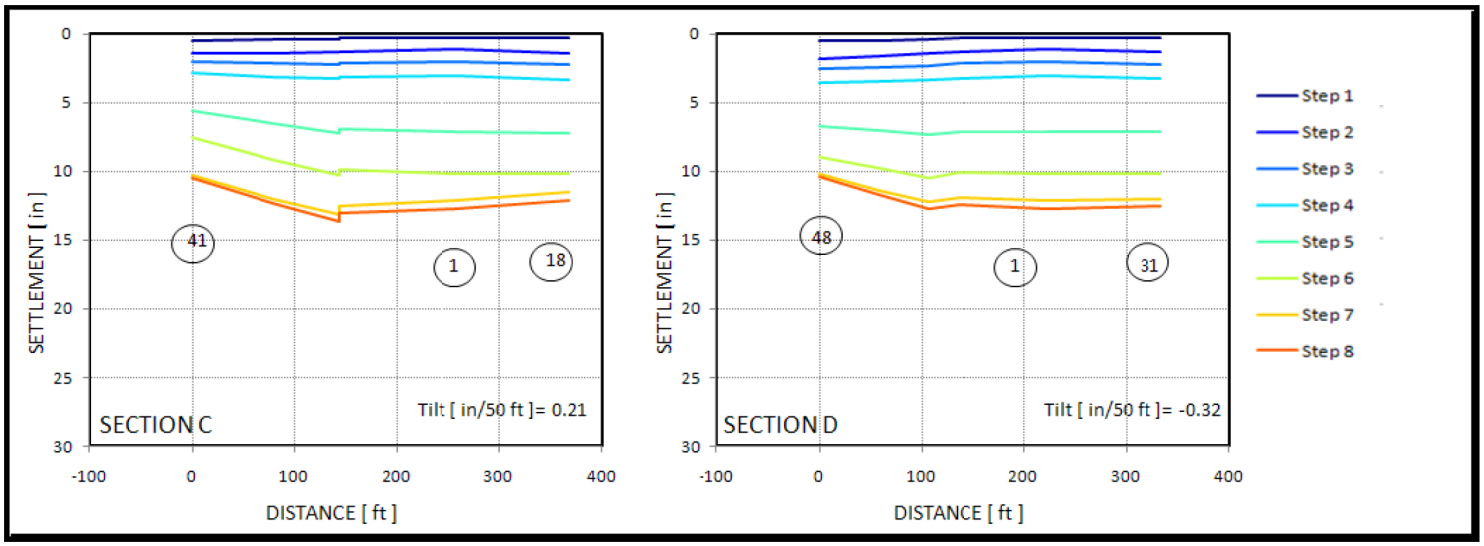

Figure 24 – Settlement on the diagonal axes of the nuclear island common mat, MT2 Model (Sections CC and DD)

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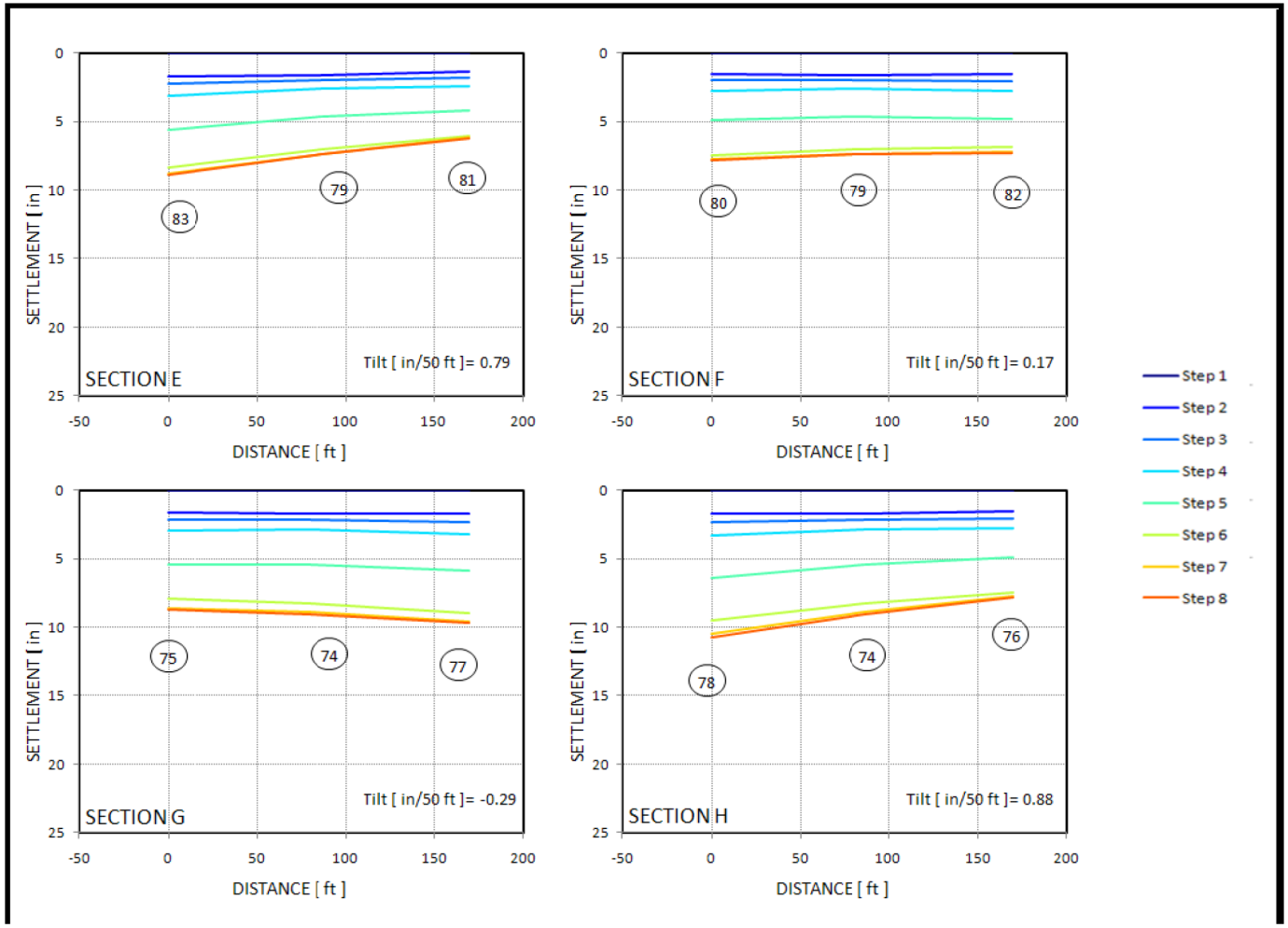

Figure 25 – Settlement on the diagonal axes of 1ESWB (Sections EE and FF) and 2ESWB (Sections GG and HH) MT2 Model

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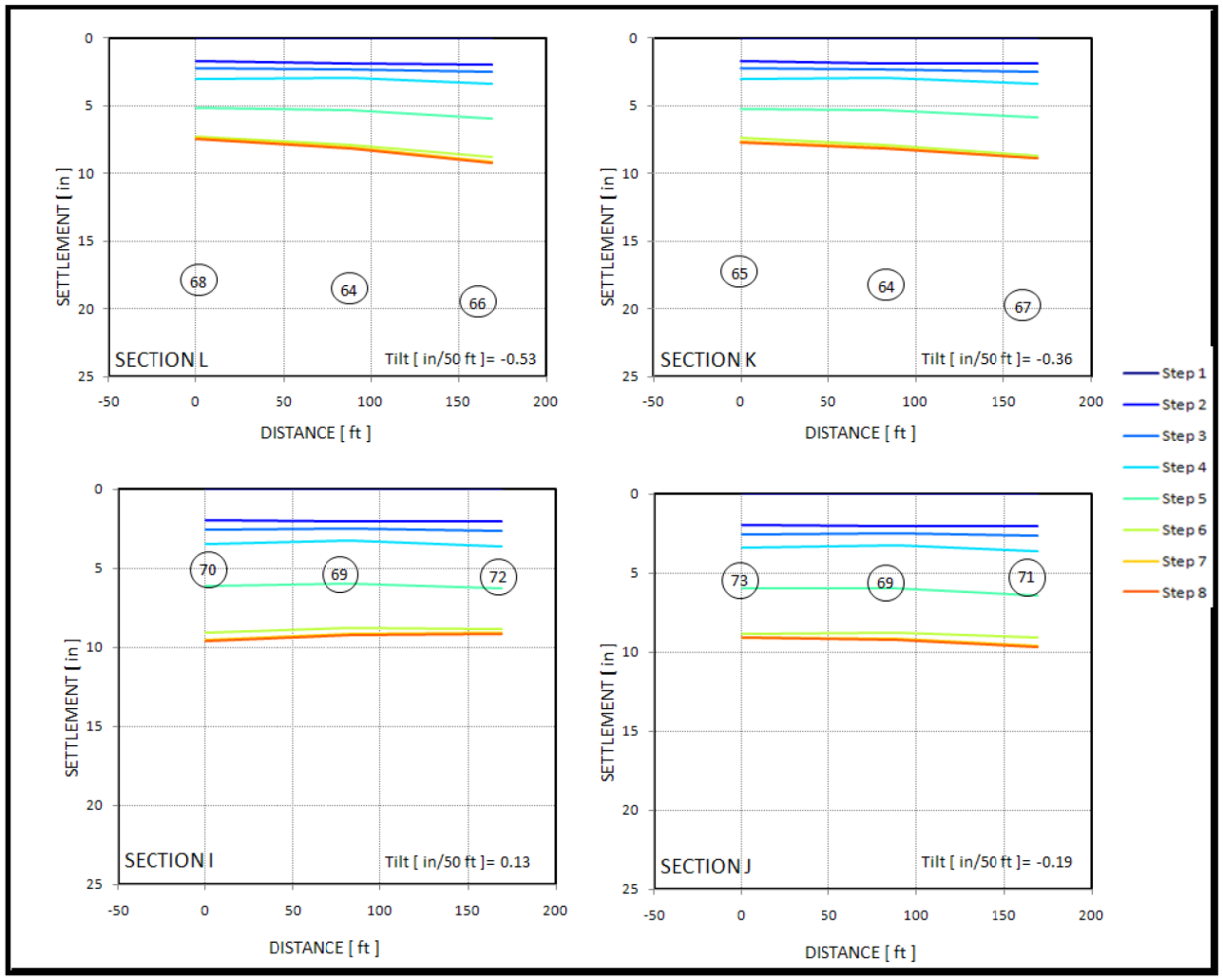

Figure 26 – Settlement on diagonal axes of 3ESWB (Sections II and JJ) and 4ESWB (Sections KK and LL), MT2 Model

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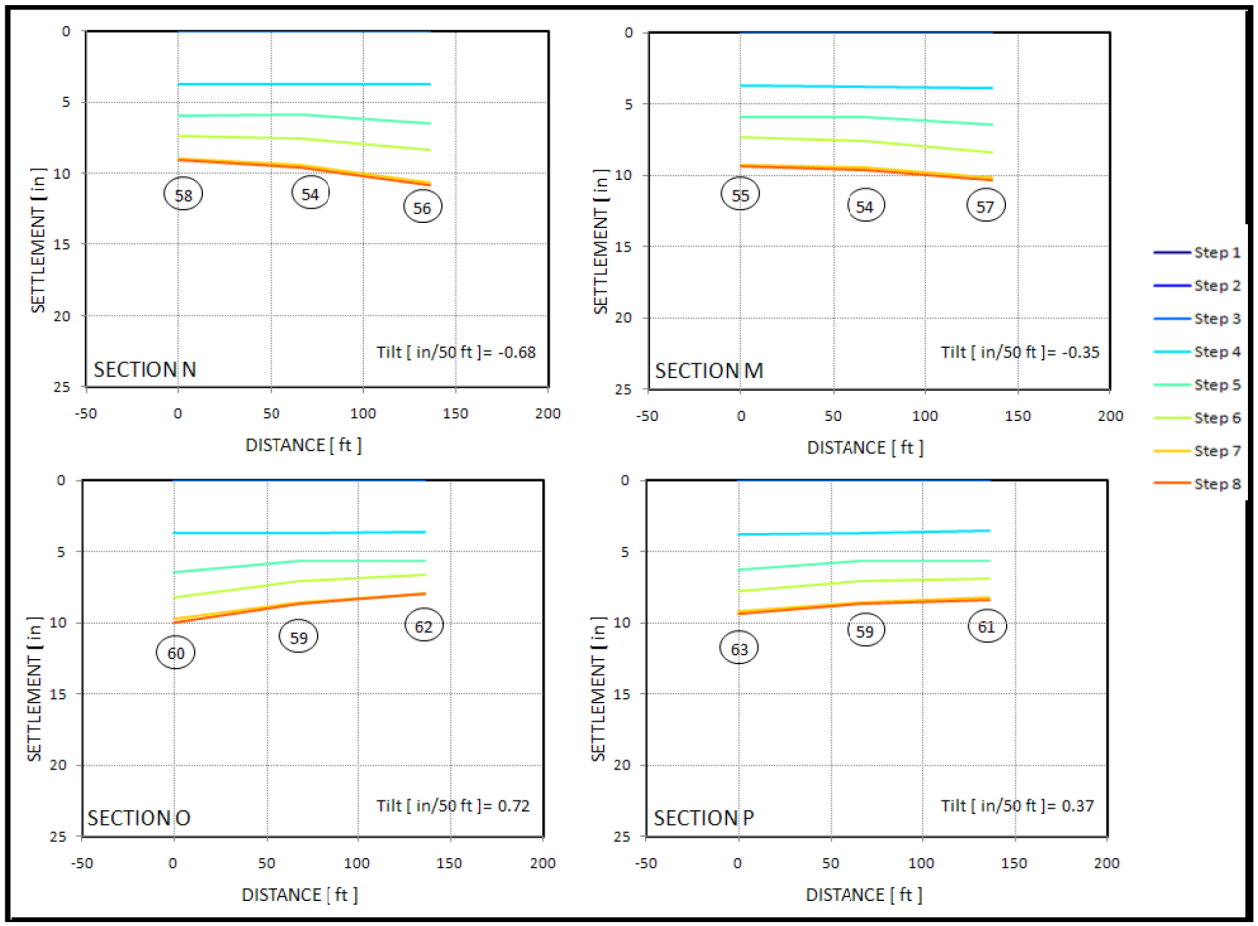

Figure 27 – Settlement on diagonal axes of 1EPBG (Sections MM and NN) and 2EPBG (Sections OO and PP), MT2 Model

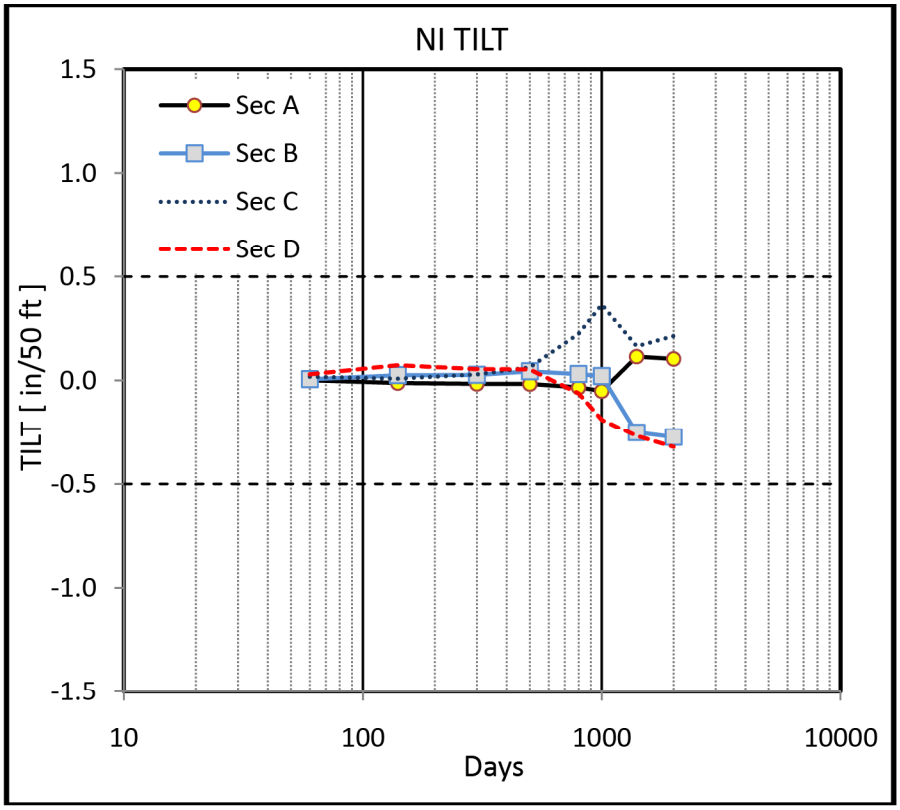

Figure 28 – NI common mat tilt as a function of time, MT2 Model

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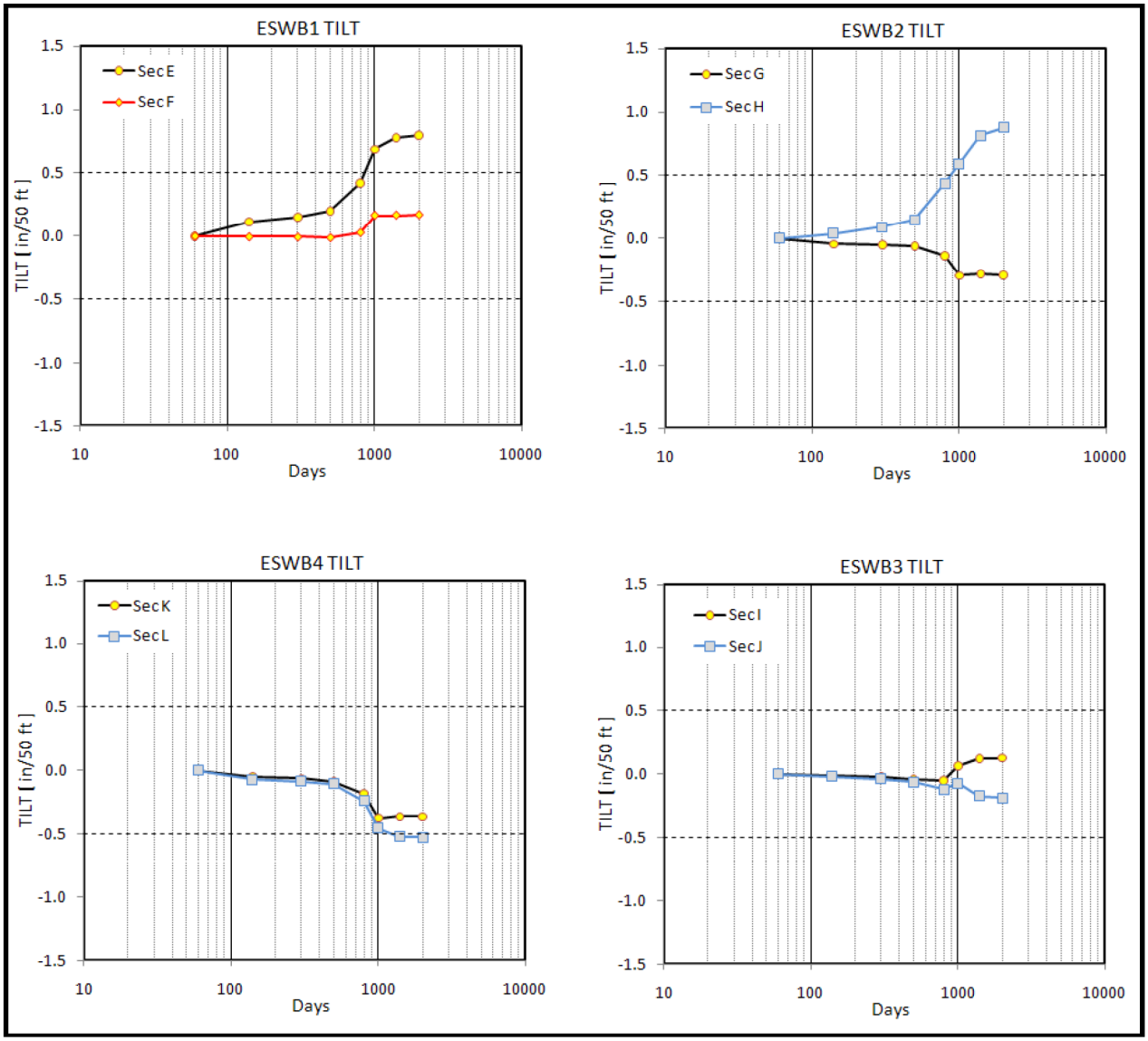

Figure 29 – Tilt for ESWB's as a function of time, MT2 Model

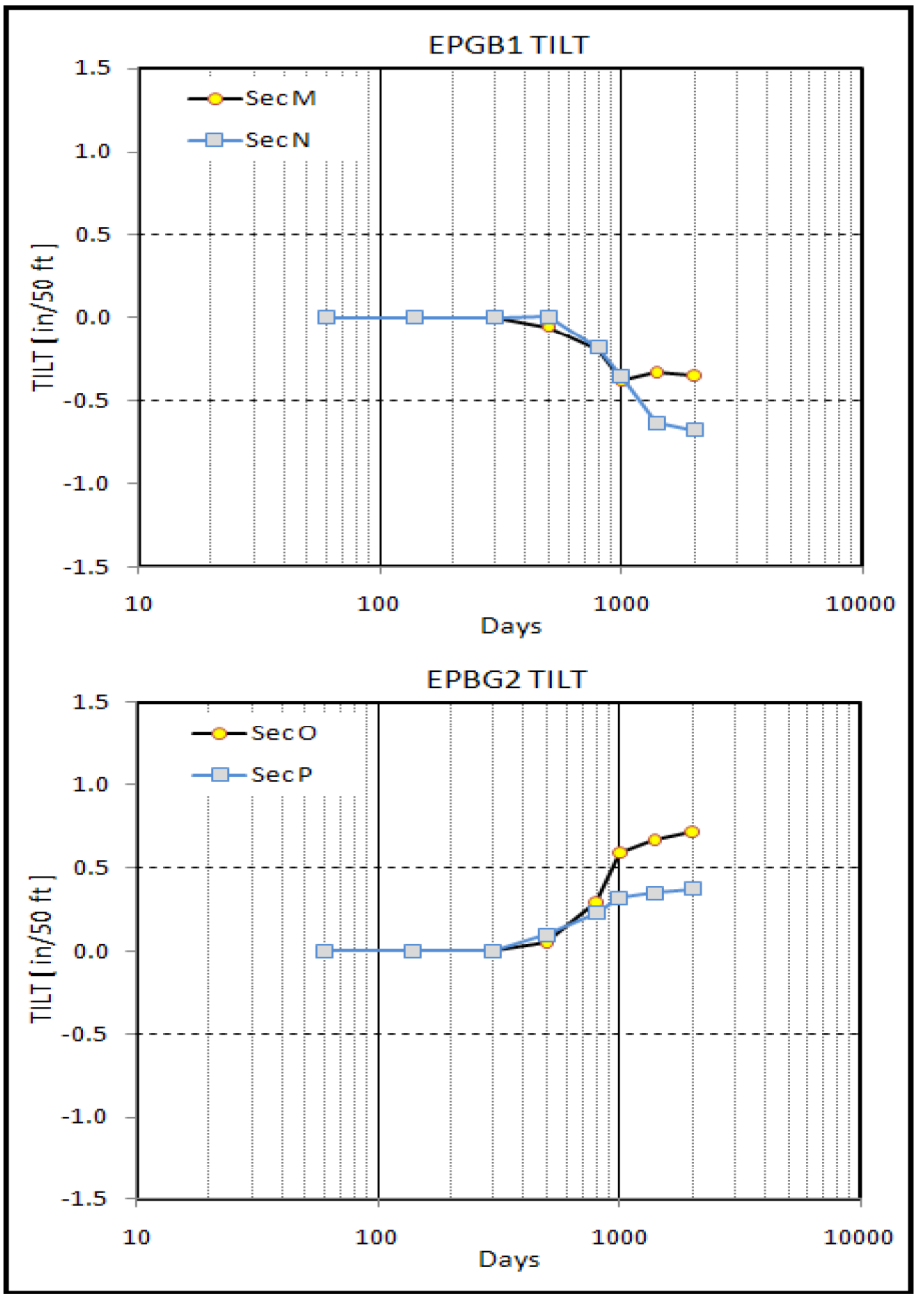

Figure 30 – Tilt for EPBG's as a function of time, MT2 model

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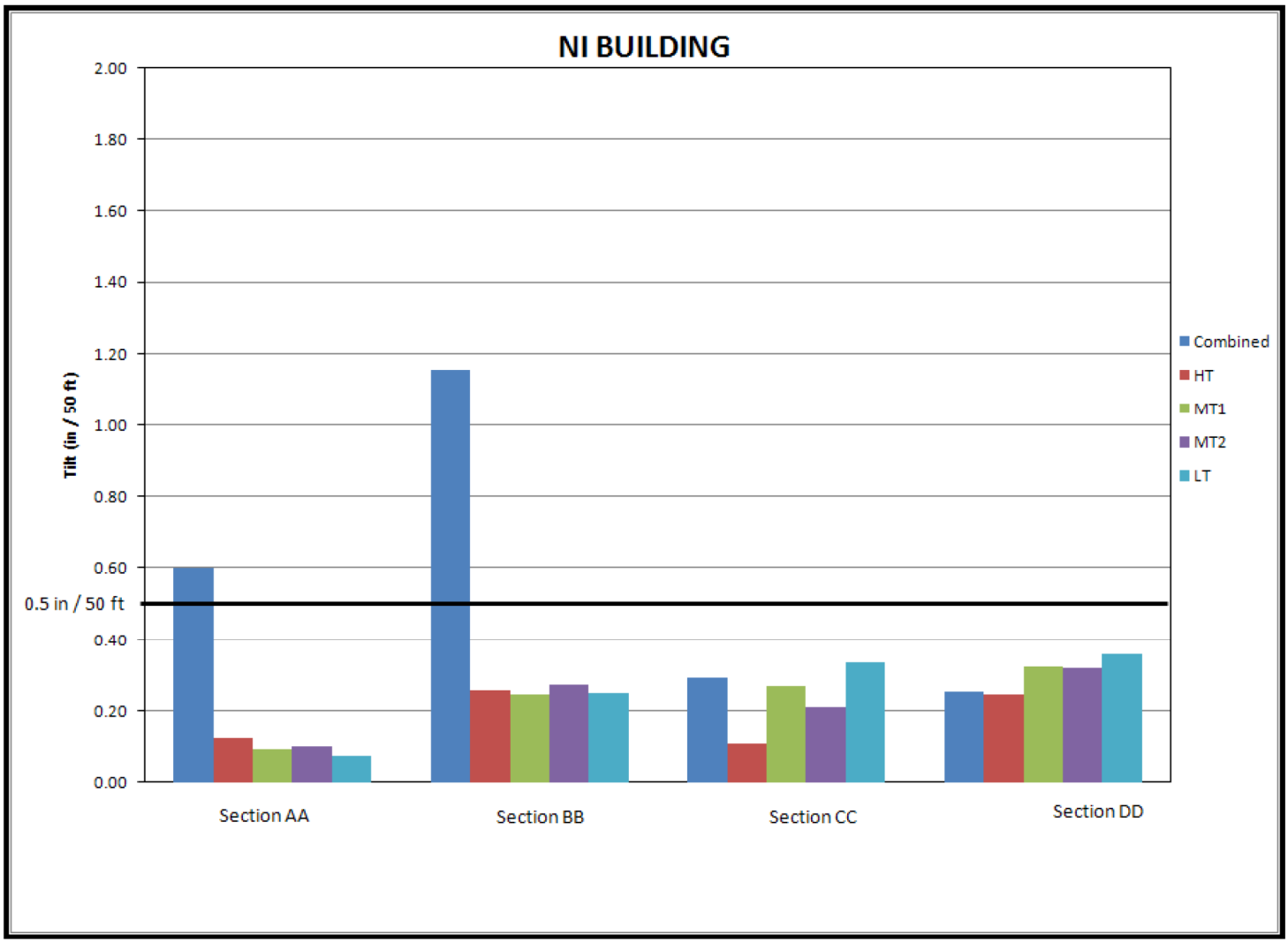

Figure 31 – Tilt at the end of 8th loading step for NI, MT2 model

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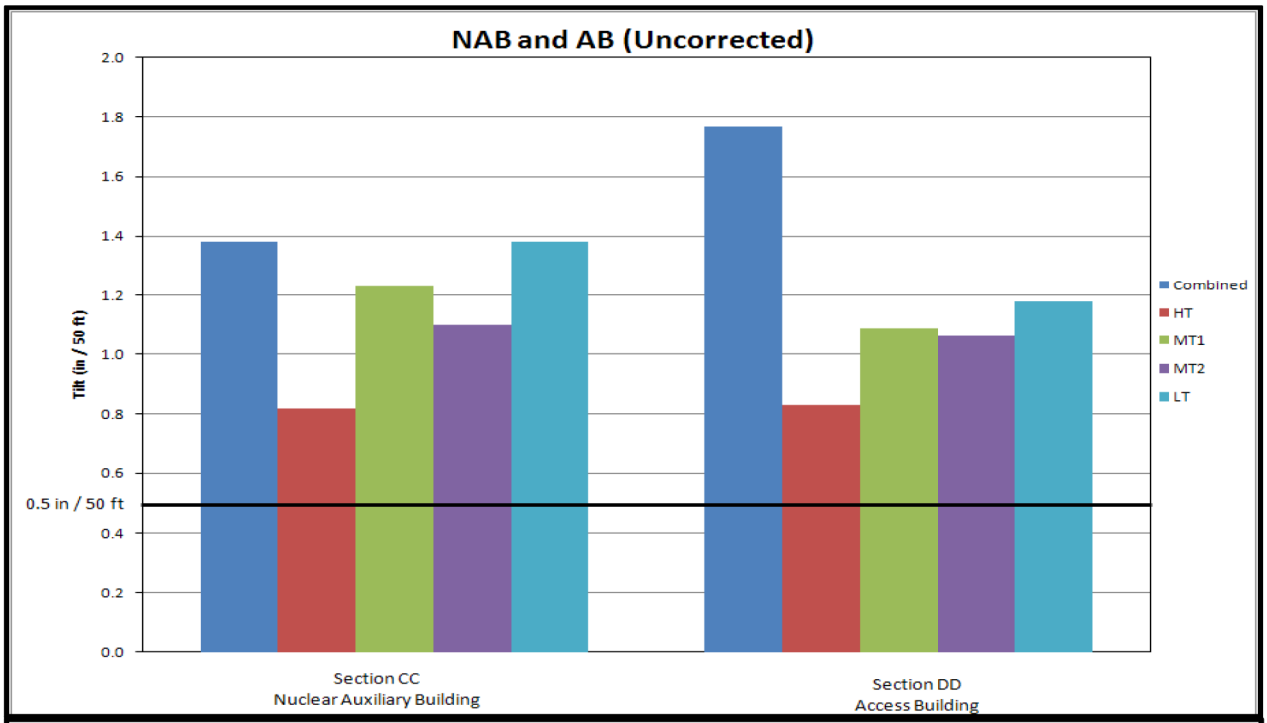

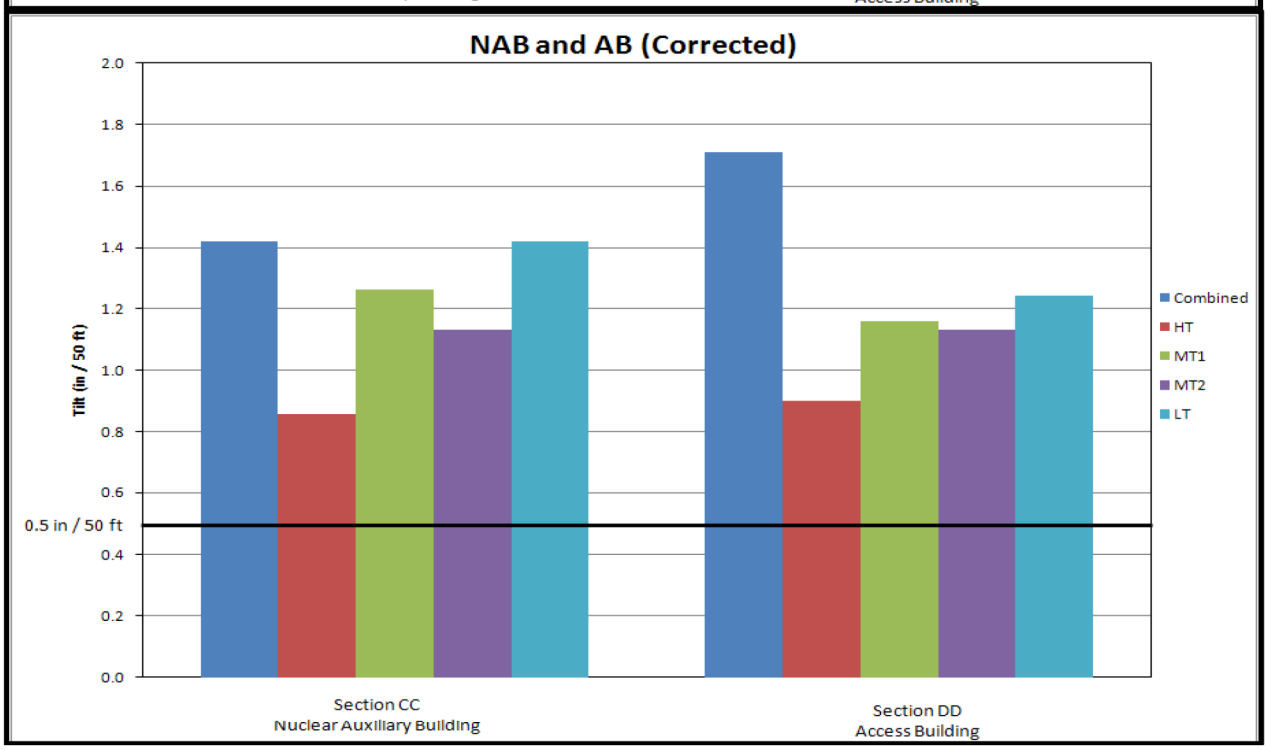

Figure 32 – Tilt at the end of 8th loading step for NAB and AB, MT2 model
(Correction to subtract the observed tilt before the construction of the building)

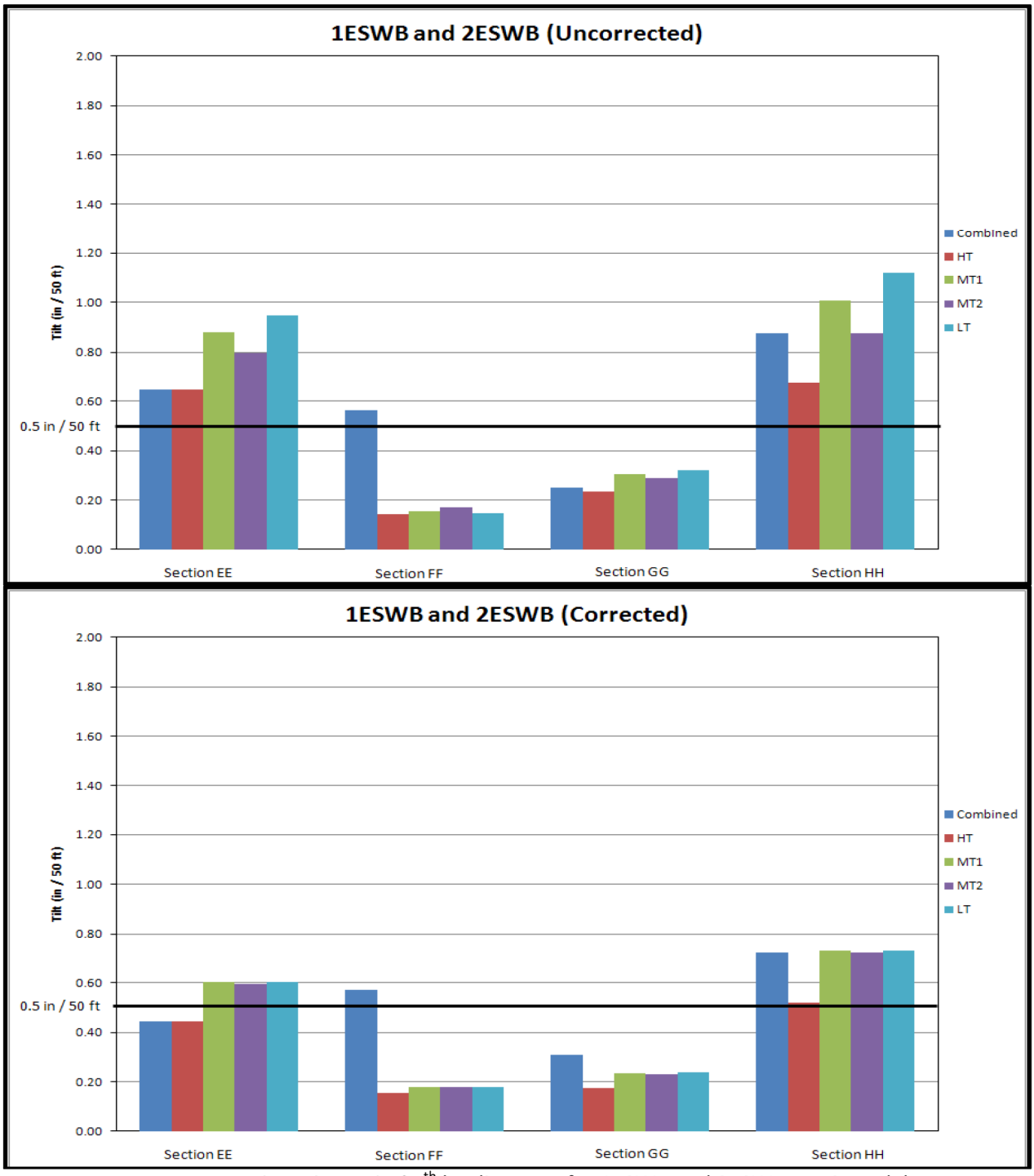

Figure 33 – Tilt at the end of 8th loading step for 1ESWB and 2ESWB, MT2 model
(Correction to subtract the observed tilt before the construction of the building)

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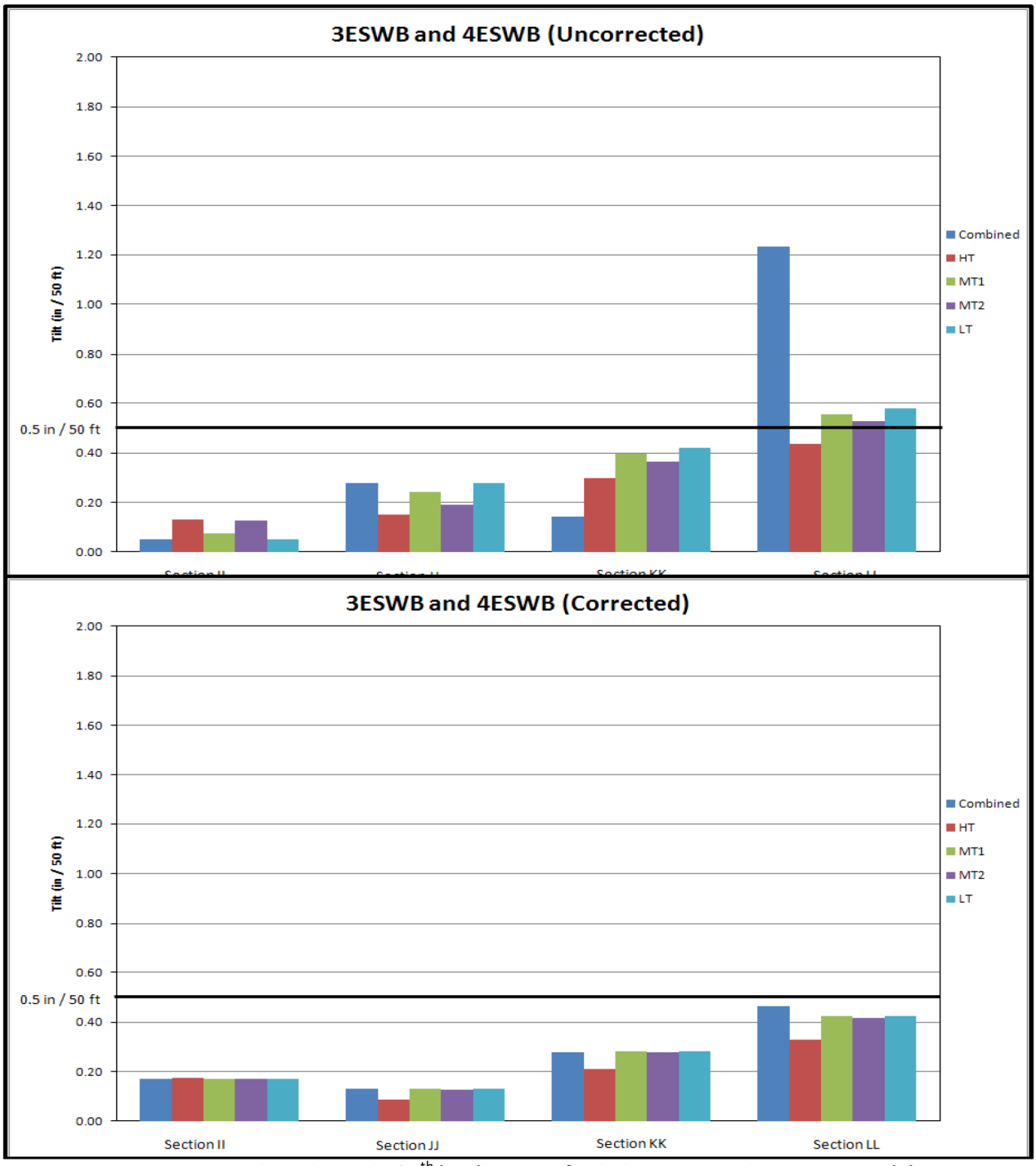

Figure 34 – Tilt at the end of 8th loading step for 3ESWB AND 4ESWB, MT2 model

(Correction to subtract the observed tilt before the construction of the building)

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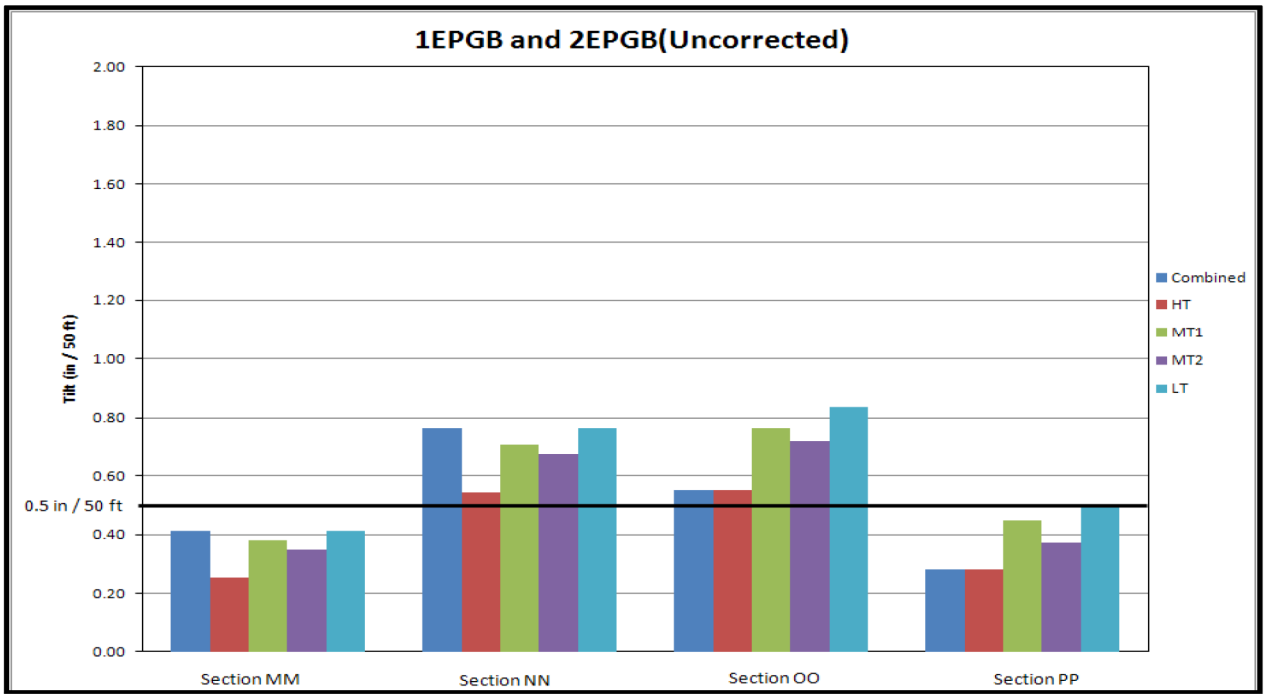

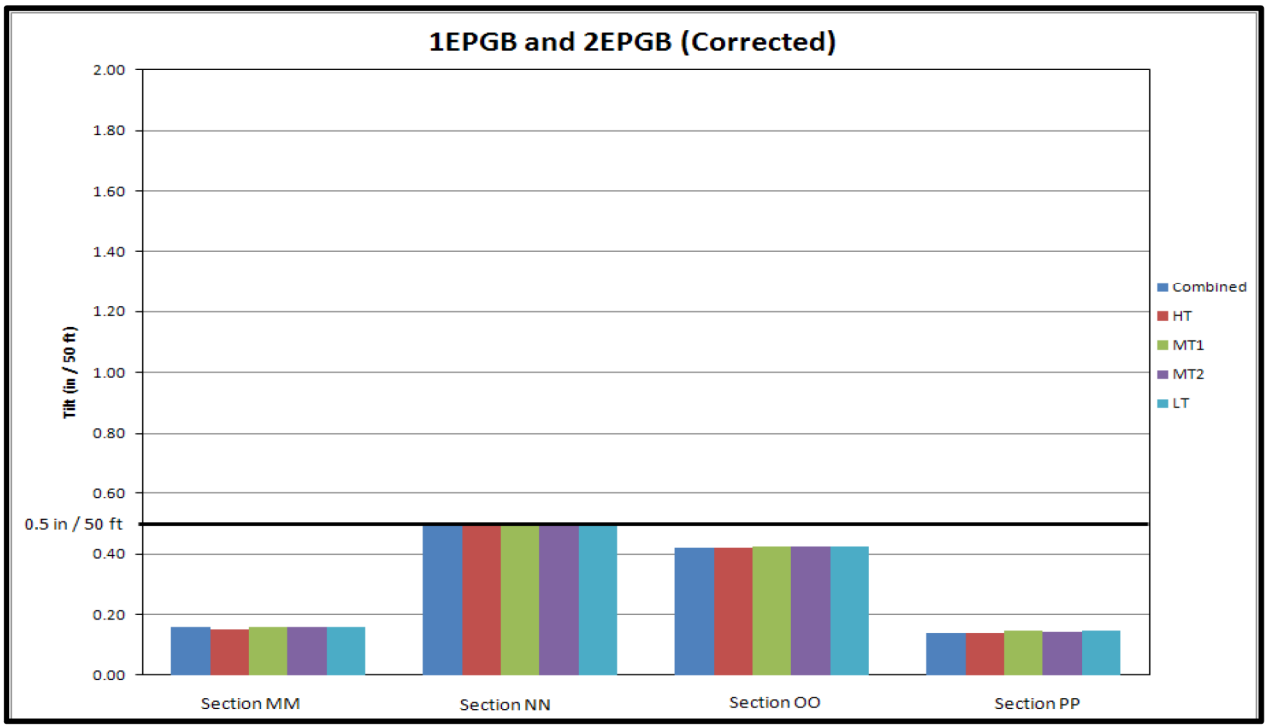

Figure 35 – Tilt at the end of 8th loading step for EPGBs, MT2 model
(Correction to subtract the observed tilt before the construction of the building)

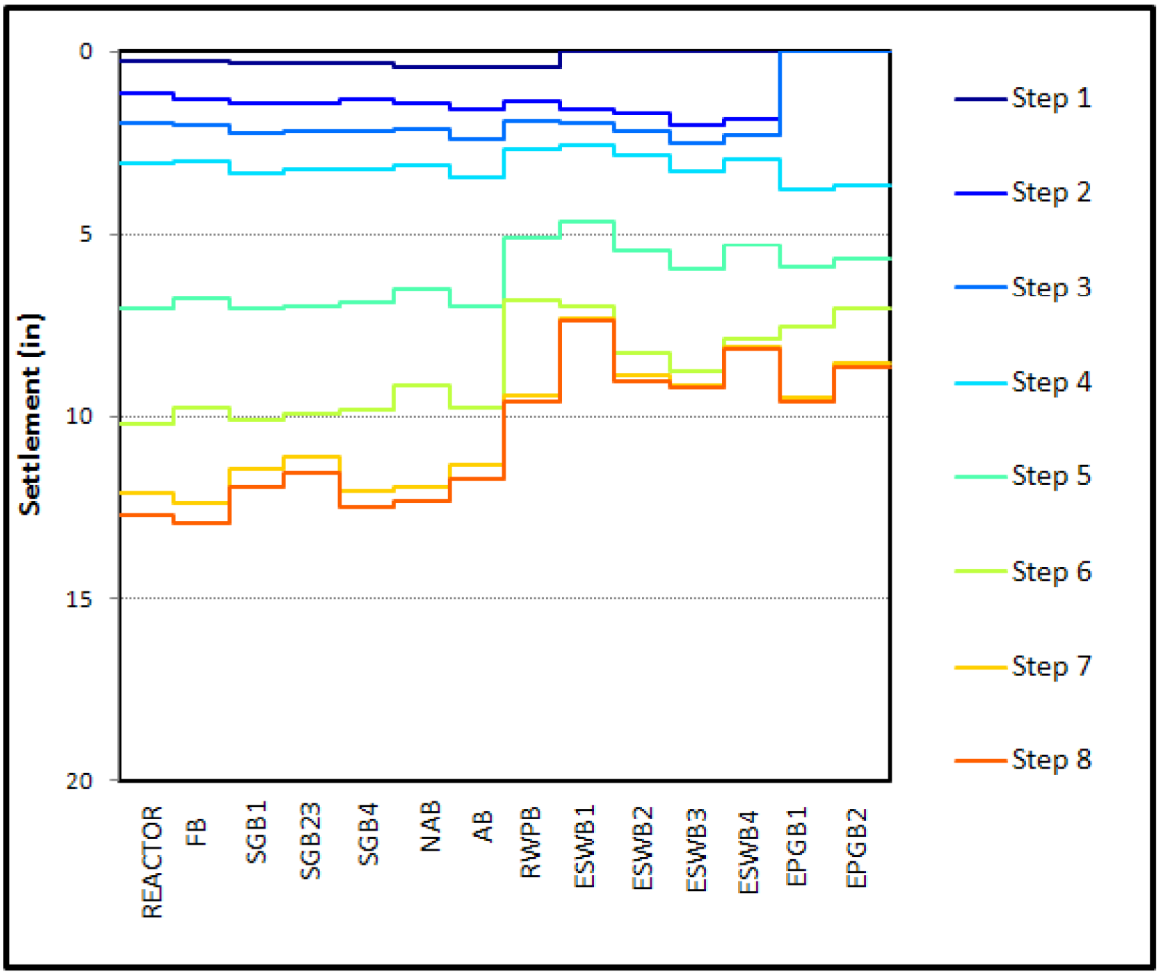

Figure 36 – Settlement at the center of all buildings, MT2 model

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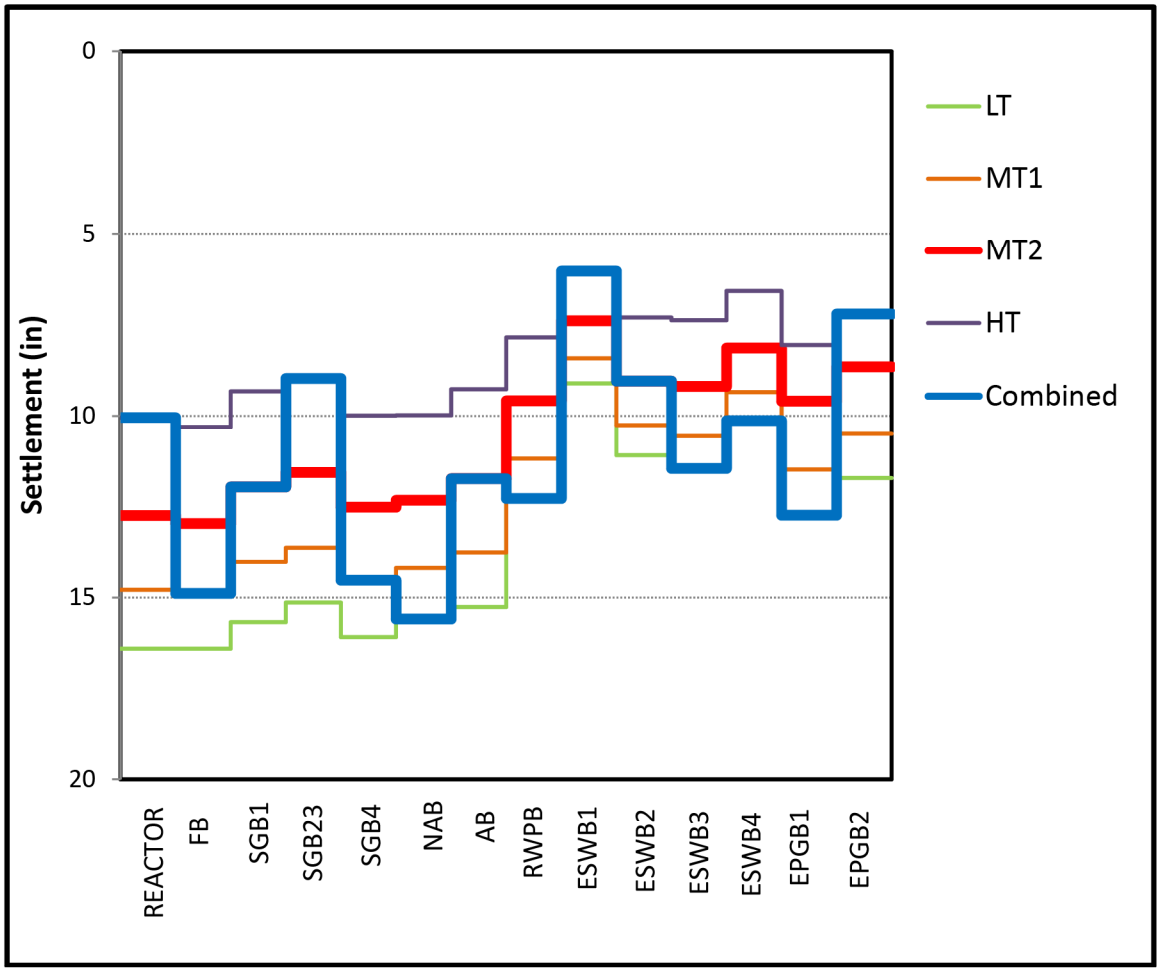

Figure 37 – Settlement at the center of all buildings, all models and combined results.

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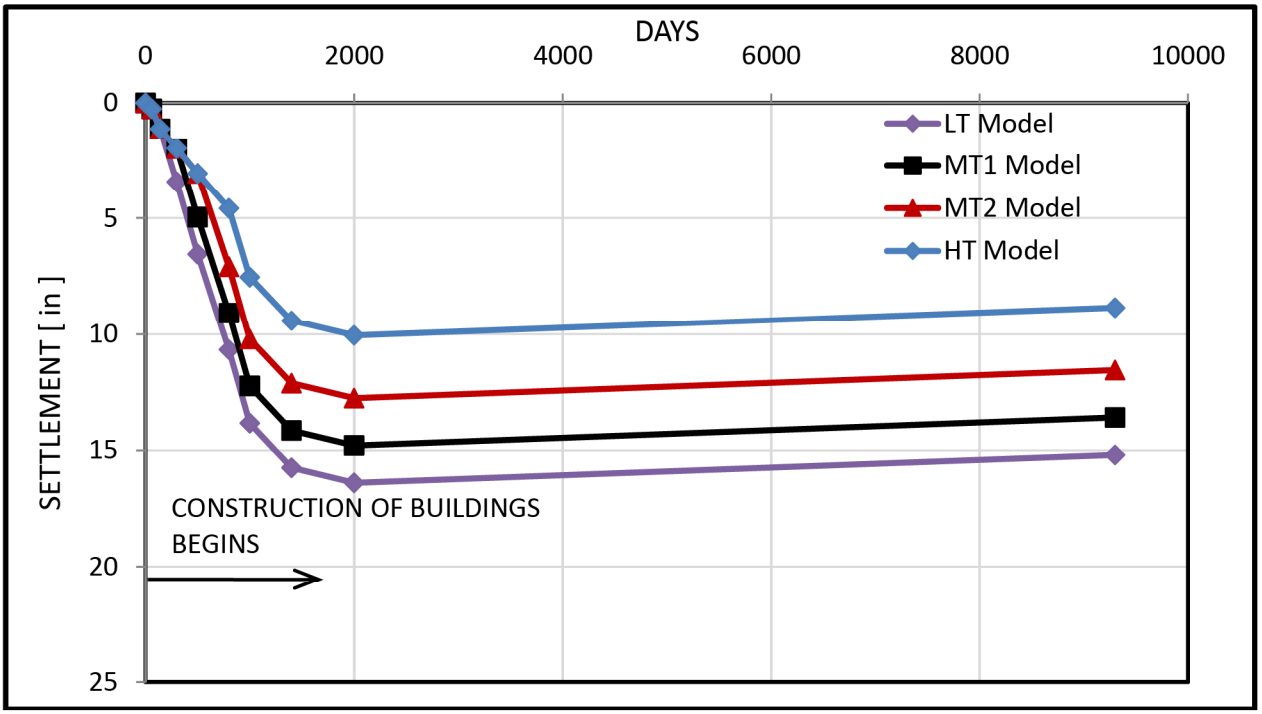

Figure 38 – Settlement at the center of NI common base mat as a function of time

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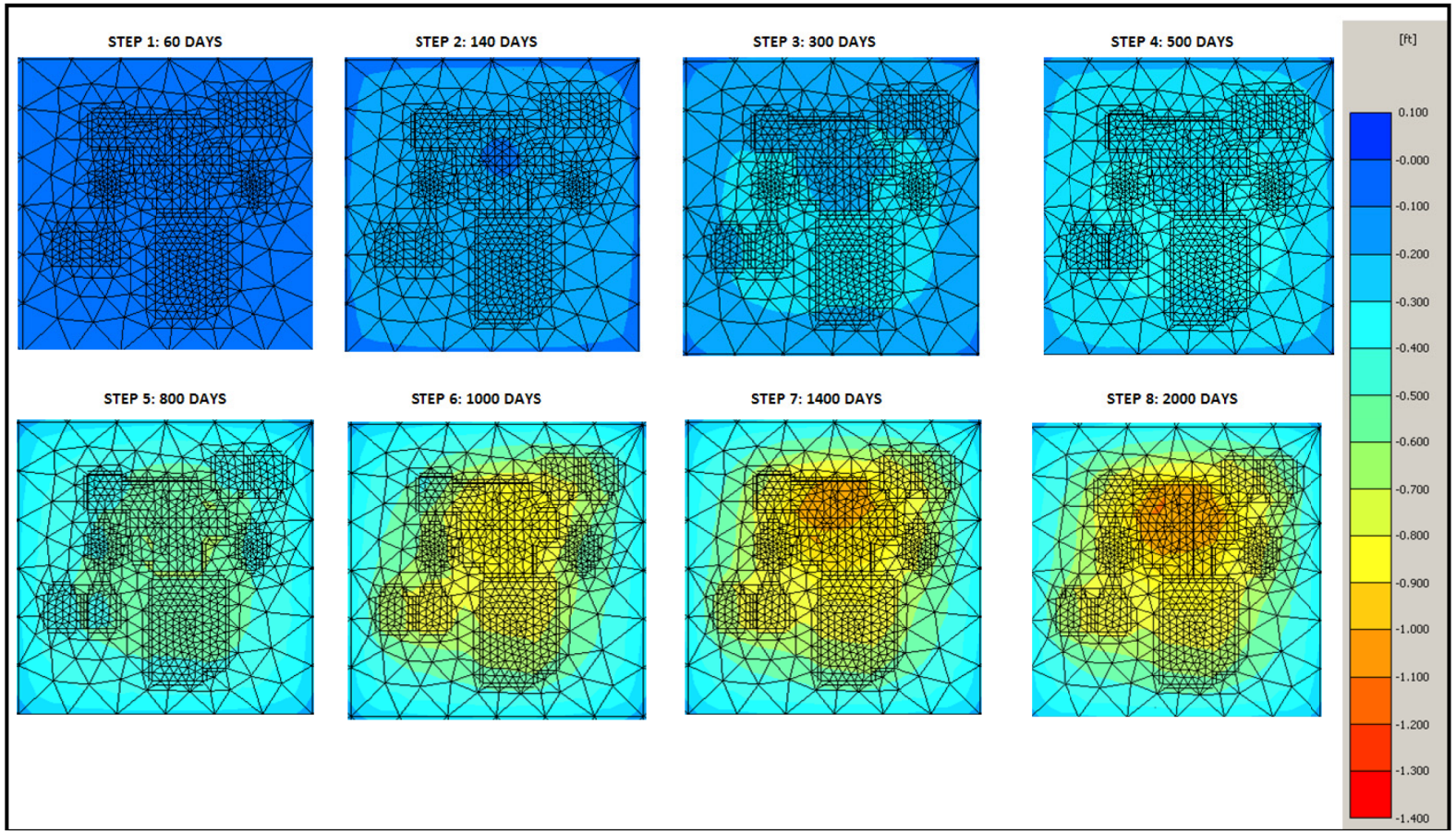

Figure 39 – Settlement on the power block area as a function of time, MT2 Model

APPENDIX 1

COMPUTATIONS FOR THE STIFFNESS INCREASE RATE BY DEPTH FOR NANJEMOY SAND LAYER

STIFFNESS INCREASE RATE COMPUTATIONS:

Equation 1 was used as a basis for the dependence of stiffness on the confining stress (Ref. 14)

$$E_{50} = E_{50}^{ref}\left(\frac{c\cot\phi + \sigma_3^{'}}{c\cot\phi + \rho^{ref}}\right)^m \qquad (1)$$

Given:

E_0=3.17E+6 psf @ -207.5 El (assumption, Ref. 8)

m = 0.5 (assumed property for dense sand) (Ref. 15 and Ref. 17)

c = 0 psf

$\phi=40^o$ $K_0 = 1 - \sin\phi = 0.357$

To find the rate of incrase in E by depth, first, the stiffness at the depth El. -760 ft was determined using Eq. 1 and data from Boreholes 301 and 401. Then, the rate was determined by subtracting the stiffness at El. -207.5 ft from the stiffness at El. -760 ft and dividing the difference in stiffness with the difference in elevation (760 – 207.5 = 552.5 ft). Two rates were calculated using the information from B301 and B401. The rate used was simply the average of these two rates. Initial stresses used in the calculations below were obtained from Ref. 18.

B301 (unit weights for power block area taken from Ref. 4)::

$$\sigma_{v@-207.5}^{'} = 33240\ psf \qquad \sigma_3^{'} = 11867\,psf \qquad \left(\frac{\sigma_3^{'}}{\sigma_1^{'}} = K_0\right)$$

$$3.17E6 = E_{50}^{ref}\left(\frac{11867}{100}\right)^{0.5} \quad E_{50}^{ref} = 291000\,psf$$

$$E_{50_{@760}} = 291000\left(\frac{\sigma_{3_{@760}}^{'}}{100}\right)^{0.5}$$

$$\sigma_{v_{@760}}^{'} = 33240 + (127 - 62.4)(760 - 207.5)$$

$$\sigma_{v_{@760}}^{'} = 68931.5$$

$$\sigma_3^{'} = 24609\ psf$$

$$E_{50} = 4.57E6psf \quad Rate = \frac{dE}{dy} = \frac{4.57E6 - 3.17E6}{760 - 207.5} = 2534psf/ft$$

B401 (unit weights for power block area and construction laydown area taken from Ref. 4)::

$\sigma_v^{'} - 30590\ psf \quad \sigma_3^{'} - 10921\, psf$

$$3.17E6 = E_{50}^{ref}\left(\frac{10921}{100}\right)^{0.5} \quad E_{50}^{ref} = 303339\, psf$$

$$\sigma_{v_{@\,760}}^{'} = 30590 + (125 - 62.4)(760 - 207.5)$$

$$\sigma_{v_{@\,760}}^{'} = 65177\ psf$$

$$\sigma_3^{'} = 23268\ psf$$

$$E_{50} = 303339\left(\frac{23268}{100}\right)^{0.5} = 4.63E6\, psf$$

$$Rate = \frac{4.63E6 - 3.17E6}{760 - 207.5} = 2643\, psf / ft$$

$$AverageRate = \frac{dE}{dy} = \frac{2643 + 2534}{2} = 2589\, psf / ft$$

APPENDIX 2

RATIO OF FINAL STRESS TO PRECONSOLIDATION PRESSURE

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ENGINEERS & CONSULTANTS

By: EOT Date: 10/6/2009 Subject: CCNPP Unit 3 Power Block Area Settlement-Rev.1 Sheet No. A5 of 6

✓By: Date: Project. No. 09-4179.01

γs avg =	120	pcf	average unit weight for soil beneath the foundation (assumption)
Hexc =	41.5	ft	excavation height
w =	11760	psf	average distributed load at the NI Common mat
γ_w =	62.4	pcf	

Layer	Strata	Depth (ft)	Initial stress (psf) (1)	Stress increment $\Delta\sigma$ (psf) (2)	Final stress (psf)	Average stress (4) (psf)	OCR(3)	σ'_p (5) (psf)	$\Delta\sigma / \sigma'_p$
1	IIb1 (SP)	5	2678	9369.3	12048	7363	10	24400	0.49
2	IIb1 (SP)	15	3254	9362.0	12616	7935	10	24400	0.52
3	IIb1 (SP)	25	3802	9337.0	13139	8470	10	24400	0.54
4	IIb2 (SP)	34	4349	9285.1	13634	8991	3	14000	0.97
5	IIb2 (SP)	44	4925	9194.3	14119	9522	3	14000	1.01
6	IIb3 (SP)	54	5501	9062.1	14563	10032	10	32600	0.45
7	IIc (CL / ML)	64	6077	8888.0	14965	10521	4	34000	0.44
8	IIc (CL / ML)	74	6653	8674.2	15327	10990	4	34000	0.45
9	IIc (CL / ML)	84	7229	8425.6	15654	11442	4	34000	0.46
10	IIc (CL / ML)	94	7805	8148.1	15953	11879	4	34000	0.47
11	IIc (CL / ML)	104	8381	7848.7	16229	12305	4	34000	0.48
12	IIc (CL / ML)	114	8957	7534.0	16491	12724	4	34000	0.49
13	IIc (CL / ML)	124	9533	7210.5	16743	13138	4	34000	0.49
14	IIc (CL / ML)	134	10109	6883.7	16992	13551	4	34000	0.50
15	IIc (CL / ML)	144	10685	6558.4	17243	13964	4	34000	0.51
16	IIc (CL / ML)	154	11261	6238.4	17499	14380	4	34000	0.51
17	IIc (CL / ML)	164	11837	5926.8	17764	14800	4	34000	0.52
18	IIc (CL / ML)	174	12413	5625.7	18038	15226	4	34000	0.53
19	IIc (CL / ML)	184	12989	5336.6	18325	15657	4	34000	0.54
20	IIc (CL / ML)	194	13565	5060.5	18625	16095	4	34000	0.55
21	IIc (CL / ML)	204	14141	4798.0	18939	16540	4	34000	0.56
22	IIc (CL / ML)	214	14717	4549.3	19266	16991	4	34000	0.57
23	IIc (CL / ML)	224	15293	4314.2	19607	17450	4	34000	0.58
24	IIc (CL / ML)	234	15869	4092.5	19961	17915	4	34000	0.59
25	IIc (CL / ML)	244	16445	3883.7	20329	18387	4	34000	0.60

(1) Calculated with an average effective unit weight
(2) Increment of vertical stress at the center of the rectangular loading pattern; the Boussinesq solution is used (Ref 8).
(3) Ref. 13
(4) Average stress = (initial stress + final stress) / 2.
(5) Preconsolidation pressure (Ref. 13)

Layer	Strata	Depth (ft)	Initial stress (psf) (1)	Stress increment Δσ (psf) (2)	Final stress (psf)	Average stress (4) (psf)	OCR(3)	σ'_p (5) (psf)	$\Delta\sigma / \sigma'_p$
26	III (SP)	254	17021	3687.4	20708	18865	2	30000	0.69
27	III (SP)	264	17597	3503.0	21100	19348	2	30000	0.70
28	III (SP)	274	18173	3329.7	21503	19838	2	30000	0.72
29	III (SP)	289	19037	3089.6	22126	20582	2	30000	0.74
30	III (SP)	309	20189	2802.9	22992	21590	2	30000	0.77
31	III (SP)	329	21341	2550.0	23891	22616	2	30000	0.80
32	III (SP)	349	22493	2326.7	24820	23656	2	30000	0.83
33	III (SP)	369	23645	2129.2	25774	24709	2	30000	0.86
34	III (SP)	389	24797	1953.9	26751	25774	2	30000	0.89
35	III (SP)	409	25949	1798.0	27747	26848	2	30000	0.92
36	III (SP)	429	27101	1658.9	28760	27930	2	30000	0.96
37	III (SP)	449	28253	1534.6	29787	29020	2	30000	0.99
38	III (SP)	469	29405	1423.0	30828	30116	2	30000	1.03
39	III (SP)	489	30557	1322.6	31879	31218	2	30000	1.06
40	III (SP)	509	31709	1232.1	32941	32325	2	30000	1.10
41	III (SP)	529	32861	1150.1	34011	33436	2	30000	1.13
42	III (SP)	549	34013	1075.9	35089	34551	2	30000	1.17
43	III (SP)	569	35165	1008.3	36173	35669	2	30000	1.21
44	III (SP)	589	36317	946.7	37264	36790	2	30000	1.24
45	III (SP)	609	37469	890.5	38359	37914	2	30000	1.28
46	III (SP)	629	38621	838.9	39460	39040	2	30000	1.32

(1) Calculated with an average effective unit weight

(2) Increment of vertical stress at the center of the rectangular loading pattern; the Boussinesq solution is used (Ref .8).

(3) Ref. 13

(4) Average stress = (initial stress + final stress) / 2.

(5) Preconsolidation pressure (Ref. 13)