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Numerical Prediction of The Creep Settlement of The Improved Soft Soil Under Embankments

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ABSTRACT

The accurate prediction of consolidation settlement is one of the most important aspects of soft clay modeling, which has always been a cause of concern for geotechnical engineers. The objective of this study is to investigate the applicability of a few different computational tools for the prediction of consolidation settlement in soft ground. This argument can be summarized by the question of whether secondary consolidation begins after or during primary consolidation. As indicated in Hypothesis A, there is no creep compression during the primary consolidation period, but the creep compression occurs only in the secondary compression starting at t_{EOP} (time of primary consolidation). According to Hypothesis B, the creep occurs throughout the primary consolidation period. A creep constitutive model, namely Soft Soil Creep (Plaxis 3D), which uses Hypothesis B, and a semi-analytical software (Settle 3D), which uses Hypothesis A, were employed to study the behavior of soft soil under embankments with the same parameters. Two case studies were used in the analysis: The first case was the NHP Levee, which was constructed in California. The second case is the I-95 Boston embankment, which was constructed in 1965 over a deep layer of Boston Blue Clay. For each case, a comparison has been made between the Hypothesis A and B results with field data. The results show that the soil creeps before completing primary consolidation. Hence, Hypothesis B is more accurate for evaluating the total consolidation settlements.

1. Introduction

Creep in soils undergoes compressive deformation over time when effective stresses are constant. This occurs through the rearrangement of the soil particles. Settlement due to primary and secondary consolidation (creep) is a challenging issue with a long history. Which theory is adopted? Hypothesis A or Hypothesis B. The behavior of soils subjected to creep loads is described as the most important topic, especially for soft, organic, and loose soils.

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In addition, good experimental and theoretical studies are important for predicting the behavior of soil and the effects of ground deformations on structures.

When the soil is loaded, deformation occurs due to stress changes. The total vertical deformation resulting from the load is called a settlement. In general, the soil settlement caused by a load may be divided into three broad categories with respect to the mode of occurrence: immediate settlement, primary consolidation settlement, and secondary consolidation settlement, which is also called creep.

Hypothesis A assumes that creep compression does not occur during the primary consolidation process and considers that creep deformations separately appear shortly after the time of primary consolidation or we can say after the dissipation of excess PWP. Otherwise, Hypothesis B suggests that creep compression does not wait for the primary consolidation to finish and starts simultaneously during the process of PWP dissipation. Hence, creep is considered to occur throughout the whole compression process. Since the introduction of Hypotheses A and B, the relationship between the creep and the primary consolidation has been a controversial issue. A lot of numerical and experimental studies support Hypothesis A. Many other studies support Hypothesis B.

Numerical studies using computational tools were performed to calculate the settlement owing to creep. One of them, which supports Hypothesis A, is SETTLE 3D software, and the other is the PLAXIS 3D software, which supports Hypothesis B. This research compares the two hypotheses using case studies for embankments on soft soil with and without improvement techniques, and compares the results from F.E. simulations for these cases with field measurements to determine which hypothesis is more accurate.

Since the introduction of Hypotheses A and B, the relationship between the creep and the primary consolidation has been a controversial issue. A lot of numerical and experimental studies support Hypothesis A [1-5]. Other studies support Hypothesis B [6-8].

Many researchers, e.g. [9-15], have accepted the assumption of creep compression starting during the primary consolidation stage, and this expectation is consistent with Hypothesis B. However, a variety of advanced and complicated numerical programs and constitutive models are required to solve many linear and nonlinear partial differential equations to determine the value of consolidation settlements based on Hypothesis B, which can be too complicated for use by engineers today.

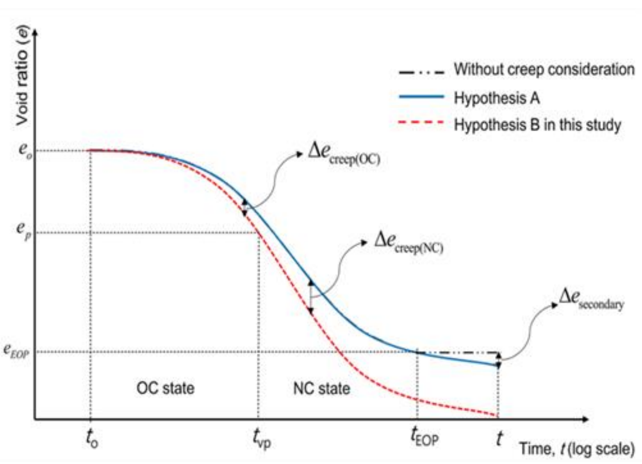


Fig. 1. Relations between void ratio(e), effective stress, and log(t) under NC and OC conditions. [16]

Degago et al. [17] mentioned that the numerical model used to illustrate the experimental observations that were already used to support Hypothesis A is based on the isotache concept (SSC model). However, it was highlighted that other results that support Hypothesis A have been presented. In an earlier study, Hypothesis B was validated for four specific test embankments [6].

This study targets how soft clay consolidates. Our main goal is to determine the settlement due to creep using Hypotheses A and B, and then compare the results with laboratory tests and in-situ measurements. This study investigates the following two case studies.

Two case studies are used in the analysis; the first case is the NHP Levee, which was constructed in California. It is one of several existing levees that surround the Hamilton Army Air Field (HAAF) Base Wetlands Restoration project. This 3.35 m high embankment was constructed over a 12 m thick layer of Bay Mud in San Francisco.

The second case is the I-95 Boston embankment, which was constructed in 1965 over a deep layer of Boston Blue Clay as a portion of Interstate Highway I-95 in the North of Boston. The underlying soil layers consist of soft peat, poorly graded marine sand, a layer of Boston Blue Clay (40 m), dense glacial till, and grey argillite bedrock.

The results and curves will help us to follow Hypothesis A, which assumes an end-of-primary (EOP) void ratio-effective vertical stress relationship independent of the duration of primary consolidation, t_{eop}, or Hypothesis B, which assumes an EOP void ratio-effective vertical stress dependent on the duration of primary consolidation. Finally, this helps us determine which method should be applied.

2. Theories of Soil Creep

There are two major approaches for predicting soil settlement that take secondary compression into account. These approaches are referred to as Hypotheses A and B [18]. Figure 1 provides a brief description of Hypotheses A and B. The start of the time at which the creep effect occurs is when these two hypotheses diverge.

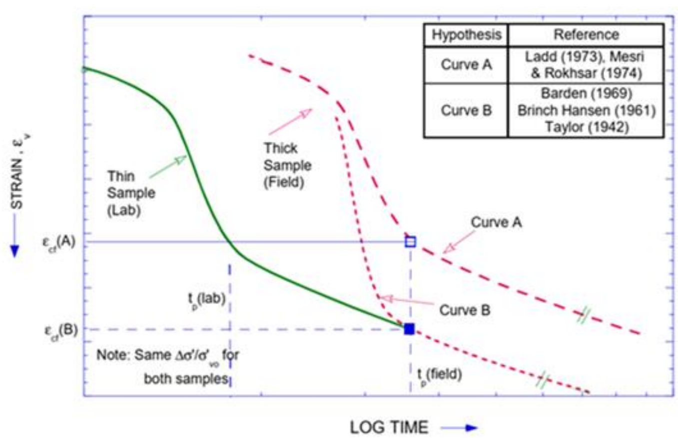


Fig. 2. Hypothesis (A) vs Hypothesis (B) [18]

2.1. Hypothesis A

Hypothesis A assumes that creep compression does not occur during the primary consolidation process, and considers that creep deformation appears shortly after the end of primary consolidation or after the dissipation of excess pore water pressure. Otherwise, Hypothesis B suggests that creep compression does not wait for the primary consolidation to finish and starts simultaneously during the process of PWP dissipation. Hence, the creep is considered to occur throughout the whole compression process. Owing to the fact that the primary compression is independent of creep deformation, the void ratio (e) at the end of primary consolidation (EOP) is supposed to be unique, despite the thickness of soil layers or drainage conditions.

Hypothesis A was formally accepted [5,18,19]. In addition, the concept introduced by Mesri and his co-workers [1,2] is an important method used to evaluate the total exhibited settlement of soils. Moreover, this concept supports Hypothesis A. As indicated in Hypothesis A, there was no creep compression during the primary consolidation period, and compression only started during the secondary consolidation at the t_{EOP}. "That interrelationship causes the uniqueness of e_{EOP} and effective stress at the end of primary consolidation" [4].

- Hypothesis A Formula [3]

$$S_{total\ A} = S_{"primary"} + S_{"secondary"}$$

$$= \begin{cases} U_v S_f & for\ t \le t_{EOP} \\ U_v S_f + \frac{C_{\alpha e}}{(1+e_0)} \log\left(\frac{t}{t_{EOP}}\right) H & for\ t > t_{EOP}\ z \end{cases} \quad (1)$$

$$C_{\alpha e} = \frac{-\Delta e}{\Delta\ log\ t} \quad (2)$$

Where: S_{totalA} represents the computed consolidation settlement using Hypothesis A, S_{prim} is the primary consolidation settlement, S_{sec} is the secondary consolidation, e_o is the initial void ratio and S_f is the final settlement $C_{\alpha e}$ is the coefficient of secondary compression, U_v is the degree of consolidation in the vertical direction and H is the layer thickness.

2.2. Hypothesis B

Hypothesis B suggests that creep compression does not wait for the primary consolidation to finish and starts simultaneously during PWP dissipation. Hence, creep was considered to occur throughout the compression process. Because it is the primary compression independent of creep deformation, the void ratio (e) at the EOP is supposed to be unique, despite the soil layer thickness or drainage conditions. The geotechnical community has previously accepted this theory [6,12,13,17,20,21], although the prediction of settlement values using this method seems to be quite complicated compared to the results from Hypothesis A.

- Hypothesis B: Formula [22].

$$S_{total\ B} = S_{"primary"} + S_{creep}$$

$$= U_v S_f + \frac{C_{\alpha e}}{(1+e_0)} log\left(\frac{t}{t_0^c}\right) H \quad anytime\ t \ge 1\ day \quad (3)$$

Where:

S_{totalB} the consolidation settlement using Hypothesis B, $S_{"primary"}$ is the primary consolidation settlement, $S_{"creep"}$ is the secondary compression, e_o is the initial void ratio, and S_f is the final settlement of "primary consolidation", $C_{\alpha e}$ is the coefficient of secondary compression, and U_v is the degree of consolidation in the vertical direction.

3. Numerical Model

In this study, a creep constitutive model, namely Soft Soil Creep (Plaxis 3D), uses Hypothesis B, and another F.E model (Settle 3D), which uses Hypothesis A, is employed to study the behavior of soft soil under embankments with the same parameters for the two models.

Analytical Exact solutions are valid for a limited range of geotechnical problems. Otherwise, approximate solutions can be obtained from the numerical analyses. The finite element (FE) method is a modeling technique that divides the surface area, structure, or region into a finite number of elements. In practice, 3D modeling using FE analyses is commonplace. In this study, the FE analysis tool Plaxis 3D v 2021 was used for the numerical modeling.

The PLAXIS 3D program can be used to analyze and model many geotechnical problems. For example, the construction of tunnels, deep excavation, and the flow of groundwater in embankment projects. etc. The main difference between geotechnical analysis and other types of structural analysis is the complexity associated with real soil behavior and the presence of pore fluid.

Selecting an appropriate soil model to accurately simulate soil behavior is an important and complicated issue. This research used a model provided by Plaxis 3D FE software called the Soft Soil Creep Model (SSCM). This model can be used to study the behavior of soft soil under the application of external loads, and is used in the analyses of the investigated case studies.

Other models have been used to simulate embankment fill and sandy layers in some cases, such as the Mohr Column (MC) model and Linear Elastic (LE) model.

3.1. Hypothesis A Solution

For the purpose of comparison between the numerical analysis and analytical solutions, some of the analyzed conditions were solved using analytical methods.

In this regard, this study employed a software code known as Settle 3D to achieve this goal. It also determines the consolidation analysis by applying one-dimensional consolidation equations. These options indicate that it can compute the settlement either using the linear model or the nonlinear model, depending on the model to be utilized.

The linear consolidation model in Settle 3D is the conventional solution for consolidation and is expressed as follows:

$$\Delta\varepsilon = m_v \ \Delta\sigma' \tag{4}$$

where: $\Delta\boldsymbol{\varepsilon}$ is the vertical strain.

m_v is the soil coefficient of volume change.

$\Delta\sigma`$ is the increase in effective stresses.

In the non-linear material model, the modulus depends on the applied stress. The relationship between the moduli is usually indicated by the void ratio against the logarithm of the effective stress (Equ 5).

$$\Delta\varepsilon = \frac{C_r}{1+e_0} log\left(\frac{P_c}{\sigma_i'}\right) + \frac{C_c}{1+e_0} log\left(\frac{\sigma_f'}{P_c}\right) \tag{5}$$

Where:

$\Delta\boldsymbol{\varepsilon}$ is the vertical strain, P_c is the pre-consolidation stress, C_c is the compression index, C_r is the recompression index, e_o is the void ratio, $\sigma`_i$ is the initial stresses, and $\sigma`_f$ is the final stresses.
For the secondary compression calculation, Settle3D uses the analytical method, which is based on the parameter C_α, which is the secondary compression index. The change in vertical strain between times t_1 to t_2 within the secondary compression period ($\Delta\boldsymbol{\varepsilon}_s$) is calculated by the following formula:

$$\Delta\varepsilon_s = \frac{C_\alpha}{1+e_p} log\left(\frac{t_2}{t_1}\right) \tag{6}$$

where e_p is the void ratio at the end of the primary consolidation.

In Settle3D, it is possible to model several options for settlement cases. One of these is the stress computation method, which is one of the major parameters of the profile generated by the software. Four different methods are used for calculating the stresses generated throughout the soil body, namely, Boussinesq, 2:1, Westergaard, and Multiple Layer methods. The Boussinesq method is used in this study for stress calculations.

For clay layers, there is no option of immediate settlement, which has been provided with the layered fill material, which is cohesionless in nature.

Modeling the secondary compression behavior in SETTLE 3D can be provided by two choices. The first one is to initiate the secondary compression when the primary consolidation comes to 98% of its ultimate value. The second option assumes that the secondary settlement begins after placing the load. The first option was used in the analysis of the aforementioned case studies.

3.2. Hypothesis B Solution

The SSC model is the most common elasto-viscoplastic soil model commercially available with PLAXIS 2D and 3D software, and it has been used as the base for the majority of numerical modeling simulations described in this research. The total strain (ε) in the SSC model is composed of an elastic component (ε_e) and a viscoplastic (creep) component (ε_c),
see Eq. 7.

Buisman [23] introduced the following relation to explain creep behavior under the application of constant effective stress:

$$\varepsilon = \varepsilon_c - C_B \log\left(\frac{t}{t_c}\right) \tag{7}$$

Where: ε_c is the strain up to the end of consolidation, *t* is the time measured from the beginning of the loading, t_c *is* the time to the end of primary consolidation and C_B is a material constant.

The visco-plastic component can be further divided into strains during and after consolidation respectively (see Figure 3), where σ'_o is the initial effective stress, σ' is the final effective stress, σ_{po} is the initial pre-consolidation stress before loading, σ_{pc} is the pre-consolidation stress at EOP and $t' = t - t_c$ is the effective creep time (with t and tc denoting the times from the beginning of loading and to EOP respectively).

Parameter τ_c was determined using a standard oedometer test.

Soft Soil Creep Model is a more sophisticated Plaxis model based on the Soft Soil Model, but also considers secondary compression or creep during the other calculations.

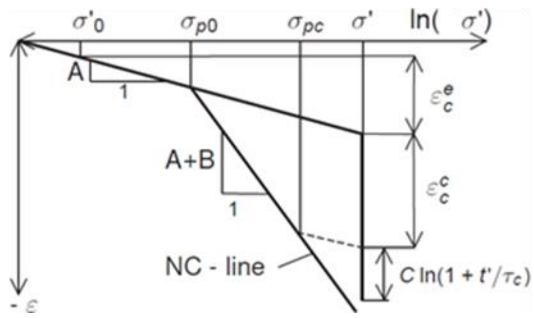


Fig. 3. Idealized stress-strain curve from an oedometer test [24]

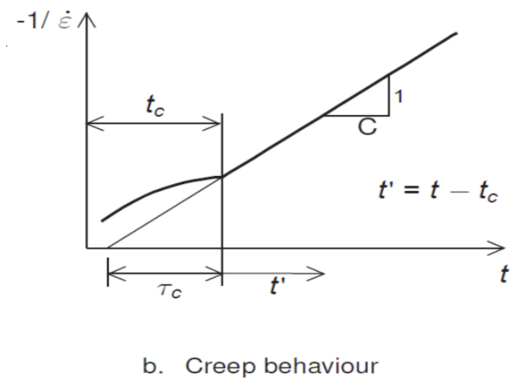


Fig. 4. Consolidation and creep behavior in a standard oedometer test [24]

4. Determination of The End of Primary Consolidation (EOP)

The end time of the primary consolidation process is determined using the following equation:

$$Tv = \frac{Cv * t}{H^2} \quad (8)$$

Where T_v is the time factor

c_v is the consolidation coefficient

H is the drainage path

t is the time according to the degree of consolidation

4.1. The logarithm-of-Time Method [25]

To determine the EOP time for known observed data of settlement versus time, Casagrande [25] suggested an accepted method based on a graphical solution.

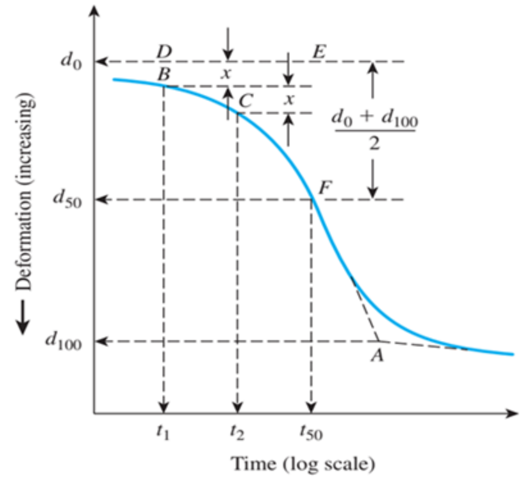


Fig. 5. Log of time method to determine the time of end of primary consolidation [26]

$$T_{50} = 0.197 = \frac{Cv * t_{50}}{H^2} \quad (9)$$

T_{50} is the time factor for a 50% average degree of consolidation.

t_{50} is the time corresponding to the 50% average degree of consolidation.

4.2. The Square-Root-of-Time Method [27]

The period time of the EOP can be determined using the Taylor method from recorded dial gauge readings versus time data, and we can use the graphical procedure proposed by Taylor [27]:

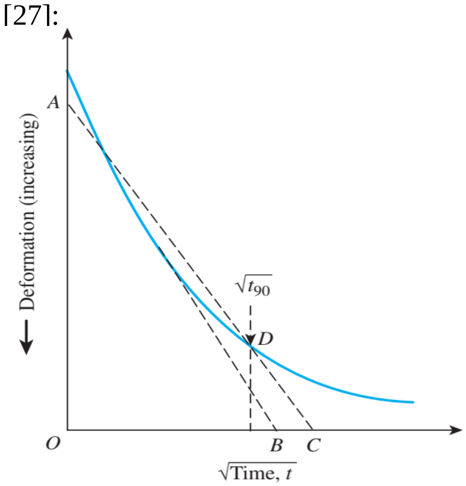


Fig 6. Time square-root fitting method for determination of end-of-primary consolidation [26]

$$T_{90} = 0.848 = \frac{Cv * t_{90}}{H^2} \quad (10)$$

T_{90} is the time factor for a 90% average degree of consolidation.

T_{90} is the time corresponding to the 90% average degree of consolidation.

5. The New Hamilton Partnership Levee (NHPL) Case Study

The (NHPL) is one of several existing embankments that surround the Hamilton Army Air Field (HAAF) Base. The levee is located in the City of Novato, north of San Francisco, California. As part of a federal program, a new levee system is required to protect neighboring residential, agricultural, and industrial areas from flooding. Hence, it is required to construct a group of levees around the perimeter of the new wetlands.

One of these is the NHPL, which is a new embankment constructed on a deep clay layer of Bay Mud. URS, the geotechnical consulting firm, performed an intensive site investigation and observation program with a new instrumentation and analysis program to study the behavior of the NHPL. The analysis program was performed to calibrate the analytical and finite element (FE) models for the purpose of using it in the design of this and other levees.

The existing NHPL alignment is located on a thick layer (9-12 m) of recent San Francisco Bay Mud (SFBM). The NHPL was built between March and October 1996 and is approximately 2,200 m long and 3.65 m high as a flood-controlled embankment structure for the New Hamilton Partnership residential area.

A massive field-testing program was then performed. The field tests included borings and sampling, (FVT),(CPTU), and geophysical studies. In addition, field instrumentation at the test sections included piezometers to measure in situ PWP, inclinometers to measure lateral deformations, and Sondex devices to measure subsurface vertical settlement profiles.

The URS program for site investigation started on December 19, 2001, and continued to February 2003. It included tubed sample borings for the laboratory tests. Settlement and pore water pressure readings were taken under the embankment at various levels using the previously described instrumentation. The readings were recorded periodically along the NHPL about 3.8 and 5.2 years after construction.

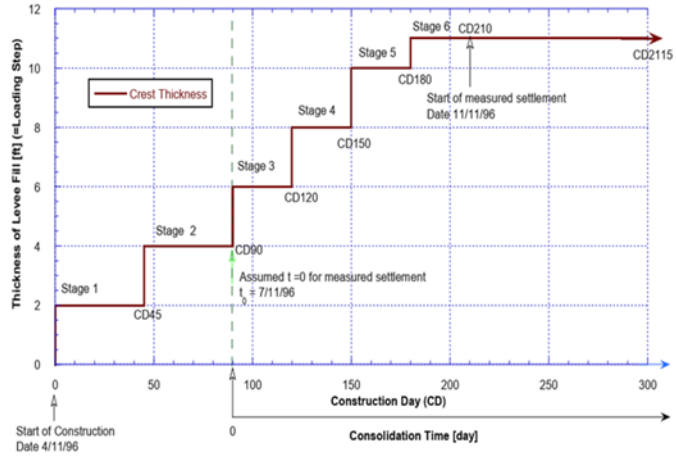


Fig. 7. Construction sequence for NHP Levee [25]

5.1. Hypothesis A (SETTLE 3D Model)

Analytical solutions of the targeted case were obtained using Settle3D, which uses the Terzaghi consolidation theory. The 3D geometry of the case study was defined in the model, as shown in Figure 8. The G.W.T was set to a 5.0 ft (1.50 m) depth, and the stages of construction were defined previously in the phase description of the case.

Figure 8 illustrates the geometry of the 3D model used for modeling and analysis of the case study. A nonlinear soil model was used, and the determined parameters are listed in Table 1.

Figure 9 shows the results from the semi-analytical solution using SETTLE 3D, which represents Hypothesis A.

According to the results from the comparison between the time and settlement curves, we can see that the results from Hypothesis A are in acceptable agreement with the field observations of settlement values. The results of the FE model were less than the field measurements from the beginning and were almost the same as the 250-day mark.

Table 1

Constitutive model and soil parameters for NHP Levee for SETTLE 3D

Type	Parameter	Levee Fill	Cracked Pavement	Base Course	Bay Mud Crust	Bay Mud 1	Bay Mud 2	Alluvium (Old Bay Mud)
Model		M.C	M.C	M.C	S.S.C	S.S.C	S.S.C	Linear Static
Initial Stress state Parameters	γ (kn/m^3)	20	24	23	15.7	14.5	14.5	20
	e_o	-	-	-	2	2	2	-
	ν	0.3	0.2	0.2	0.15	0.15	0.15	0.3
	OCR	-	-	-	5	2	1.5	-
Critical State Parameters	Cc	-	-	-	1.20	1.20	1.20	-
	C_r	-	-	-	0.18	0.18	0.36	-
	$C\alpha$	-	-	-	0.02484	0.02484	0.02484	-
	C_V (m^2/day)	-	-	-	0.06	0.06	0.06	-
	E (kn/m^2)	1440	9570	9570	-	-	-	47880
Strength Parameters	c' (kn/m^2)	1.0	2.0	2.0	2.0	2.0	2.0	1.0
	φ'	37	35	35	25	25	25	37
Flow Parameters	K_x & Ky (m/day)	0.1	1	1	6.00E-04	6.00E-04	6.00E-04	0.001
	kz (m/day)	0.1	1	1	4.50E-04	4.50E-04	4.50E-04	0.001

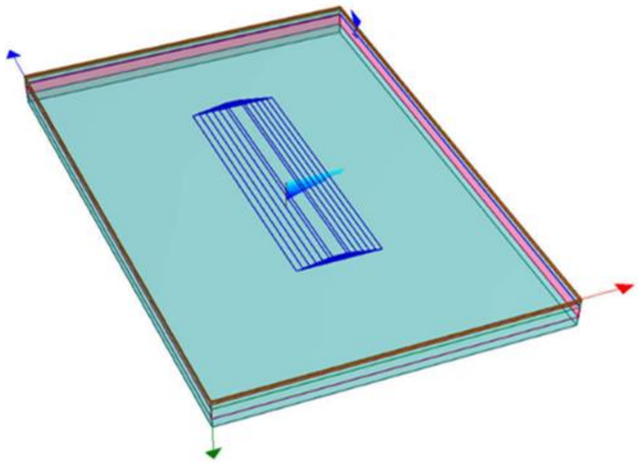


Fig. 8. Settle 3D Model for NHP Levee

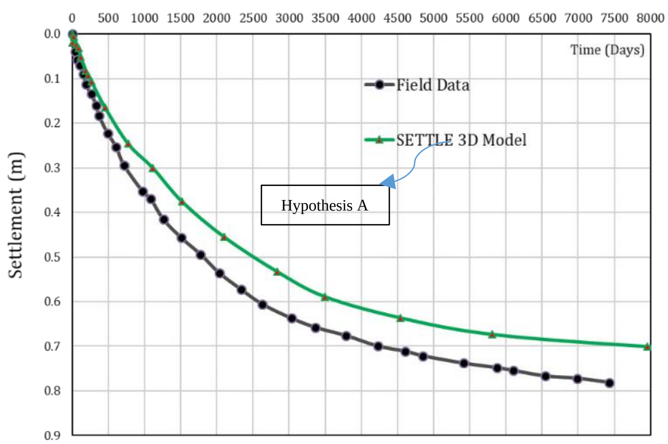


Fig. 9. Total Settlement– - time curves using the semi-analytical method by SETTLE 3D for NHP Levee.

Then, the results of the numerical model aligned with the field data until the end of the field readings at 2000 days. At the 2000-day mark, the settlement using Hypothesis A is almost 0.43 m while the field measurement is 0.52 m with relative errors of about 20%.

5.2. Hypothesis B (PLAXIS 3D Model)

For the modeling and analysis of the NHP Levee, the soft soil creep (SSC) model was used to model the three layers of Bay Mud clay. As mentioned, the main advantage and characteristic of the SSC model is the accurate depiction of the behavior and response of soft soil deposits while considering secondary consolidation.

The effective strength parameters in undrained conditions were used for the clay layers, as this will allow PLAXIS 3D 2021 to automatically calculate the increase in shear strength parameters with consolidation. The reclamation sand was modeled using the Mohr-Coulomb (MC) model and drained soil conditions. The soil parameters used in the FEM are stated inTable 3.

As the embankment lies over a large area, modeling the entire embankment is found to be time-consuming without any technical gain. Thus, a portion of the embankment was modeled and used to represent the embankment. Moreover, a sensitivity analysis was performed to select appropriate dimensions for the FE model. As shown in Figure 11.

Levee fill was from elevation of the levee crest at 9.4 ft down to the levee base at -1.6 ft, giving an 11 ft levee height. The widths

of the levee fill were 23.0 ft at the top and 89.0 ft at the bottom. The levee fill was divided into six clusters (sub-layers).

Figure 10 shows a cross-section of the embankment. Slopes have a 1V:3H inclination, and the embankment itself is composed of sandy soil.

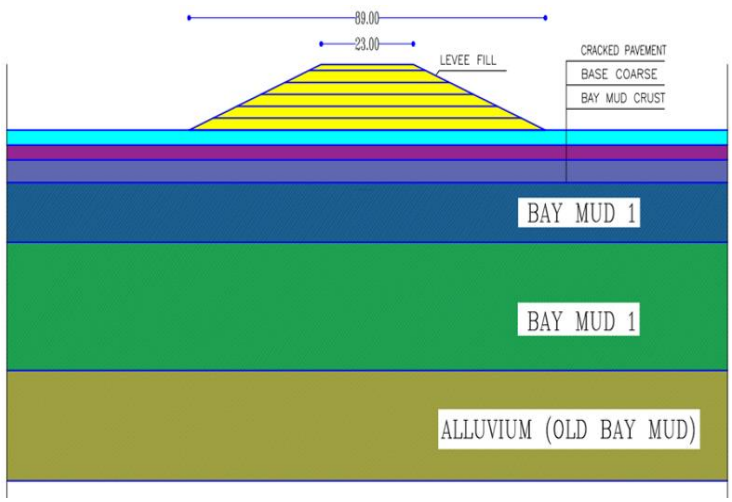
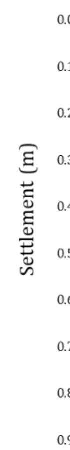


Fig. 10. Geometry and soil conditions of the prototype model based on Bay Mud Clay under NHP Levee.

The considered boundary conditions for all the vertical boundaries were determined by allowing the application of vertical settlements while preventing horizontal movement from taking place, and the drainage of the pore water through all the vertical boundaries was prohibited. However, for the lower horizontal boundary of the model, both the vertical and horizontal movements are prevented, while the drainage of the pore water isn't allowed to take the presence of an old Alluvium layer underlying the Bay Mud clay layer into consideration. The 3D model geometry and the meshing concept are shown in Figure 11.

Mesh sensitivity tests are carried out to ensure that the meshing is dense enough to produce reasonable results, so a very fine mesh was chosen. Additionally, the bottom border is constrained in both directions, while the side boundaries are restrained horizontally. For consolidation analysis, the drainage boundaries at the ground surface and bottom were opened, whereas the lateral borders were closed.



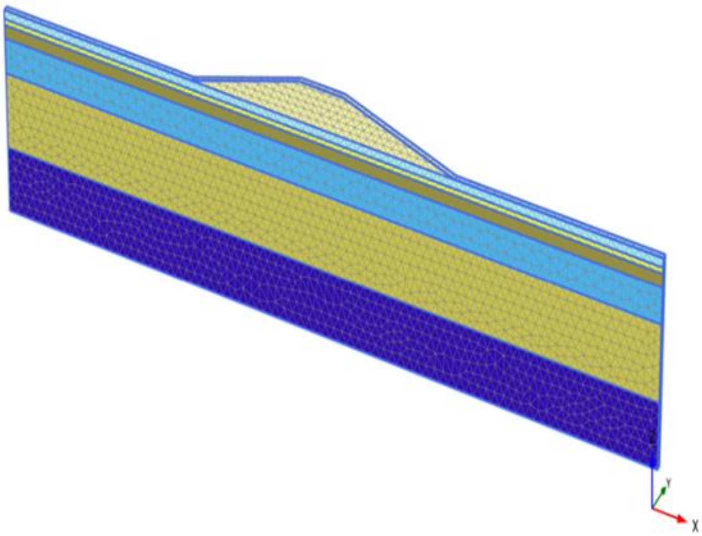


Fig. 11. 3D model for NHP Levee using very fine meshing (10183) Elements

In this section, we have a comparison between the results of 3D FEM and field measurements. The field settlement and the corresponding settlement results of the numerical model are shown in Figure 12.

Generally, good agreement was found between the results of the numerical model and field measurements. The results of the numerical model are almost the same as the field measurements until the 700-day mark is reached.

Then, the results of the numerical model aligned with the field data until the end of the field readings at 2000 days, with slightly lower settlement values compared with the field measurements.

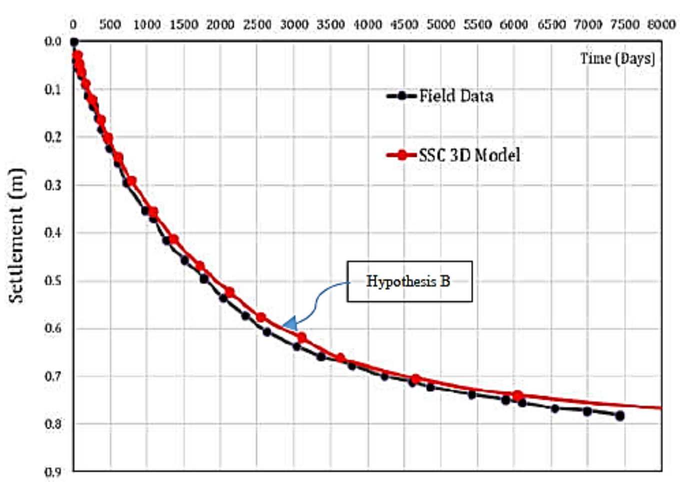


Fig. 12. Comparison between the 3D SSC FEM and Field measurements.

5.3. Discussion

The results show that the soil creeps during primary consolidation, rather than completing it after the primary consolidation. The settlement calculated by the PLAXIS SSC 3D model (Hypothesis B) is larger than the settlement calculated by Settle 3D, which represents the analytical solution (Hypothesis A) by an average of 9.30%. In addition, it can be concluded that the settlement values were almost the same in the early stage up to 500 days.

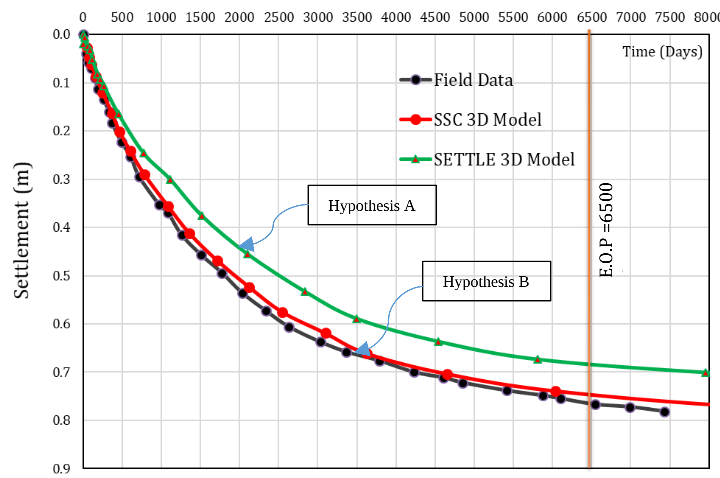


Fig. 13. Comparison between SSC 3D Model, Settle 3D model, and field measurements (Case 1).

Table 2

Construction phases of the NHP Levee. [28]

Phase	1	2	3	4	5	6	7	8	9	10	11	12	13
Analysis	CS	C	CS	C	CS	C	CS	C	CS	C	CS	C	C
Time	1	45	1	45	1	30	1	30	1	30	1	30	Δu = 1 kPa
Cum. Time	1	46	47	92	93	123	124	154	155	185	186	216	-

Table 3

Constitutive model and soil parameter for NHP Levee for Plaxis 3D

Type	Parameter	Levee Fill	Cracked Pavement	Base Course	Bay Mud Crust	Bay Mud 1	Bay Mud 2	Alluvium (Old Bay Mud)
Model		M.C	M.C	M.C	S.S.C	S.S.C	S.S.C	Linear Static
Initial Stress state Parameters	γ (kn/m^3)	20	24	23	15.7	14.5	14.5	20
	eo	-	-	-	2	2	2	-
	ν	0.3	0.2	0.2	0.15	0.15	0.15	0.3
	OCR	-	-	-	5	2	1.5	-
Critical State Parameters	λ*	-	-	-	0.174	0.174	0.174	-
	κ*	-	-	-	0.052	0.052	0.104	-
	μ*	-	-	-	3.60E-03	3.60E-03	3.60E-03	-
	Cα	-	-	-	0.02484	0.02484	0.02484	-
	E (kn/m^2)	1440	9570	9570	-	-	-	47880
Strength Parameters	c' (kn/m^2)	1.0	2.0	2.0	2.0	2.0	2.0	1.0
	φ'	37	35	35	25	25	25	37
Flow Parameters	K_x & Ky (m/day)	0.1	1	1	6.00E-04	6.00E-04	6.00E-04	0.001
	kz (m/day)	0.1	1	1	4.50E-04	4.50E-04	4.50E-04	0.001

6. I-95 Boston Embankment Case Study

In This Section, we investigate the I-95 Boston Embankment, a well-known case study. This project is a portion of Interstate Highway I-95 North of Boston, which was constructed in 1965 over a thick layer of Boston Blue Clay (BBC). The MIT-MDPW (Massachusetts Institute of Technology - Massachusetts Department of Public Works) performed a massive study and investigation of this embankment, which was constructed throw the period from 1967 to 1969. The crest width was 28 m with a height of 12.2 m, and the footing was 84 m in width.

The comprehensively collected data and observations are used for evaluating various behavior analyses such as vertical deformations, PWP, stability, and overturning [29-32]. The G.S. is 1.5 m above the mean sea level.

The instrumentation included piezometers, inclinometers, and settlement rods to measure the performance of the underlying 40 m of Boston Blue Clay (BBC) during the staged construction of the 11 m high embankment and subsequent consolidation [31-32] In addition, and an extensive laboratory test program was conducted on the properties of the BBC [33].

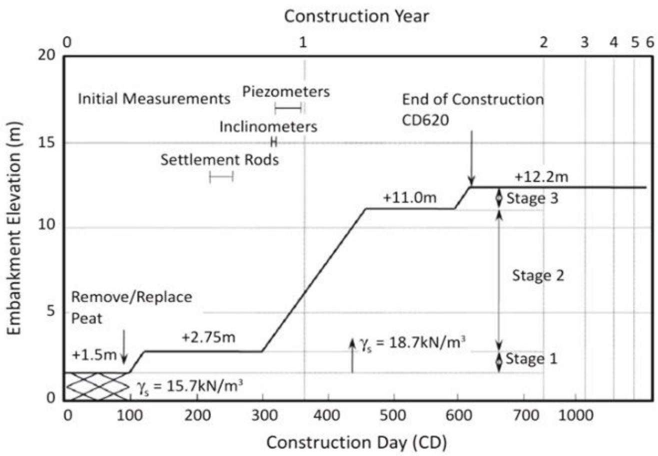


Fig. 14. Construction sequence for I-95 Boston Embankment [31]

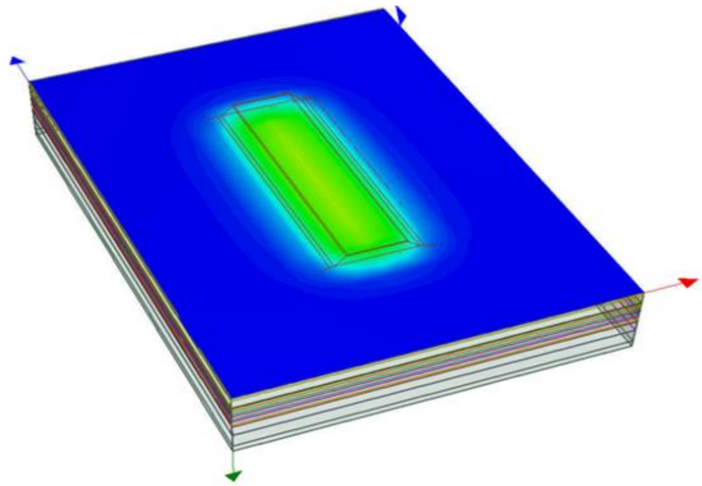


Fig. 15. Settle 3D Model for I-95 Boston Embankment.

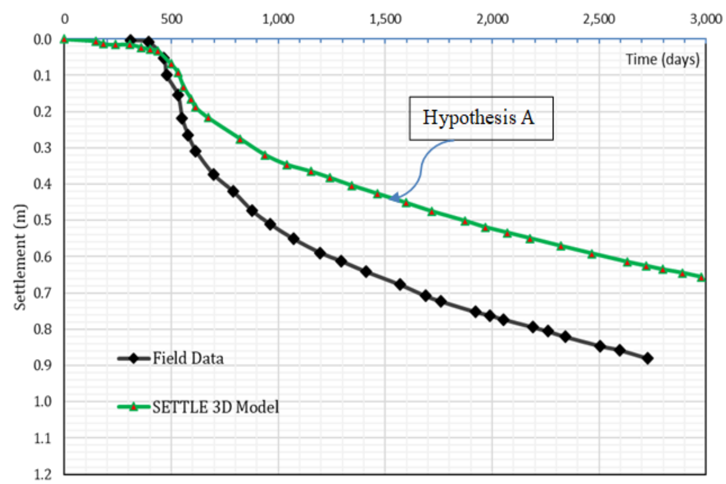


Fig. 16. Total Settlement - time curves using the semi-analytical method of SETTLE 3D for I-95 Boston Embankment.

Settlement and pore water pressure readings were obtained under the embankment at various levels using the previously described instrumentation and FEM results. The readings were recorded periodically along the I-95 Boston Embankment for the construction (620 days) and 5.3 years after construction.

6.1. Hypothesis A (SETTLE 3D Model)

Analytical solutions of the targeted case are performed using Settle3D, which uses Terzaghi consolidation theory. The 3D geometry of the mentioned case study is defined in the model, as shown in Figure 15. The G. W. T. is set to 0.80 m depth, and the stages of construction are defined previously in the phase description of the case. Figure 15 illustrates the geometry of the 3D model used for modeling and analysis of the case study. A nonlinear soil model was used, and the determined parameters are listed in Table 6.1. Figure 16 shows the results of the semi-analytical solution using SETTLE 3D, which represents Hypothesis A.

According to the results from the comparison between the time-settlement curves, we can see that the results from Hypothesis A give an acceptable agreement with field observations of settlement values. The results of the FE model were higher than those of the field measurements tile at the 400-day mark. Then, the results of the numerical model aligned with the field data until the end of the field readings at 2750 days, with lower settlement values compared with the field measurements. At the 2700-day mark, the settlement using Hypothesis A is almost 0.61 m while the field measurement is 0.90 m with relative errors of approximately 35%.

6.2. Hypothesis B (PLAXIS 3D Model)

Analyzing the behavior of soft ground under embankments is a challenging task for geotechnical engineers. As the embankment lies over a large area, modeling the entire embankment is found to be time-consuming without any technical gain. Thus, a portion of the embankment was modeled and used to represent it. Moreover, A sensitivity analysis is performed to choose the appropriate dimensions for the FE model. As shown in Figure 17.

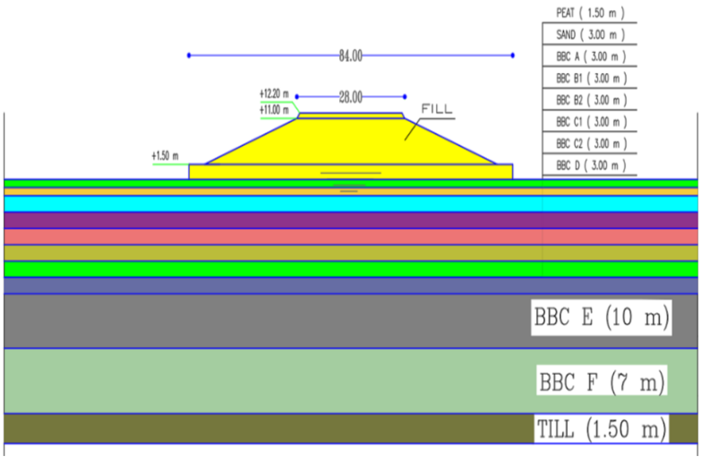


Fig. 17. Geometry and soil conditions of the prototype model based on Boston Blue Clay under the I-95 Boston Embankment.

Embankment fill was from the height of the crest at 12.2 m to the base at 0.00 m, giving a 12.2 m in height. The top and bottom widths of the embankment were 28.0 m at the top and 84.0 m, respectively. The Embankment fill was divided into three sublayers that were constructed in three stages.

The considered boundary conditions for all the vertical boundaries are determined by allowing the applications of vertical settlements while preventing horizontal movement from taking place; furthermore, the drainage of the pore water through all the vertical boundaries is prohibited. However, for the lower horizontal boundary of the model, both the vertical and horizontal movements are prevented, while the drainage of the pore water isn't allowed to take the presence of a till layer underlying the BBC layer into consideration. The 3D model geometry and applied meshing concept are shown in Figure 18.

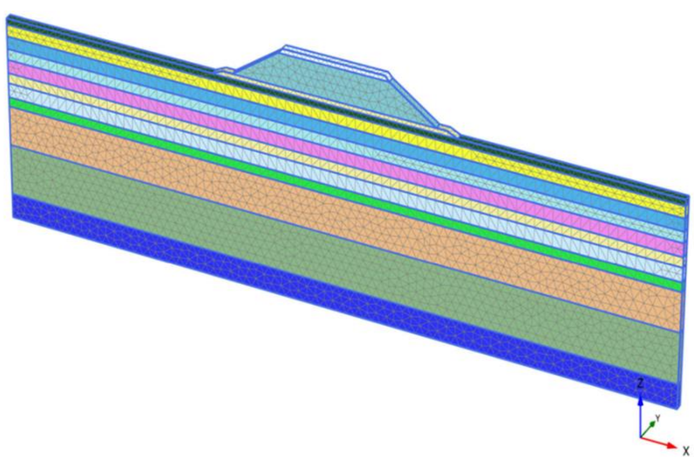


Fig. 18. 3D model for I-95 Boston Embankment using very fine meshing (12132) Elements

Mesh sensitivity tests were performed to ensure that the meshing was dense enough to produce reasonable results; therefore, a very fine mesh was chosen. Additionally, the bottom border is constrained in both directions, while the side boundaries are restrained horizontally. For consolidation analysis, the drainage boundaries at the ground surface and bottom were opened, whereas the lateral borders were closed.

Table 4

Construction phases of I-95 Boston Embankment. [31]

Phase	1	2	3	4	5	6	7
Analysis	CS	CS	C	CS	C	CS	C
Time	45	78	175	163	137	22	Δu = 1 kPa
Cum. Time	45	123	298	461	598	620	-

Hint: CS (Construction) and C (Consolidation)

In this section, we have a comparison between the results of the 3D FEM and the field measurements. The field settlement and the corresponding settlement results of the numerical model are shown in Figure 19.

Generally, the results of the numerical model agreed better with the field measurements. The results of the numerical model were slightly higher than the field measurements until the 400-day mark was reached, which might be due to the slight alteration of the construction sequence used in the numerical modeling from the actual construction sequence. However, the maximum difference is 5.0 cm at 800 days, which corresponds to a 5% difference in the results. After 600 days, the results of the numerical model aligned with the field measurements until the end of the field observations at 7.40 years. A comparison between the calculated settlement and the settlement predicted by the 3D FEM is shown in Figure 20, which indicates excellent agreement.

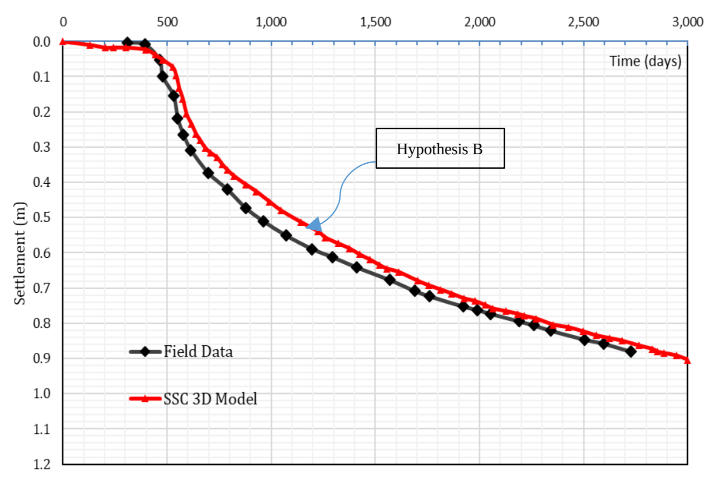


Fig. 19. Comparison between 3D SSC FEM and Field measurements.

6.3. Discussion

The settlement calculated using the PLAXIS SSC 3D model (Hypothesis B) is larger than the settlement calculated using Settle 3D, which represents the analytical solution (Hypothesis A) by an average of 25.8%. In addition, it can be concluded that the settlement values were almost the same in the early stage until 600 days.

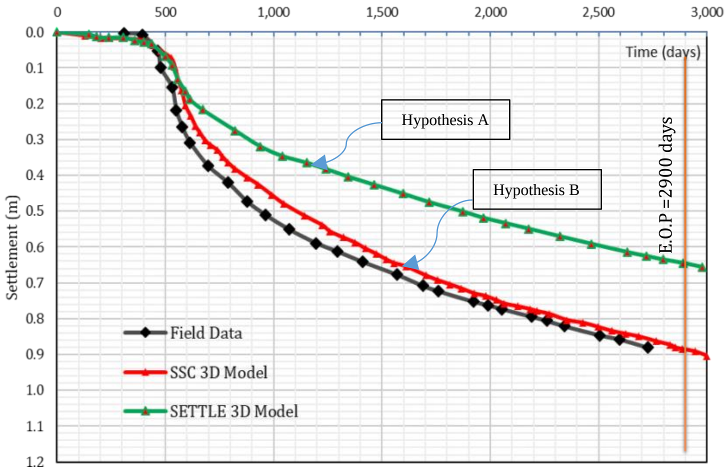


Fig. 20. Comparison between SSC 3D Model, Settle 3D model and Field measurements (Case 2)

7. Conclusion and Recommendations

This research investigates how soft clay consolidates. The main goal is to determine the settlement due to creep using Hypothesis A and Hypothesis B, and then compare the results with laboratory tests and in-situ measurements.

This paper presents a literature review on the consolidation of soft soils. Then, an embankment was constructed on soft ground for the two case studies studied and analyzed by both finite element (Plaxis 3D SSC Model) and semi-analytical methods (Settle 3D), and the results from the two methods were compared with the measured values of settlement of the soft soil layers. The main advantage of the Soft Soil Creep Model (SSC model) is that it mainly depicts the behavior of soft soils while considering secondary compression, that is, creep.

Therefore, it could be summarized the conclusion in the following points:

- The models using Plaxis 3D (SSC model) match better with the field measurements for the two cases.
- The calculated settlement by PLAXIS SSC 3D model (Hypothesis B) was larger than the calculated settlement by Settle 3D (Hypothesis A) by 9.30 % for NHP levee and 25.8 % for I-95 Boston Embankment with an average of 17.5% for the two cases.
- Hypothesis B is more accurate for calculating the consolidation settlements compared with Hypothesis A.

8. Recommendations for Future Study

- Performing a similar study using the application of other soil constitutive models (for example MIT-SR model) to explore more aspects of the soil behavior and creep.
- Calibrate the investigated soil models to the behavior of soft soil clay samples in Egypt, such as East Port Said soil.
- Characterization of the soil profiles and parameters for different strategic locations in Egypt and preparation of a geotechnical map to be used as a guide for future investigations.

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