A CASE STUDY ON FINITE ELEMENT SIMULATION OF THE STAGE CONSTRUCTION METHOD

Piash Saha*[1] and Md. Rokonuzzaman[2]

[1]Lecturer, Dhaka International University, Bangladesh, e-mail: piashbddotnet@gmail.com
[2]Professor, Khulna University of Engineering & Technology, Bangladesh, e-mail: rokon@ce.kuet.ac.bd

***Corresponding Author**

ABSTRACT

This study is based on the field data of approach road on the Mawa-side of the Padma multipurpose bridge project which focuses on a group of points. Firstly, it validates the Finite Element Model (FEM) data by comparing it to field settlement data. Further, it estimates settlement with preloading and preloading with vertical drains with respect to time numerically. Lastly, its emphasis on selecting the preferable soil model. Preloading is one of the most cost-effective methods for soft soil stabilization. The results using PLAXIS 2D software showed that the excess pore water pressure for the case was significantly higher and gradually dissipated without vertical drains compared to the case with vertical drains. The explanation is that vertical drains reduce the excess pore pressure effectively and help accelerate its dissipation during the time of consolidation. In addition, the level of settlement with vertical drains during consolidation is higher than without vertical drains used in impermeable soil layers due to low hydraulic conductivity. These values are closer to the field measurements. Several different soil models of varying types were used when modeling the sample embankments; Mohr-Coulomb, Hardening soil, and Soft soil respectively. Because the study includes Mohr-Coulomb's first experience was the most desirable, and it provides fairly realistic results compared to field measurements. Based on the analysis, preloading without vertical drains can be said to be an economical method to avoid post-construction settlements in the Mawa-side project. If the time is highly constrained, a feasible method can be vertical drains with preloading.

Keywords: *Finite element method, Consolidation settlement, Soil model, With and without drains, Preloading.*

1. INTRODUCTION

Many Several techniques for improvement have been developed to match local soil conditions, often consolidation-based soft clay methods. Preloading with vertical drains is an effective ground improvement strategy, requiring ground surface loading to induce most of the underlying soft formation's ultimate settlement (B Indraratna, Rujikiatkamjorn, & Xueyu, 2012) Typically, an additional or surcharge load equal to or greater than the anticipated loading of the base is used to facilitate consolidation with the help of vertical drains (Iyathurai, 2005). Using vacuum pressure can reduce the amount of surcharge filling material required to obtain the same consolidation settlement because it produces suction, which increases the effective stress and accelerates consolidation (Buddhima Indraratna, Rujikiatkamjorn, Balasubramaniam, & Wijeyakulasuriya, 2015). Historically, designing foundation structures or keeping a high mass on soft soils (like clay) has created civil engineering problems. It may take many years simply to overload as a process of soil consolidation. Infrastructure projects are rapidly built on marginal soils worldwide due to rapid growth and urbanization (Abramson, Lee & Boyce, 2002).

One of the most successful and widely used methods is to increase the bearing capacity of these soils and reduce the excess pore water pressure in combination with preloading. After the Second World War, the use of sand drain or vertical drain has undergone enormous development, largely due to better installation methods and increased knowledge of the control factors (Parsa-pajouh, 2014). The key benefits of vertical drains are: (i) increasing the shear strength of the soil by decreasing the ratio of void and moisture content; (ii) decreasing the preloading time necessary to decrease the same rate of post-construction settlements; (iii) decreasing the differential settlement during primary consolidation;and (iv) curtailing the height of surcharge fill required to achieve desired pre-compression (Samson & Rochelle, 1972). It is difficult to quantify the immediate settlement as it is often dependent on the rate of construction of the embankment. For the development intent, the immediate settlement can be assumed to be in the range of 10–20 percent of the embankment's primary settlement. Consolidation settlement prediction can be based on a fully coupled numerical method as shown by Hsi and Small (Lim, 2003).

This approach measures the deformation of the soil and the dissipation concurrently of excess pore water pressure during the construction of the embankment (Fatahi, Minh Le, Quang Le, & Khabbaz, 2013). The reliability of the finite element system has been tested in several implementations on the surface, requiring integration by contrasting numerical results with field measurements (Connolly, Giannopoulos, & Forde, 2013).

1.1 Description of the study area

The Padma Bridge is a multipurpose road-rail bridge across the Bangladesh-based Padma River. It is our country's most daunting bridge project to date. This is a bridge with two tiers of steel truss. The bridge combines a 4-lane highway with another level rail bridge. The length of the bridge is about 6,15 km. On either side of the Padma River, there are two access paths. One is Janjira and the other is an access street known as Mawa. The path to janjira is bigger. The width is approximately 10.5 km. Mawa approach road is approximately 1.6 km long. The purpose of the study is a geotechnical analysis of the Mawa side approach route.

1.2 Description of the project

For the Mawa portion of the Padma Multipurpose Bridge Project, construction of an embankment with a side slope of 3:1 and a crest length of 9.5 m and a height of 6 to 3 m began in 2014. The embankment is situated in a valley with base sediments consisting of a 6.5-meter thick soft silty-clay layer for the investigated section of the road, finished within 600 days by phased construction. In the preliminary planning studies, Staged construction on the existing silty-clay layer was chosen for economic reasons. In this study, for comparison with quantitative results, only the data from settlement plates are used (Tavenas, Chapeau, La Rochelle & Roy, 1974).

2. METHODOLOGY

2.1 Problem, geometry and material properties

The following table 1 gives a brief idea about the structure, status, and content of the parts replicated in the PLAXIS. It also displays the parameters of the material along with the material model used in the analysis. The model of the mohr-coulomb is used for simplicity.

Table 1: Material Model and Properties

Parameters	Relevant Test	Foundation Soil (Clay)			Fill
		MC	SS	HS	MC
Type		Undrained	Undrained	Undrained	Drained
Cohesion, c	CD TEST	1.00 kN/m^2	1.00 kN/m^2	1.00 kN/m^2	1 kN/m^2
The angle of friction, Ø	CD TEST	35.00°	35.00°	35.00°	30.00 °
γ_{sat} or, γ_{unsat} (kN/m^3)	CD TEST	14	14	14	19
K_y=K_x (m/day)	CD TEST	0.01	0.01	0.01	-
Power, m	CD TEST	-	-	0.50	-
E_{50}^{ref}	CD TEST	-	-	5927.00 kN/m^2	-
$E_{ur}^{ref} = 3 * E_{50}^{ref}$ Unloading elasticity	CD TEST	-	-	17724.00 kN/m^2	-
Poisson ratio, v	CD TEST	0.330	-	-	-
Young modulus, E^{ref}	CD TEST	4000 kN/m^2	-	-	-
G^{ref}	CD TEST	1503.759 kN/m^2	-	-	-
E_{50}^{ref} or, E_{oed}^{ref} Loading elasticity	1D CONSOLIDATION TEST	2000 kN/m^2	-	-	20000 kN/m^2
Coefficient of compressibility, c_c	1D CONSOLIDATION TEST	-	0.21	-	-
Void ratio, e	1D CONSOLIDATION TEST	-	0.9106	-	-
λ^* (lambda)	1D CONSOLIDATION TEST	-	$\frac{c_c}{2.3\ (1+e)}$=0.048	-	-
k^* (kappa)	1D CONSOLIDATION TEST	-	$\frac{2*c_r}{2.3\ (1+e)}$=0.0183	-	-

2.2 Construction Phases

Following the user manual of PLAXIS (Version 8.6) the calculation of the embankment stage were done. The process consists of the construction phase in which the embankment load is applied incrementally over time and the dissipation phase in which a time interval is introduced to allow the excess pore water pressure to dissipate. The restructuring steps are shown in the figure 1 below.

Identification	Phase no.	Start from	Calculation	Loading input	Time	Water	First
Initial phase	0	0	N/A	N/A	0.00 ...	0	0
Gravity	6	0	Plastic analysis	Total multipliers	0.00 ...	0	1
Stage1	1	6	Consolidation analysis	Staged construction	13.0...	0	5
Dissipation1	5	1	Consolidation analysis	Staged construction	135....	0	10
Stage2	7	5	Consolidation analysis	Staged construction	16.0...	0	16
Dissipation2	8	7	Consolidation analysis	Staged construction	72.0...	0	19
Stage3	9	8	Consolidation analysis	Staged construction	20.0...	0	24
Dissipation3	10	9	Consolidation analysis	Staged construction	1.00 ...	0	27
Stage4	11	10	Consolidation analysis	Staged construction	27.0...	0	28
Dissipation4	12	11	Consolidation analysis	Staged construction	243....	0	31
ZeroEPWP	13	12	Consolidation analysis	Minimum pore pressure	205....	0	37
HFL	2	13	Plastic analysis	Staged construction	0.00 ...	2	43
Safetyafterconst...	3	12	Phi/c reduction	Incremental multipliers	0.00 ...	0	62
SafetyLongTerm	14	13	Phi/c reduction	Incremental multipliers	0.00 ...	0	162
SafetyHFL	4	2	Phi/c reduction	Incremental multipliers	0.00 ...	2	262

Figure 1: Calculation Steps for soil Section

3. ILLUSTRATIONS

3.1 Settlement result with and without drain

The figure 2 below illustrates the settlement-time curves of an embankment used on silty clay soil with and without vertical drains obtained through numerical analysis with PLAXIS 2D. Both curves are very close to each other and at the beginning show large displacements. It can be seen that most of the time both curves have nearly linear behavior. Using drain, it was shown in Figure 2 below that it takes around 538 days for the ultimate 288 mm settlement. Likewise, it requires about 581 days without using the drain for ultimate settlement. The ultimate settlement is established at an early stage with the aid of the sand drain. The purpose of using sand drain is thus fulfilled.

Nevertheless, as sand drains are, it depends on the user's economy. The immediate settlements, in the figure 3, come from the soil's elastic behavior. The second phase of settlement in the figure is primary consolidation settlements, which could result from volume shifts in the clay due to the gradual dissipation of excess pore water pressure by triggering the first part of the embankment. Finally, the last settlement could be attributable to the dissipation of excess pore water pressures and soil plastic modification by external load increases.

Table 2 below indicates the average vertical deformation in the middle of the embankment and the total excess pore water pressure after the time measurement.

Table 2: Results from the PLAXIS 2D analysis, with and without drains

Results	**Deformation**	**Time**	**Excess pore pressure**
Units	mm	Days	kN/m^2
With drain	288	538	0.1
Without drain	307	581	0.1

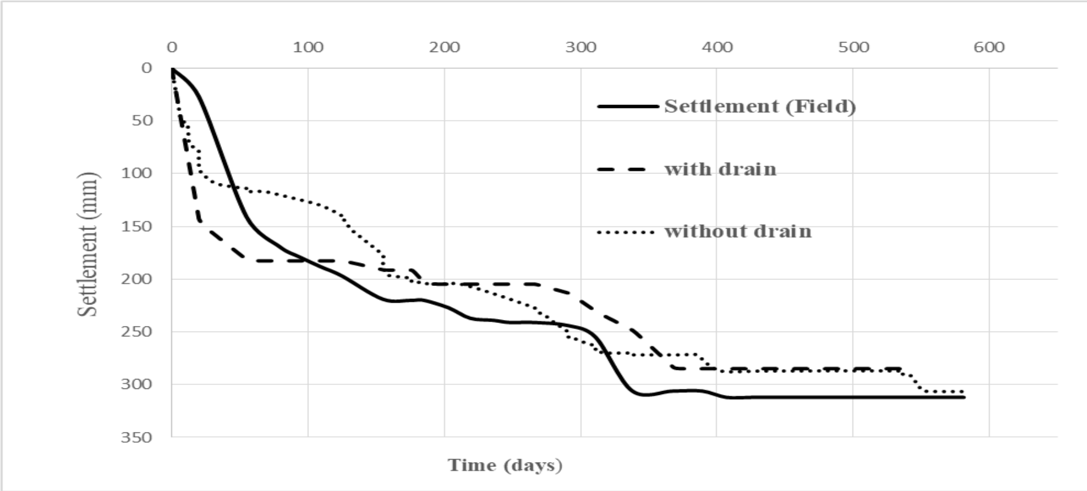

Figure 2: comparison of settlement vs. time curve of FEM data with field data

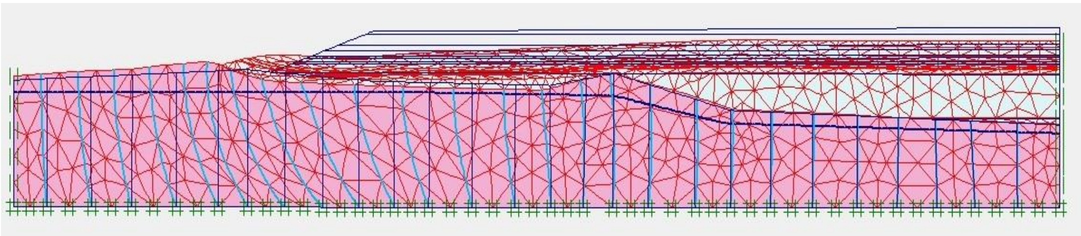

Figure 3: Deformed mesh generated using sand drain

3.2 Comparison of material model

This figure 4 indicates that the prototype from Mohr-Coulomb is superior to the other two. The Mohr-Coulomb model suits the field data much better than the soft soil in measurements at 0 m depth below the embankment. For deeper layers, the differences between these models are becoming less.

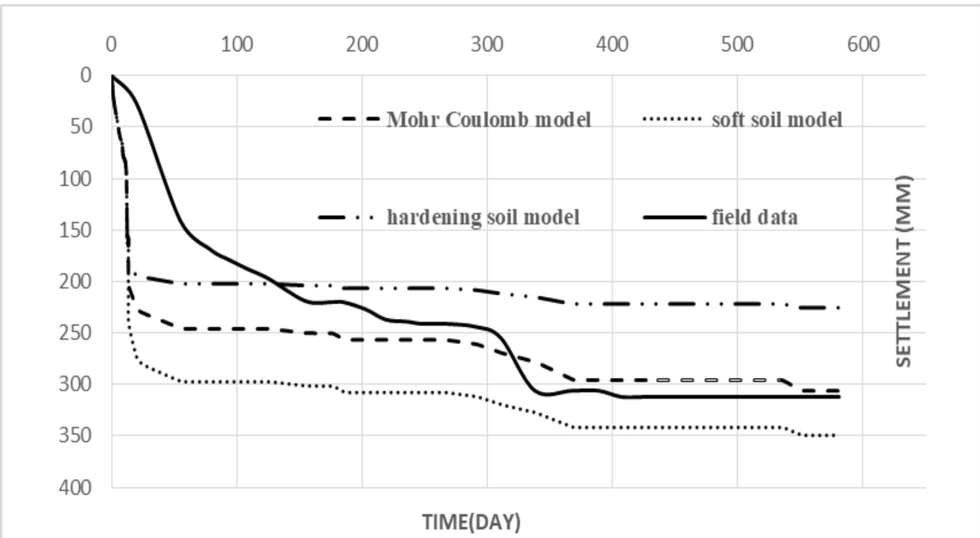

Figure 4: comparison of the time-settlement curve for different soil models with field data

3.3 Permeability Check

In the soil section, a detailed parametric analysis was carried out. The effect of variance has been worked out by comparing the horizontal and vertical permeability of each of the product models used in the system shown in figure 5. Settlement analysis shows that the Mohr-Coulomb soil model showed little settlement data result due to varying permeability in the undrained state of foundation soil. When permeability k_x=k_y= 0.001 m/day then it settled down to 306.19mm, for k_x=k_y= 0.01 m/day settlement was 306.04 mm and for k_x=k_y= 1.0 m/day was 306.18 mm.

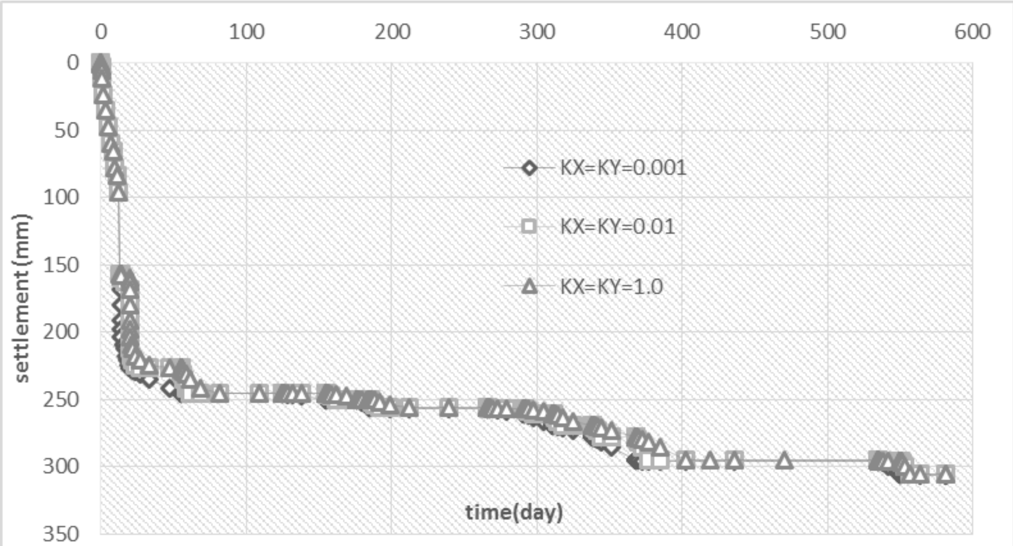

Figure 5: Time-settlement curve with isotropic permeability for the MC model

4. CONCLUSIONS

The main objective of this work was to show the validity of finite element program PLAXIS 2D compared with the field settlement data; with and without vertical drains and to find the best soil material model suitable for the foundation soil. The scope of this research was limited to analyze the model and predict the deformation by using the Mohr-Coulomb, hardening soil and the soft soil model in the numerical analysis.

i. The results obtained from PLAXIS 2D are adequate in comparison to field observations. In both situations with and without vertical drains, the results were much similar for the total displacements but they differ for excess pore water pressures.
ii. Out of the three material models, soft soil model data is most suitable for the project. As the Mohr-Coulomb model matches much better with the field data than the Soft soil and hardening soil model does.
iii. We have also analyzed the change of permeability with three material models. It is seen that the change of permeability didn't show any significant change in the settlement with isotropic permeability.
iv. But, in the case of anisotropic permeability, the settlement rate was found higher compared to isotropic one.
v. We recommend the method of preloading the soil sample without vertical drains to stabilize the ground because it is an economical method and we don't need special equipment at the site.

ACKNOWLEDGMENTS

I convey my gratitude to Professor Dr. Md. Rokonuzzaman, my thesis supervisor who has encouraged me thoroughly during my thesis project with his precious explanations, patience on all occasions and providing relentless technical support. Without my supervisor, this project would not have been completed successfully.

REFERENCES

Abramson, L. E. E. W., Lee, T. S., & Boyce, G. M. (2002). *Slope stability and Stabilisation Methods*.703.

Connolly, D., Giannopoulos, A., & Forde, M. C. (2013). Numerical modelling of ground borne vibrations from high speed rail lines on embankments. *Soil Dynamics and Earthquake Engineering*, *46*, 13–19. https://doi.org/10.1016/j.soildyn.2012.12.003

Fatahi, B., Minh Le, T., Quang Le, M., & Khabbaz, H. (2013). Soil creep effects on ground lateral deformation and pore water pressure under embankments Soil creep effects on ground lateral deformation and pore water pressure under embankments. *Geomechanics and Geoengineering Geomechanics and Geoengineering: An International Journal*, *8*(2), 107–124. https://doi.org/10.1080/17486025.2012.727037

Indraratna, B, Rujikiatkamjorn, C., & Xueyu, G. (2012). Performance and prediction of surcharge and vacuum consolidation via prefabricated vertical drains with special reference to highways , railways and ports. *ISSMGE - TC211 International Symposyum on Ground Improvement*, II-145-II–168.

Indraratna, Buddhima, Rujikiatkamjorn, C., Balasubramaniam, A. S., & Wijeyakulasuriya, V. (2015). Predictions and Observations of Soft Clay Foundations Stabilized with Geosynthetic Drains and Vacuum Surcharge. *Ground Improvement Case Histories: Embankments with Special Reference to Consolidation and Other Physical Methods*, 209–204. https://doi.org/10.1016/B978-0-08-100192-9.00007-7

Iyathurai, S. (2005). Modelling of vertical drains with smear installed in soft clay. *PhD*, 305.

Lim, G. (2003). *Stabilisation of an excavation by an embedded improved soil layer*. 219. Retrieved from https://scholarbank.nus.edu/handle/10635/13635

Parsa-pajouh, A. (2014). *Analysing Ground Deformation Data to Predict Characteristics of Smear Zone Induced by Vertical Drain Installation for Soft Soil Improvement A thesis in fulfilment of the requirement for the award of the degree Doctor of Philosophy from Ali Parsa-Pajouh* , . (February).

Samson, L., & Rochelle, P. La. (1972). Design and Performance of an Expressway Constructed OverPeat by Preloading. *Canadian Geotechnical Journal*, *9*(4), 447–466. https://doi.org/10.1139/t72-044

Tavenas, F. A., Chapeau, C., La Rochelle, P., & Roy, M. (1974). Immediate Settlements of Three Test Embankments on Champlain Clay. *Canadian Geotechnical Journal*, *11*(1), 109–141. https://doi.org/10.1139/t74-008