Numerical Analysis of Staged Construction of an Embankment on Soft Soil

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Abstract

The objective of this study was to predict the excess pore pressure and settlement of an embankment over soft ground, treated with vertical drain, through numerical analysis of staged construction. To carry out finite element analysis, numerical modeling software PLAXIS 3D was used. The practical demonstration was demonstrated by validating two case studies; the first one was a trial embankment at the Krishnapatnam Ultra Mega Power Project in Nellore, Andhra Pradesh, India and the second one was the Second Bangkok International Airport or Suvarnabhumi Airport, about 30 km from the city of Bangkok, Thailand. After the successful validation of the program, detailed finite element modelling of an embankment resting on soft soil was conducted. Moreover, the degree of consolidation and factor of safety were also determined. There was rapid dissipation of excess pore pressure and maximum settlement at the mid-height of the embankment. In contrast, the dissipation of excess pore pressure was very slow just below the embankment and it increased with the increment of the depth of the clay layer. Moreover, with the rise of the distance from the centre of the embankment, the dissipation of the excess pore pressure also raised and took less time, the settlement also increased.

Keywords - Numerical analysis, PLAXIS 3D, staged construction, embankment, soft soil.

Declarations

This type of work have not been done in Bangladesh before. Moreover, the manuscript is original, does not infringe any copyright or other proprietary rights of third parties, is not submitted for publication to another journal, and has not been previously published.

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Conflicts of interest

The authors wish to confirm that there are no known conflicts of interest associated with this publication.

Availability of data and material

Data are available in the Geotechnical Engineering Lab of Bangladesh University of Science and Technology (BUET), Dhaka.

Code availability

PLAXIS 3D.

Authors' contributions

Ayesha Binta Ali: Data curation, Methodology, Validation, Investigation, Writing – Original draft preparation.

Mehedi Ahmed Ansary: Conceptualization, Data curation and Supervision.

1 Introduction

Though Bangladesh is a developing country, industrial development has risen significantly here. The construction of embankments for new roads, highways and railways are increasing these days over soft ground due to the deficiency of suitable land for infrastructure development. Numerical analysis is used to solve this problem by staged construction with a factor of safety (FS). The 1st objectives of the study to verify the PLAXIS 3D software by two case studies. The first one was a trial embankment at the Krishnapatnam Ultra Mega Power Project in Nellore, Andhra Pradesh, India and the second one was the Second Bangkok International Airport or Suvarnabhumi Airport, about 30 km from the city of Bangkok, Thailand. The 2nd objective is to conduct a numerical analysis of staged construction of an embankment using PLAXIS 3D software.

Bergado et al. (2002) studied prefabricated vertical drains (PVDs) in soft Bangkok clay: a case study of the new Bangkok International Airport project in Thailand. In their study, they presented the performance of a full-scale embankment which was constructed on soft clay in Bangkok with PVDs and they showed that the consolidation settlements raised with the corresponding reduction of the spacing of the PVDs at a particular period of time. In the last phase, their full-scale study results had proven the efficacy of PVDs for the improvement of soft Bangkok clay soil. They used Terzaghi's one-dimensional (1D) consolidation theory to determine the 1D primary consolidation settlements. The degree of consolidation of the clay soil below the test embankments was determined both from the dissipation of the pore-pressure and the settlements of the test embankments. According to Bergado et al. (2002), the degree of consolidation could be obtained from the measured pore pressures if the compression ratio was assumed to be constant. Moreover, the degree of consolidation could also be obtained from the measured settlements of the test embankments. The solution to the problem of consolidation of a soil cylinder containing a central sand drain was presented by Barron (1948). Hansbo (1979) recommended some modifications which could deal mainly with simplifying assumptions because of the physical dimensions, characteristics of the PVDs and the effects of PVDs installation.

Soud (2016) studied numerical analysis of prefabricated vertical drains (PVDs) improved soft soil beneath an embankment during staged construction. In his study, the effects of both PVDs and the staged construction were considered simultaneously and the combined advantages resulting from the partial consolidation of soil were investigated. Soud (2016) used Settle 3D as the first attempt to verify the case study of the full-scale embankment on soft Bangkok clay with PVDs at the new Bangkok International Airport project in Thailand. Then he used Settle 3D as the second attempt to investigate the effect of using PVDs in the soft soil improvement of Al-Basra site south of Iraq.

Shen et al. (2005) studied analysis of the field performance of embankments on soft clay deposits with and without PVDs improvement. Two full-scale test embankments which were constructed on soft clay deposits located in the eastern coastal region of China were presented in their study. One of the embankments was constructed on natural subsoil and the other embankment was constructed on PVDs improved subsoil. The thickness of the soft clay deposit with and without PVDs improvement was 23 m and 19 m respectively. The PVDs were installed in a triangular pattern at a depth of 19 m with a spacing of 1.5 m. The finite element method was used to analyse the field performance of the two embankments. For the analysis of the PVDs improved subsoil, a simple approach

was employed using the equivalent vertical hydraulic conductivity (k_{ve}) of PVDs improved subsoil. It was seen from the analytical results that PVDs raised the bulk vertical hydraulic conductivity of soft soil by about 30 times which was compared to the original. According to Shen et al. (2005), in the design of embankments constructed on soft subsoil, so many factors should be considered. One of the important factors is how the time required for consolidation of soft soil can be reduced. Prefabricated vertical drains (PVDs) are frequently used to speed up the consolidation of soft soils (Holtz et al., 2001; Bergado et al., 1996; Chai et al., 1996; Hansbo, 1981; Jamiolkowski et al., 1983; Rixner et al., 1986). With the use of PVDs, the vertical hydraulic conductivity of soft soil can be increased in a macro-sense by more than 1 order of magnitude (Bergado et al., 1996; Hansbo, 1981). According to them, FEM was generally used to analyze the behaviour of PVDs improved soft soil under any embankment loading. There are three approaches for the modelling of the PVDs improved soil. The first method utilizes a one-dimensional (1D) drainage element (Chai et al., 1995; Hird et al., 1992; Hird et al., 1995). The second method select a macro-element in FEM program for the consideration of the drainage behaviour of vertical drains (Sekiguchi et al., 1986; Zeng & Xie, 1987). The third method is basically a simple approximate approach and an equivalent value of vertical hydraulic conductivity (k_{ve}) of PVDs improved soil is estimated (Chai et al., 2001). By using this approach, the behavior of PVDs improved subsoil can be inspect in a similar way to that of non-treated natural subsoil. As the third method is simple and can be directly applied in any situation, it was adopted in that study.

Ali & Gofar (2010) studied settlement of embankment on prefabricated vertical drain treated ground. They presented the differentiations between the field measurements of settlements and the analysis performance by FEM for a case study. The blending of preloading and vertical drain was choosen as a stabilization method for the soft soil. The thickness of soft soil deposit as well as the shear strength and consolidation properties of the soil were collected from the site investigation report. Finite element code PLAXIS version 7.2 was utilized for the modeling of the embankment for numerical analysis. In their study, they used PLAXIS for the utilization of an equivalent vertical permeability of the vertical drain and simulated plain strain condition based on Terzaghi's average degree of vertical consolidation. They used Asaoka method for the comparision of the field measurement data with the deformation as well as rate of settlement of soft clay soil beneath the embankment predicted by the FEM. They found good agreement between the actual measurement and the settlement prediction in terms of time for reaching 90% consolidation when the diametre ratio of two and permeability ratio of six was used in the FEM while the effect of well resistance was insignificant. The study was actually based on the construction project of a 1.2 km road between Chikus and Langkap in Perak, Malaysia.

Ozcoban et al. (2007) studied the staged construction and settlement of Alibey Dam which was founded on soft clay. It was located near Istanbul in Turkey on the Alibey Stream which was 4.5 km north of its point of convergence with an ancient submerged river mouth named Golden Horn. PLAXIS 1998 was used for that analysis as it allows the simulation of time-dependent and nonlinear behavior of soils and the non-hydrostatic and hydrostatic pore pressure development in the soil. The soil was assumed a two-phase material that consisted of soil skeleton and water. Firstly, no drainage was assumed and the immediate deformations and the stress increases were calculated using an elastoplastic plane strain analysis. Firstly, the stress increases, and immediate deformations were computed using an elastoplastic plane strain analysis assuming no drainage. Resulting excess

pore pressures were computed by the software from the volumetric strains where saturated conditions were assumed that was followed by the consolidation stage which was based on Biot's theory providing the resulting consolidation settlements and the dissipation of excess pore pressures.

2 The Validation of FEM Model

The numerical analysis was validated for two case studies described below.

2.1 Case Study 1

The first case study was chosen from a study by Rajesh et al. (2018) which was a trial embankment at the Krishnapatnam Ultra Mega Power Project (KUMPP) in Nellore, Andhra Pradesh, India which was located on a soft soil site. Fig 1 shows the cross section of the embankment at the KUMPP. The summary of material properties used in the model at the KUMPP for verification is presented in the following Table 1. The comparisons between the paper and PLAXIS 3D for both excess pore pressure and settlement without considering drain are showing in the following Fig 2 and 3 respectively.

Fig 1. Cross section of the embankment at the KUMPP.

Table 1. Summary of material properties at the KUMPP (Rajesh et al., 2018).

Parametre	Embankment	Very dense sand	Medium dense sand	Very dense sand	Clay
Unsaturated unit weight, γ_{unsat} (kN/m ³)	17	17	17	17	17
Saturated unit weight, γ_{sat} (kN/m ³)	19	20	20	20	17
Coefficient of permeability in x direction, k_x (m/day)	2	0.0173	0.0173	0.0173	1.6 * 10 ⁻⁰⁵
Coefficient of permeability in y direction, k_y (m/day)	2	0.0043	0.0043	0.0043	4.0 * 10 ⁻⁰⁶
Elastic modulus, E_{ref} (kPa)	30000	40600	42130	40600	1059
Poisson's ratio, ν	0.3	0.25	0.25	0.25	0.35
Cohesion, c (kPa)	1	3	3	3	25
Friction angle, ϕ (°)	30	41.5	36	41.5	0
Dilatancy angle, ψ (°)	0	0	0	0	0

Fig 2. Comparison of excess pore pressures between PLAXIS 3D and the paper by Rajesh et al. (2018).

Fig 3. Comparison of settlements between PLAXIS 3D and the paper by Rajesh et al. (2018).

All the settlements in that study were measured at the centre of the embankment, and excess pore pressures were measured at a distance of 5 m from the centre; both being measured at a depth of 15 m from the ground surface. It is seen from both Fig 3 and 4 that the results found from PLAXIS 3D matched with the results found from the paper by Rajesh et al. (2018).

2.2 Case Study 2

The second case study was chosen from a study by Indraratna et al. (2010) which was the Second Bangkok International Airport or Suvarnabhumi Airport, about 30 km from the city of Bangkok, Thailand. Fig 4 shows the cross section of the embankment at the Second Bangkok International Airport.

Fig 4. Cross section of the embankment at the Second Bangkok International Airport.

The summary of material properties used in the model of the Second Bangkok International Airport for verification is presented in the following Table 2.

Table 2. Material properties used in the model of the Second Bangkok International Airport (Indraratna et al., 2010).

Depth (m)	Compression index, λ	Swelling index, κ	Poisson's ratio, ν	Coefficient of vertical permeability, k_v 10^{-9} m/s	Coefficient of horizontal permeability, k_h 10^{-9} m/s
0.0 – 2.0	0.3	0.03	0.30	15.1	30.1
2.0 – 8.5	0.7	0.08	0.30	6.4	12.7
8.5 – 10.5	0.5	0.05	0.25	3.0	6.0
10.5 – 13.0	0.3	0.03	0.25	1.3	2.6
13.0 – 15.0	1.2	0.10	0.25	0.3	0.6

The comparisons between the case study and PLAXIS 3D for both excess pore pressure and settlement without considering drain are showing in the following Fig 5 and 6 respectively.

Fig 5. Comparison of excess pore pressures between PLAXIS 3D and the paper by Indraratna et al. (2010).

Fig 6. Comparison of settlements between PLAXIS 3D and the paper by Indraratna et al. (2010).

All the settlements and excess pore pressures were measured at the mid-depth of the clay layer. It is seen from both Fig 6 and 7 that the results found from PLAXIS 3D matched with the results found from the study by Indraratna et al. (2010).

3 Methodology of the Study and Data Collection

The study methodology is:

- 1) Soil sample was collected from the selected location of soft soil where the embankment was constructed.
- 2) A series of laboratory tests such as Specific Gravity Test, Sieve Analysis, Hydrometer Analysis, Liquid Limit Test, Plastic Limit Test, One-Dimensional (1-D) Consolidation Test, Triaxial Test were conducted on the collected soil samples to determine the soil Parameters like specific gravity, grain size, void ratio, compression index, swelling index, cohesion, angle of internal friction, permeability according to ASTM standards.
- 3) Then Numerical Analysis was done. The embankment construction was divided into four phases. After the first construction phase, a consolidation period of 30 days was introduced to allow the excess pore pressure to dissipate. After the second construction phase, another consolidation period was introduced from which the final settlements were determined. Hence a total of four calculation phases were defined besides the initial phase.

3.1 Geological Location and Geotechnical Engineering Properties of the Soil

The location of the study area was the Akhaura–Laksam–Chittagong railway line connecting Akhaura and Chittagong, via Laksam in Bangladesh. The total length of the rail line is 321 kilometers and only 102 kilometer of it is double track and non-contiguous. Efforts are on to make the entire length double-track. According to the site investigation report, the site was underlain by alluvial deposits. The soil profile was divided into 3 to 4 sub-layers according to the difference in soil characteristics, physical and mechanical properties. The subsurface consisted of a 6 m to 13.5 m layer of soft soil at various depth and underneath the soft clay, there was thick layer of very dense fine sand.

3.2 Derivation of Soil Parameters from Field Test

Field investigation was performed in the form of SPT at the selected site. Wash boring technique was used for SPT. Disturbed sample was collected, and SPT N-values were recorded at every 1.5 m depth interval up to 13.5 m from the existing ground level.

3.3 Derivation of Soil Parameters from Laboratory Test

Collected sample was tested in the Geotechnical Engineering Laboratory of Bangladesh University of Engineering and Technology (BUET). Index and strength properties were determined to evaluate the sub-soil condition of the study area. The grain size distribution curve found from the particle size analysis by sieve and hydrometer is shown in Fig 7.

Fig 7. Grain size distribution curve.

Summary of the grain size distribution and summary of the index properties of selected cohesive sample are shown in Table 3 and Table 4 respectively.

Table 3. Summary of the grain size distribution of selected cohesive sample.

Chainage (km)	Borehole no./ Sample no.	Depth (m)	G_s	% Finer	Grain Size Distribution		
				no. 200	% Sand	% Silt size	% Clay
				Sieve (0.075	size	(0.005 to	size
				mm)	(>0.075	0.075 mm)	(<0.005
				mm)	mm		mm)
164+600	BH-6	3.0 & 4.5	2.55	98.4	1.6	66.2	32.2
	D2 & D3						

Table 4. Summary of the index properties of selected cohesive sample.

Chainage (km)	Borehole no./ Sample no.	Depth (m)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	USCS Symbol	Shrinkage Limit, SL	Linear Shrinkage, LS
164+600	BH-6	3.0 & 4.5	75	27	48	CH	20	15
D2 & D3								

3.4 One-Dimensional Consolidation Test

One-dimensional consolidation test was carried out on the collected sample. The e - $\log P$ curve of the sample is presented in Fig 8. From loading and unloading portions of the e - $\log P$ curve, the values of the compression index C_c , the swelling index C_s , the modified compression index, λ^* and the modified swelling index, κ^* were determined. These parameters are presented in Table 5.

Fig 8. e - $\log P$ curve.

Table 5. One-dimensional consolidation test results.

Compression index, C_c	Swelling index, C_s	Modified compression index, λ^*	Modified swelling index, κ^*
0.98	0.19	0.184	0.037

The coefficient of vertical permeabilities for various stress ranges were computed using the following equations:

$$k = c_v m_v \gamma_w \quad (1)$$

$$c_v = (T_{90} * H_{dr}^2) / t_{90} \quad (2)$$

$$m_v = a_v / (1 + e_{av}) \quad (3)$$

$$a_v = \Delta e / \Delta P \quad (4)$$

Where, k = Coefficient of permeability, c_v = Coefficient of consolidation, T_{90} = Time factor for 90% consolidation = 0.848, H_{dr} = Thickness of soil layer, m_v = Coefficient of volume compressibility, e_{av} = Average void ratio, a_v = Coefficient of compressibility, and γ_w = Unit weight of water = 62.4 lb/ft³.

The values of the coefficient of vertical permeabilities are shown in Table 6. The coefficient of permeability $k = 5.2 * 10^{-7}$ ft/min = $2 * 10^{-4}$ m/day for stress of 100 kPa for 90% consolidation was selected for the modelling.

3.5 Triaxial Compression Test

Consolidated Drained Triaxial Compression Test was conducted for the study. The Mohr Circles and Mohr-Coulomb Failure Envelope are shown in Fig 9. The effective cohesion, c' and the effective angle of internal friction, ϕ' were determined from the Mohr-Coulomb Failure Envelope. The values of effective cohesion, $c' = 14$ kN/m² and effective angle of internal friction, $\phi' = 17^\circ$.

Table 6. The values of the coefficient of permeabilities.

Void ratio, e	Δe	Pre-ssure, P (kPa)	ΔP	Co-efficient of compressibility, a_v	Ave-rage void ratio, e_{av}	Co-efficient of volume compressibility, m_v (ft ² /lb)	Co-efficient of consoli- dation, c_v for t_{90} (ft ² /min)	Co-effecient of permeability, k for t_{90} (ft/min)
1.31		10						
1.3	0.01	25	15	0.0067	1.305	$1.38 * 10^{-5}$	$4.4 * 10^{-4}$	$3.8 * 10^{-7}$
1.26	0.04	50	25	0.0016	1.28	$3.36 * 10^{-5}$	$3.7 * 10^{-4}$	$7.9 * 10^{-7}$
1.19	0.07	100	50	0.0014	1.225	$3.01 * 10^{-5}$	$3.5 * 10^{-4}$	$6.6 * 10^{-7}$
1.1	0.09	200	100	0.009	1.145	$2.01 * 10^{-5}$	$4.2 * 10^{-4}$	$5.2 * 10^{-7}$
0.96	0.14	400	200	0.007	1.03	$1.65 * 10^{-5}$	$4.3 * 10^{-4}$	$4.4 * 10^{-7}$
0.72	0.24	800	400	0.006	0.84	$1.56 * 10^{-5}$	$3.4 * 10^{-4}$	$3.3 * 10^{-7}$

Fig 9. Mohr circles and mohr-coulomb failure envelope.

4 Numerical Modelling and Results

PLAXIS 3D was chosen for the research study because it is mainly developed for numerical analysis, and for its popularity amongst practicing engineers.

4.1 Geometry of the Embankment

The embankment was 7 metres wide. The slope had an inclination of 1:2. The embankment was symmetric, so only half of the embankment was modelled (in this study the right half was chosen). A representative section of 2 metres was considered in the study. Fig 10 shows the cross-section of the road embankment and soil layers and Fig 12 shows the 3D view of the road embankment and soil layers.

According to the site investigation report, the study area consisted of mainly coastal deposits. The groundwater table which might vary with the season was found to be at 1 metre below the ground surface at the time of boring. The subsoil consisted of 13.5 metres of soft soil. Under the soft soil layer, there was a dense sand layer of which 4 metres were considered in the model. The embankment itself was composed of loose sandy soil.

Fig 10. Cross-section of the embankment and soil layers.

Fig 11. 3D view of the embankment and soil layers.

4.2 Soil Parameters

The soil layers comprising the embankment foundation was defined using a borehole. Table 7 shows the material properties of the road embankment and subsoil.

Table 7. The material properties of the road embankment and sub-soil.

Parameter	Embankment	Sand	Clay	Unit
Material model	Hardening Soil (<i>HS</i>)	Hardening Soil (<i>HS</i>)	Soft soil	-
Drainage type	Drained	Drained	Undrained A	-
Soil unit weight above phreatic level, γ_{unsat}	16	17	15	kN/m ³
Soil unit weight below phreatic level, γ_{sat}	19	20	18	kN/m ³
Initial void ratio, e_{init}	0.5	0.5	1.321	-
Secant stiffness in standard drained triaxial test, E_{50}^{ref}	$2.5 * 10^4$	$3.5 * 10^4$	-	kN/m ²
Tangent stiffness for primary oedometer loading, E_{oed}^{ref}	$2.5 * 10^4$	$3.5 * 10^4$	-	kN/m ²
Unloading / Reloading stiffness, E_{ur}^{ref}	$7.5 * 10^4$	$1.05 * 10^5$	-	kN/m ²
Power for stress level Dependency of stiffness, m	0.5	0.5	-	-
Modified compression index, λ^*	-	-	0.184	-
Modified swelling index, κ^*	-	-	0.037	-
Cohesion, c^*	1.0	0.0	14.0	kN/m ²
Friction angle, ϕ'	30.0	33.0	17.0	degree (°)
Dilatancy angle, ψ'	0.0	0.0	0.0	degree (°)
Data set	USDA	USDA	USDA	-
Soil type	Loamy sand	Sand	Clay	-
Horizontal permeability (x direction), k_x	3.499	7.128	$2 * 10^{-4}$	m/day
Horizontal permeability (y direction), k_y	3.499	7.128	$2 * 10^{-4}$	m/day
Vertical permeability, k_v	3.499	7.128	$2 * 10^{-4}$	m/day
Strength reduction factor, R_{int}	1.0	1.0	1.0	-
Over consolidation ratio, OCR	1.0	1.0	1.0	-
Pre overburden pressure, POP	0.0	0.0	0.0	kN/m ²

4.3 Vertical Sand Drains

Vertical Sand Drains were arranged in a square pattern. The distance between two consecutive drains is 2 m in a row or column. Only one row of drains was considered in this study. The model geometry of the embankment with vertical drain is shown in the following Fig 12.

Fig 12. Model geometry of the embankment with vertical drain.

4.4 Influence of Mesh Size

In this study, three types of meshing namely coarse mesh, medium mesh, and fine mesh were used to show the influence of mesh size. 4 (four) nodes were selected for the computation of results. Table 8 shows the 4 nodes that were selected for the computations of the results. Fig 13 shows the cross-section of the road embankment and soil layers with the selected 4 nodes. The computed results of the pore pressures and settlements for the selected 4 nodes using various types of mesh size (Coarse, Medium, and Fine) are shown in the following Fig 14 and Fig 15 respectively.

Table 8. Selected 4 nodes for the computation of the results.

Node	Coordinate	Description
A	(1, 0, 2.5)	Mid height of the embankment
B	(1, 0, -1)	Just below the embankment
C	(1, 0, -7)	Mid depth of the clay layer
D	(13.5, 0, -1)	At the toe of the embankment

Fig 13. Cross-Section of the road embankment and soil layers with the selected 4 nodes.

(a)

(b)

(c)

(d)

Fig 14. Influence of mesh size on excess pore pressure profiles, (a) Node A, (b) Node B, (c) Node C and (d) Node D.

(a)

(b)

(c)

(d)

Fig 15. Influence of mesh size on settlement profiles, (a) Node A, (b) Node B, (c) Node C and (d) Node D.

It is observed from the above-plotted graphs that the output results were close to each other. So, it can be said that the variation of different types of meshing in the output results was quite insignificant. It is also noticeable that the finer the meshing system, the more time was required for the computation. As the differences between different kinds of meshing were of little importance in this study, medium meshing was selected for the staged construction.

4.5 Staged Construction

In the initial phase, the embankment was not present. Therefore, the corresponding volumes were deactivated in the initial phase. As the embankment was required to construct in a short time, the time required for the construction of the embankment in every stage must such that it was just enough to maintain the stability up to that height of the embankment for the target factor of safety (FS). This is actually required to maintain the stability of the increased height in the following stage. Again, some short of time is required to be allowed for a certain percentage of consolidation.

4.6 Boundary Conditions

As horizontal flow should not occur, the left vertical boundary (X min) must kept closed because of symmetry. The bottom was open because the excess pore pressure could freely flow into the deep as well as permeable sand layer. The upper boundary was obviously open as well. The view of the groundwater flow subtree after the definition is given in Table 9.

Table 9. Boundary conditions for groundwater flow.

Location	Boundary Limits	X_{min}	X_{max}	Y_{min}	Y_{max}	Z_{min}	Z_{max}
Dense sand	$X = 0$ to 60 m $Y = 0$ to 2 m $Z = -17.5$ m to -13.5 m	Closed	Open	Closed	Closed	Open	Open
Soft clay	$X = 0$ to 60 m $Y = 0$ to 2 m $Z = -13.5$ m to 0	Closed	Open	Closed	Closed	Open	Open
Embankment (1 st layer)	$X = 0$ to 27 m $Y = 0$ to 2 m $Z = 0$ to 2.5 m	Closed	Open	Closed	Closed	Open	Open
Embankment (2 nd layer)	$X = 0$ to 27 m $Y = 0$ to 2 m $Z = 2.5$ m to 5 m	Closed	Open	Closed	Closed	Open	Open

4.7 Consolidation Analysis

The embankment construction was divided into four phases. Firstly, the first construction phase was conducted. Then a consolidation period of 30 days was introduced and the excess pore pressure was allowed to dissipate. After that the second construction phase was conducted and then another consolidation period was introduced from which the final settlements were determined. Hence a total of four calculation phases were introduced along with the initial phase.

4.8 Variation of Excess Pore Pressures

The excess pore pressure profiles for every 4 nodes are shown in Fig 16.

Fig 16. Excess pore pressure profiles for the selected 4 nodes together.

It is found from Fig 16 that, the maximum excess pore pressure was found at node B, which was just below the embankment. The value was about 75 kN/m² at the last construction phase, and the dissipation of excess pore pressure was very slow with time at that node. The minimum excess pore pressure was found at node A, which was at the mid-height of the embankment. The dissipation of excess pore pressure was very fast and reached to the value of zero rapidly with time. That means, the dissipation of excess pore pressure was rapid at the mid-height of the embankment. On the other hand, the dissipation of excess pore pressure was very slow just below the embankment and it raised with the rise of the depth of the clay layer.

4.9 Displacement Increments

The settlement profiles for every 4 nodes are shown in Fig 17.

Fig 17. Settlement profiles for the selected 4 nodes together.

It is found from Fig 17 that, as the dissipation of excess pore pressure was very fast at node A, which was at the mid-height of the embankment, the settlements were higher at that nodes. The settlement was about -1.35 metres in 200 days at node A. On the other hand, the settlement was lower at node D (about -0.2 metres in 200 days), which was at the toe of the embankment. That means, the settlement was higher at the mid-height of the embankment. On the contrary, the settlement reduced with the rise of depth of the clay layer.

4.10 Degree of Consilodation

The degree of consolidation was calculated using the following formula:

$$\text{Degree of consolidation, } U = 1 - u_z / u_o \quad (5)$$

Where, u_z = excess pore pressure at time t and u_o = initial excess pore pressure. The degree of consolidation at every phases is shown in the following Fig 18.

Fig 18. Degree of consolidation at every phases.

4.11 Safety Analysis

Considering the stability during construction is as important as considering the final stability in the design of an embankment. It is evident from the output results that after the second construction phase, a failure mechanism initiated to develop (Fig 19).

Fig 19. Failure mechanism develop after the second construction phase.

The evaluation of safety factor is shown in Fig 20 and It is seen from the curves that a constant value of about 1.38 was obtained.

Fig 20. Evaluation of safety factor.

5 Conclusions

Based on the present study, the following conclusions are drawn:

- The software was validated by comparing the field data of the case studies with the software analysis.
- The results found from the two case studies and software analysis were in a reasonable agreement to each other.
- There was rapid dissipation of excess pore pressure and maximum settlement at the mid-height of the embankment.
- In contrast, the dissipation of excess pore pressure was very slow just below the embankment and it increased with the increment of the depth of the clay layer.
- Moreover, with the rise of the distance from the centre of the embankment, the dissipation of the excess pore pressure also raised and took less time, the settlement also increased.

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List of Symbols

The following symbols are used in this paper:

γ_{unsat} = Soil unit weight above phreatic level
 γ_{sat} = Soil unit weight below phreatic level
 γ_w = Unit weight of water
 k = Coefficient of permeability/Hydraulic conductivity
 k_x = Horizontal hydraulic conductivity (in x direction)
 k_y = Vertical hydraulic conductivity (in y direction)
 k_h = Horizontal hydraulic conductivity
 k_v = Vertical hydraulic conductivity
 k_s = Smear zone hydraulic conductivity
 ν = Poisson's ratio
 c = Cohesion
 c' = Effective cohesion
 ϕ = Angle of internal friction
 ϕ' = Effective angle of internal friction
 ψ = Dilatancy angle
 ψ' = Effective dilatancy angle
 λ = Compression index
 κ = Swelling index
 λ^* = Modified compression index
 κ^* = Modified swelling index
 G_s = Specific Gravity
 LL = Liquid Limit
 PL = Plastic Limit
 PI = Plasticity Index
 SL = Shrinkage Limit,
 LS = Linear Shrinkage
 C_c = Compression index
 C_s = Swelling index
 c_v = Coefficient of consolidation
 T_{90} = Time factor for 90% consolidation
 H_{dr} = Thickness of soil layer
 m_v = Coefficient of volume compressibility
 e = Void ratio
 e_{av} = Average void ratio
 e_{init} = Initial void ratio
 a_v = Coefficient of compressibility
 E_{50}^{ref} = Secant stiffness in standard drained triaxial test
 E_{oed}^{ref} = Tangent stiffness for primary oedometer loading
 E_{ur}^{ref} = Unloading / Reloading stiffness
 m = Power for stress level dependency of stiffness

R_{int} = strength reduction factor
 OCR = Over consolidation ratio
 POP = Pre overburden pressure
 U = Degree of consolidation
 FS = Factor of safety

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Figures

Figure 1

Cross section of the embankment at the KUMPP.

Figure 2

Comparison of excess pore pressures between PLAXIS 3D and the paper by Rajesh et al. (2018).

Figure 3

Comparison of settlements between PLAXIS 3D and the paper by Rajesh et al. (2018).

Figure 4

Cross section of the embankment at the Second Bangkok International Airport.

Figure 5

Comparison of excess pore pressures between PLAXIS 3D and the paper by Indraratna et al. (2010).

Figure 6

Comparison of settlements between PLAXIS 3D and the paper by Indraratna et al. (2010).

Figure 7

Grain size distribution curve.

Figure 8

e-logP curve.

Figure 9

Mohr circles and mohr-coulomb failure envelope.

Figure 10

Cross-section of the embankment and soil layers.

Figure 11

3D view of the embankment and soil layers.

Figure 12

Model geometry of the embankment with vertical drain.

Figure 13

Cross-Section of the road embankment and soil layers with the selected 4 nodes.

(a)

(b)

(c)

(d)

Figure 14

Influence of mesh size on excess pore pressure profiles, (a) Node A, (b) Node B, (c) Node C and (d) Node D.

(a)

(b)

(c)

(d)

Figure 15

Influence of mesh size on settlement profiles, (a) Node A, (b) Node B, (c) Node C and (d) Node D.

Figure 16

Excess pore pressure profiles for the selected 4 nodes together.

Figure 17

Settlement profiles for the selected 4 nodes together.

Figure 18

Degree of consolidation at every phases.

Figure 19

Failure mechanism develop after the second construction phase.

Figure 20

Evaluation of safety factor.