J. Civil Eng. Mater.App. 2023 (June); 7(2): 95-108

Journal of Civil Engineering and Materials Application
Journal home page: http://jcema.com

Received: 30 July 2022 • Revised: 23 February 2023 • Accepted: 03 May 2023

Research

doi: 10.22034/jcema.2023.354101.1092

The necessity of the consideration of permeability modifier in simulations of clay treatment systems incorporating PVDs and surcharge

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ABSTRACT

Three trial embankments as TS1, TS2, and TS3 that were built for the investigation of a soil treatment project in Bangkok were modeled and verified based on the reported data. To clarify the importance of integration of the hydraulic modifier function vs stress, in the verified models, the modifier functions were omitted and the FEM models were run in the absence of the function. It was shown that after the omission of the hydraulic modifier, the results were overestimated especially for the TS1 and TS2, which had smaller PVDs (prefabricated vertical drains) distance. For the TS1 embankment, the settlement increased from 0.78 m to 0.87 m in 210 days. In 365 days, the settlement increased from 1.27 m to 1.44 m. For the TS2 embankment, the settlement increased from 0.93 m to 1.67 m in 230 days. In 410 days, the settlement increased from 1.36 m to 2.27 m. For TS3 embankment, the settlement increased from 1.15 m to 1.79 m in 230 days. In 410 days, the settlement increased from 1.52 m to 2.24 m. The inclusion of the hydraulic function that calibrates the model for every step of loading is essential in the modelling such problems. For the design phase, this function should be calculated from lab tests, preferably undisturbed samples that were bored from the site, and the resultant function be used as an inseparable part of modeling and calculations.

Keywords: PVD, consolidation, soil treatment, hydraulic modifier, surcharge

1. INTRODUCTION

One of the main challenges in the simulation of weak clay soil treatment systems in the design phase, during the construction phase, and also even for assessment after the compilation of the project, is attaining a model that can predict the settlement, pore pressure, and lateral displacement of the project correctly. 2 uncertainties that relate to such projects:

The uncertainties related to clay include anisotropy [1-3], the presence of organic matter [4], the existence of sand lenses [3, 5], irregular stratification of the soil[6] and, so on. The uncertainties related to modification of the soil characteristics like, the installation of PVDs, the placement of surcharge embankment, the smear zone [7, 8], applying vacuum pressure [9, 10], and so on. The items stated in (1) may seem ordinary like other projects, but since in the systems that involve the placement of surcharge embankment and installation of PVDs, these items are more important. Many failures that have been reported are the result of not accounting for such measures properly, even in the cases where extensive preliminary geotechnical investigations were carried out [11-13]. The uncertainties in this category, are mostly very challenging in the determination of PVDs pattern, PVDs penetration depth, the required embankment height, and ascertaining the required vacuum pressure if needed. Regarding the items in (2), in many projects, an extensive supplementary soil investigation and laboratory test are carried out to predict the process of consolidation as real as possible [14, 15]. Even after such extensive tests and investigations, many obstacles arise during the construction of the project.

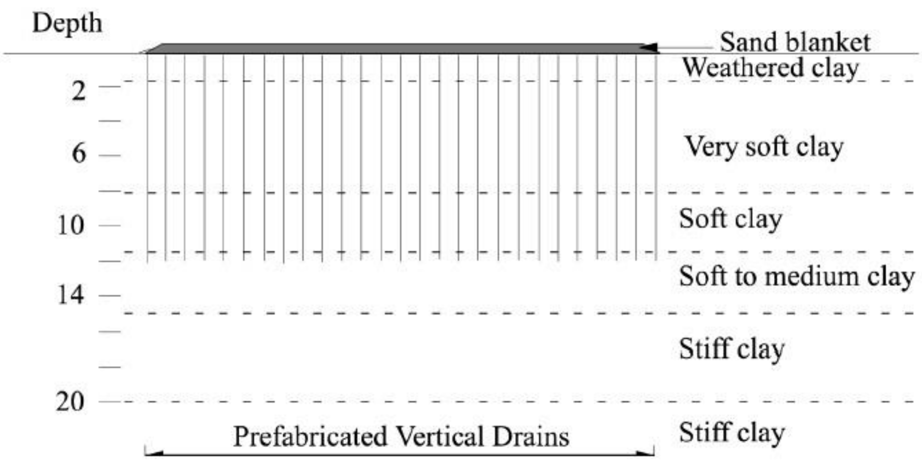
The change in hydraulic conductivity due to the installation of PVDs is one of the uncertainties that affect the modeling and assessment phase. This issue has been addressed by many researchers for a long time[16-18]. By calibration of the variation in horizontal conductivity in analytical and laboratory tests, they tried to model the consolidation process in a way that matches the measurements that were carried out in projects [19] and lab [7, 19] by various instrumentations that were installed. Although the results that were reported were satisfactory, most of the time they were done after the compilation of the project and were reported as case histories. In common practice, a reliable means needed for the design process which is the inclusion of the hydraulic modifier function vs overburden stress. This function can be easily obtained in the process of odometer and triaxial tests for different overburden pressure in the lab. In this paper first, a case study is presented and the results are shown to demonstrate the application of the hydraulic modifier function in described case study.

2. MATERIALS AND METHODS

The Bangkok Airport is situated in a wet area where there is about 10 m of soft clay under a 2 m surficial over-consolidated crust [20]. Stiff clay extending to a depth of 20 to 24 m underlies the soft clay. For analysis purposes, the subsoil is divided into three layers as shown in Figure 1 and the lower stiff clay is ignored [21].

Three pilot embankments were constructed to investigate the various PVDs characteristic and installation patterns at the site in Bangkok [22, 23]. The PVD drains were installed to a depth of 12 m. The embankments were constructed to a height of 4.2 m with 3H: 1V side slopes. The base areas were approximately 40 x 40 m. There were 1 m high berms around the base extending out 5 m that were included in the FEM analysis presented here. Three trial embankments were built as TS1 with 1.5 m PVD spacing, TS2 with 1.2 m PVD spacing, and TS3 with 1 m PVD spacing to examine the performance of the system in various situations. A one-meter-thick sand blanket was placed on the site as a construction working pad. The drains were installed on top of the sand pad. The sand blanket was included to ensure that there would be no build-up of excess pore pressure at the bottom of the embankments and to drain away water being discharged out of the clay as a result of PVDs. The sand blanket is included in the model as a boundary condition. The effect of the sand can be modeled by specifying a zero-pressure boundary condition along the ground surface. The physical implication is that there will be no build-up of positive pore pressure at the ground surface. Any water discharge at the ground surface, would disappear through the sand in the FEM model. The Modified Cam-Clay constitutive relationship is used here for the soft clay as in most of the literature it was used since it gives the best results for such complex coupled analyses [19, 24]. The clay is essentially normal to slightly over-consolidated. It appears that the degree of over-consolidation varies somewhat with depth. In this analysis, the Lambda and Kappa values were taken to be the same for the very soft and the lower soft clay. This gives results closer to what was measured and avoids unnecessary sophistication. The weathered surficial clay is over-consolidated and consequently, it is acceptable to treat this layer as behaving in a linear-elastic manner. Using a linear-elastic constitutive relationship also helps with maintaining numerical convergence near the ground surface where the stresses approach zero. The sand fill is also modeled as a soft linear-elastic material and the soil parameters are set as being total-stress parameters. This avoids having to deal with pore-pressures build up in the embankment. These simplifying assumptions are acceptable because we are primarily interested in using the fill as a means to apply the surcharge preloading here and don't want to investigate the stability or other issues related to unsaturated consolidation in the embankment body. The actual stress-strain response of the sand is not of significant importance [20].

Figure 1. the schematic view of the soil treatment area layers [19]



2.1. Hydraulic conductivity

The most important parameter in an analysis like this is hydraulic conductivity (coefficient of permeability). By the very nature of the deposition process, the conductivity can vary significantly. In addition, the stratification tends to make the conductivity somewhat higher in the horizontal direction than in the vertical direction is shown as Figure 1 [25]. Furthermore, the insertion of the drains disturbs the soil around the drain and alters the conductivity. The disturbed zone is often called a smear zone [26]. Also, drains are installed on some kind of pattern, and spacing and flow to the drains are two-dimensional in plain view. Analyses however are generally more conveniently carried out in a 2D section. Indraratna and Redana [27] have done extensive studies on how to adjust conductivities for a 2D plane-strain analysis, how to assess the smear zone thickness and conductivity and how to model the size of the drain itself. The details are in the paper reference cited at the end. A brief summary is presented here to show how these effects can be accounted for. The flow is predominately horizontal to the drains and consequently most of the discussion centers around horizontal conductivity. The vertical conductivity can be a ratio of the horizontal conductivity but this is not all that important since there is very little or no vertical flow that can be a practical and realistic assumption for FEM modeling.

The equivalent thickness of a drain for a 2D analysis can be taken as [28]:

$$d_w = \frac{2\ a+b}{\pi} \qquad (1)$$

Where a is the thickness of the PVD drain and b is the width. So for a typical drain that is 4 mm thick and 100 mm wide, the 2D model thickness can be 66 mm or say 0.06 m.

The simplest form of converting from an axisymmetric to plane-strain conductivity is [20]:

$$\frac{K_{hp}}{K_h} = \frac{0.67}{\left[\ln\ n\ -0.75\right]} \qquad (2)$$

Where n is the ratio of the drain spacing D to the equivalent drain thickness dw. If the drain spacing is 1.5 m and the equivalent drain thickness is 0.06, then n is 25. The plane-strain conductivity then is about 27 percent of the corresponding axisymmetric horizontal conductivity. This is in the absence of any well resistance and any effect of a smear zone. As an easy figure to remember, the plane-strain conductivity is about a quarter of the corresponding axisymmetric conductivity.

[29] Suggest that the radius of a smear zone around a drain will typically be five (5) times the equivalent radius of the mandrel. For a mandrel that is 45 mm thick and 125 mm wide the equivalent radius is about 55 mm. The radius of the smear zone then is 270 mm (0.27 m). For a 2D analysis, the smear zone thickness would then be 0.54 m. Indraratna and Redana [29] have presented an equation to estimate the conductivity of the smear zone which involves various dimensional ratios and conductivity ratios. All the details would not be discussed here and are available in the [19, 26] paper for those interested. As a broad rule, the horizontal smear zone conductivity is about 10 percent of the horizontal plane-strain conductivity. For the cases presented by [29], the ratio varies between 8 to 16 percent. To be Stated in another way, the disturbance resulting from the insertion of the drain reduces the conductivity by about an order of magnitude in the smear zone.

For a layered system like this we can compute an equivalent conductivity as follows [29]:

$$K = \frac{d}{\left(\frac{d_1}{k_1} + \frac{d_2}{k_2}\right)} \qquad (3)$$

Say d1 is 1 m, d2 is 1 m, k1 is 10 m/sec and k2 is 1 m/sec. The blended equivalent K then is 1.818 m/sec.

Table 1. The blended conductivities used in the model for drains with spacing (a) TS1, 1.5 meters (b) TS2, 1.2 meters (c) TS3, 1 meter

(a)

The number of the layers	$k_{horizontal}$ (m/day)	$K_{plane-strain}$ (m/day)	K_{SMEAR} (m/day)	$K_{blended}$ (m/day)
1	4.52e-3	1.27e-3	1.02e-4	**2.48e-4**
2	1.04e-3	2.9e-4	2.33e-5	**5.67e-5**
3	4.54e-4	1.28e-4	1.03e-5	**2.5e-5**
4	4.54e-4	1.28e-4	1.03e-5	**2.5e-5**
5	4.54e-4	1.28e-4	1.03e-5	**2.5e-5**

(b)

The number of the layers	$k_{horizontal}$ (m/day)	$K_{plane-strain}$ (m/day)	K_{SMEAR} (m/day)	$K_{blended}$ (m/day)
1	4.52e-3	1.25e-3	1.02e-4	**2e-4**
2	1.04e-3	2.87e-4	2.33e-5	**4.56e-5**

3	4.54e-4	1.25e-4	1.03e-5	**2.01e-5**
4	4.54e-4	1.25e-4	1.03e-5	**2.01e-5**
5	4.54e-4	1.25e-4	1.03e-5	**2.01e-5**

(c)

The number of the layers	$k_{horizontal}$ (m/day)	$K_{plane\text{-}strain}$ (m/day)	K_{SMEAR} (m/day)	$K_{blended}$ (m/day)
1	4.52e-3	1.35e-3	1.02e-4	**1.67e-4**
2	1.04e-3	3.1e-3	2.33e-5	**3.83e-5**
3	4.54e-4	1.35e-4	1.03e-5	**1.7e-5**
4	4.54e-4	1.35e-4	1.03e-5	**1.7e-5**
5	4.54e-4	1.35e-4	1.03e-5	**1.7e-5**

We can use this information to represent the conductivity of the native clay together with the smear zone rather than create separate geometric regions for the two different zones. This makes the process of the numerical modeling easier [25].

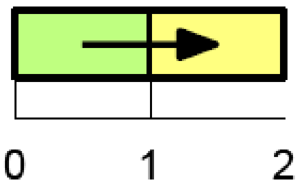


Figure 2. one element conductivity [25]

The implication in analysis here is that the smear zone around the drain dominates the dissipation of the excess pore pressures and in turn the rate of consolidation.

To modeling the conductivities Indraratna and Redana [29] presented a table of conductivities used in their analyses. For a 1.5 m drain spacing the conductivities are as in the following table. The blended K is computed based on drain spacing of 1.5 m (radius = 0.75 m) and a smear zone thickness of 0.54 m (radius = 0.27 m). The conductivities have been converted to m/day so that the time sequencing can be in days in the Geostudio suite. Soft clay at depth has a conductivity of about one order of magnitude less than the upper weathered clay [25].

The conductivity of soft soils can change significantly as the soil compresses and the void ratio decreases. Geostudio 2018, used in this paper has a mechanism whereby the conductivity can be adjusted as the effective stress increases in response to the dissipation of the excess pore pressure. This is an indirect way of adjusting the conductivity resulting from a decrease in the void ratio. One way to define data for such an adjustment in conductivity is from the results of an odometer test. The conductivity can be computed for each load increment in an odometer test as follows [25]:

The average effective stress and change in void ratio for each load increment can be determined from the test results. The coefficient of compressibility is [30]:

$$a_v = \frac{e_1 - e_2}{\sigma_2' - \sigma_1'} \quad (4)$$

The average vertical effective stress is:

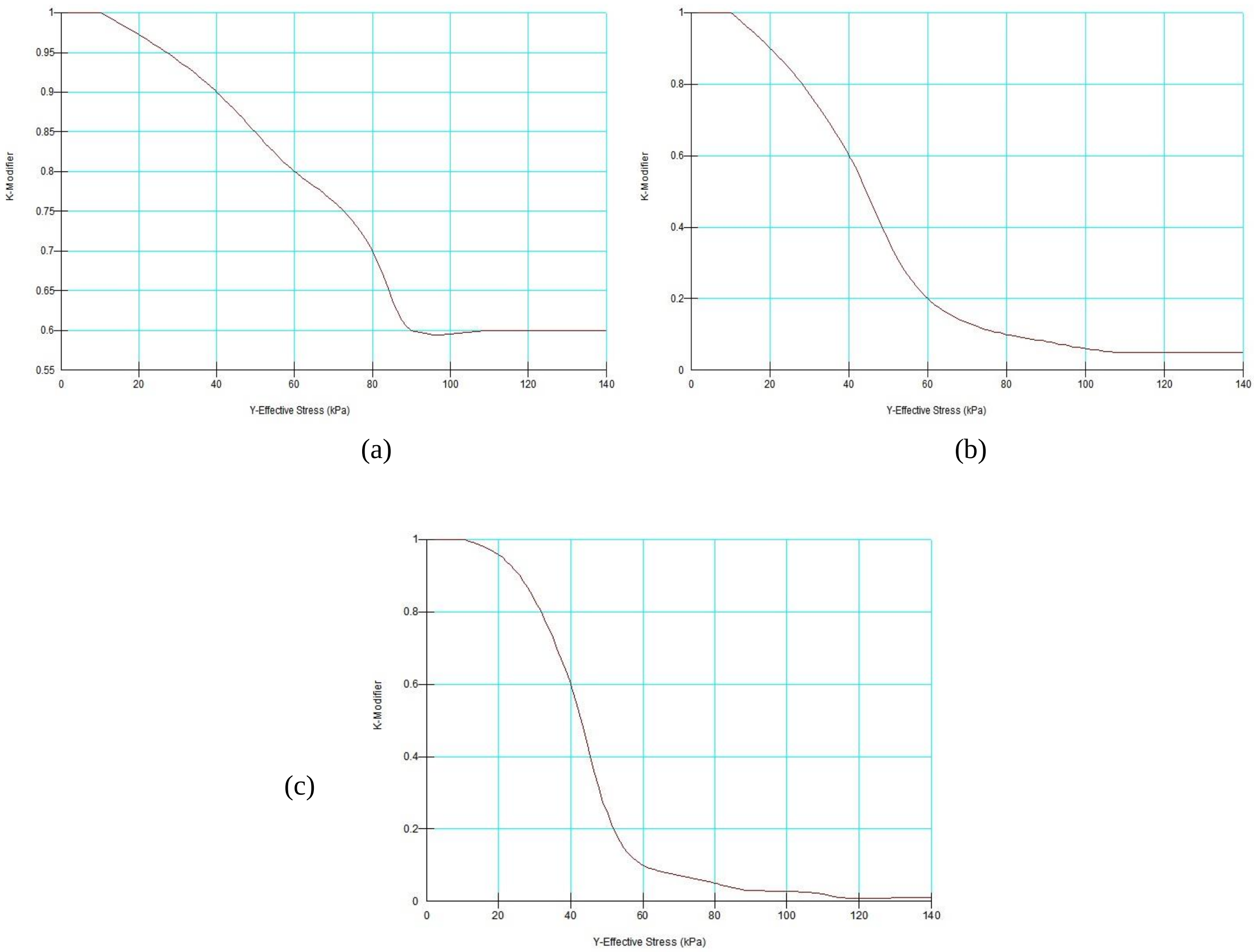
$$\sigma_{ave}' = \frac{\sigma'_2 + \sigma_1'}{2} \quad (5)$$

For each load increment, a void ratio versus time plot is available which can be used to determine the coefficient of consolidation Cv. Once these values are known the conductivity can be computed from:

$$K = \frac{C_v \gamma_w a_v}{1 + e_o} \tag{6}$$

In this way, K can be determined for the average vertical effective stress for each load increment.

The commercial Geostudio 2019 FEM code coupled Stress/Pwp analysis was used in this paper. Complementary information on modeling procedure and site soil characteristics can be found in [22, 24, 31]



(c)

Figure 3. The hydraulic modifier used for FEM modeling of layers for (a) hard clay (b) soft clay (c) very soft clay

As stated by [32] to consider the nonlinearity of the consolidation arising from evolving permeability and compressibility of the soil due to change in void ratio during consolidation and non Darcian flow regime for low permeability soil and large strain elastoplastic behavior of the soil, a permeability modifier was applied in FEM analyses [33].

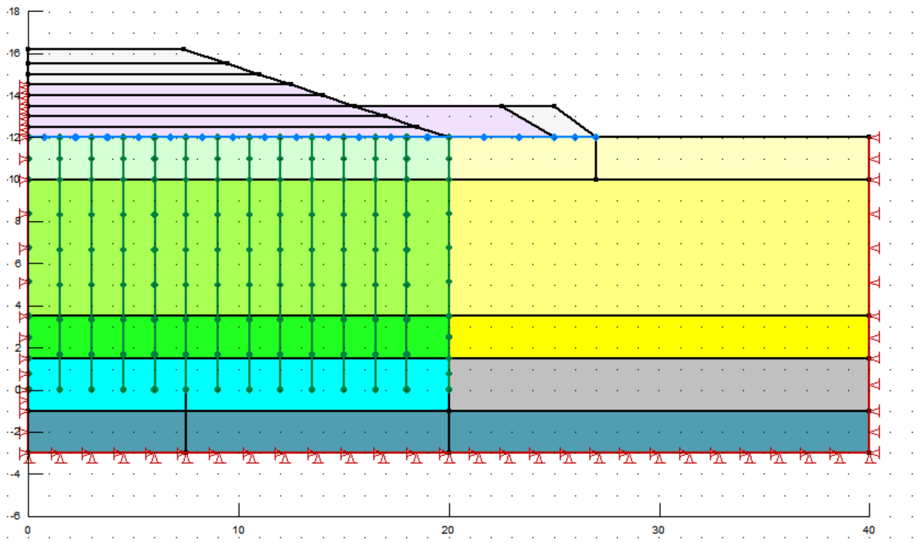


Figure 4. The FEM model and the sequential embankment loading

Figure 4 shows the schematic view of the FEM model and the sequential loading that was done in the project based on data from [34] to reflect the real stress that was applied to the project. Even the berm that was constructed for stability issues was modeled in the program base on the time that it was built.

Since in simulations of alike conditions, modified cam clay has to be proven the reasonable results, it was also used in this study [19, 24, 35, 36]. The modified cam clay parameters were set as stated in [19].

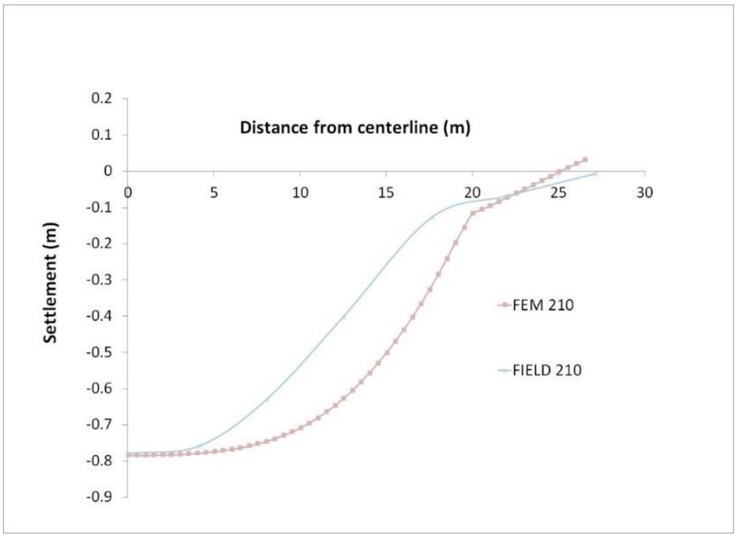
3. RESULTS AND DISCUSSION

3.1. Verification of the model's based on the measured settlement at project site

Figures 5, 6, and 7 show the verification of the models used in this study after 210 and 365 days at the centerline of the embankments. TS1 had 1.5 m PVDs spacing, TS2 had 1.2 m PVDs spacing and TS3 had 1-meter PVDs spacing. As can be seen there is a good agreement between measured and calculated settlements for the settlements. Regardless of the ultimate settlement, in 210 periods, the calculated curve is somehow overestimated, while for 365 periods, the curve matches well with the field data. The existence of sand lenses and malfunction in instrumentation at the site might be part of the reason for this issue as also reported by [22]. Another possible reason for the difference in settlement curves from field data is the delay in the consolidation of clay layers due to the degradation of clay microscopic structures that has been addressed by [37, 38]. As the distance between PVDs decreases, the settlement increase as it is illustrated in the settlement curves.

At the toe of the embankments, there is a heave in the soil that also has been reported by previous studies [39, 40] and is one of the problems in soil treatment systems incorporating surcharge preloading and PVDs. As can be seen the model has the ability of the prediction for the heave. In situations where there are urban or sensitive facilities, permanent roads or highways in the vicinity of the project, or underground urban infrastructures and piping, the resultant heave can make damage severely and consideration about this matter should be taken prior to the start of the construction. One of the ways to overcome the heave issue is the application of vacuum pressure [24, 35] since, and inward force is created by vacuum pressure that lessens the lateral displacement and heave under and in the vicinity of the embankments.

(a)



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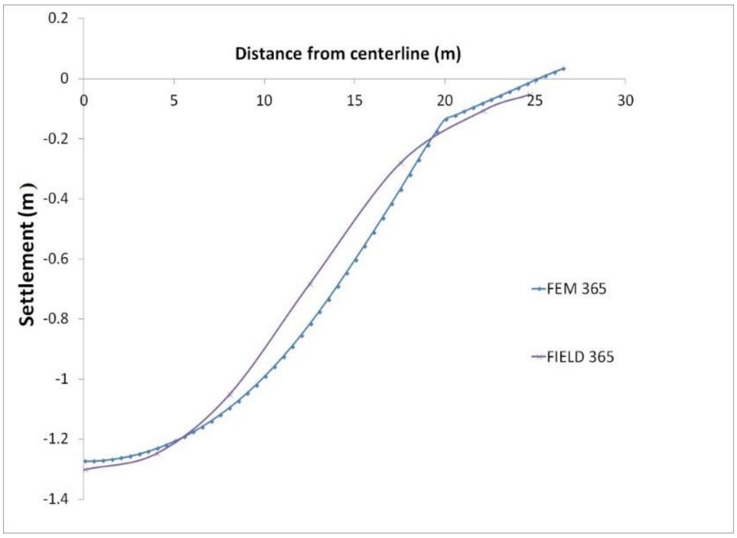
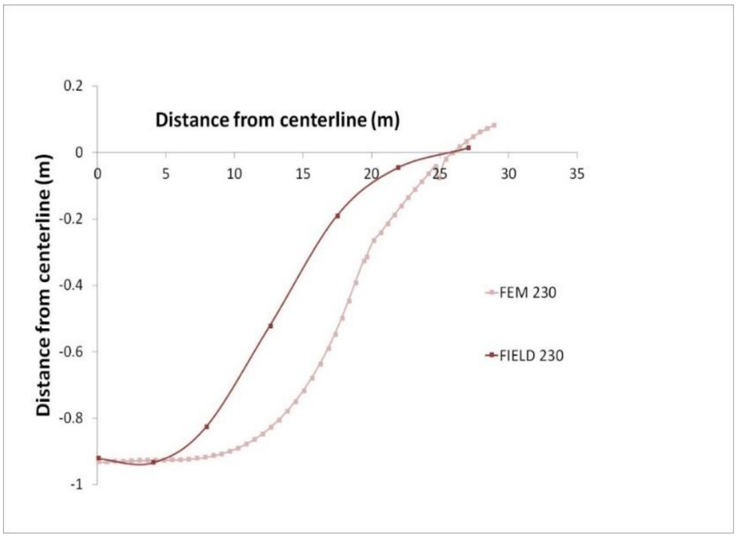


Figure 5. The FEM results of TS1 settlement vs measured data [22] (a) after 210 days (b) after 365 days at the centerline of the embankment

(a)



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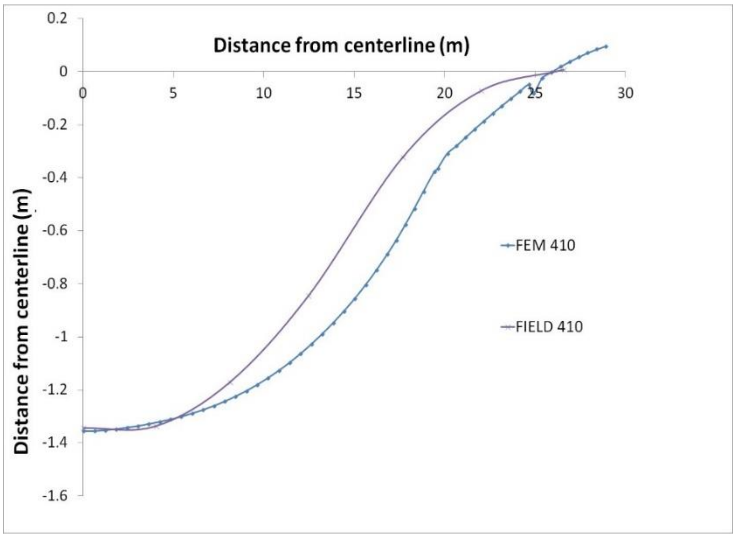
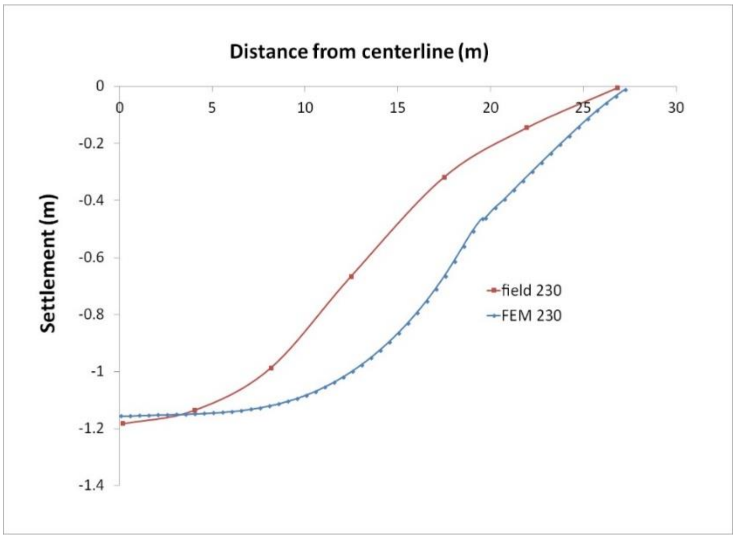


Figure 6. The FEM results of TS2 settlement vs measured data [22] (a) after 230 days (b) after 410days at the centerline of the embankment

(a)



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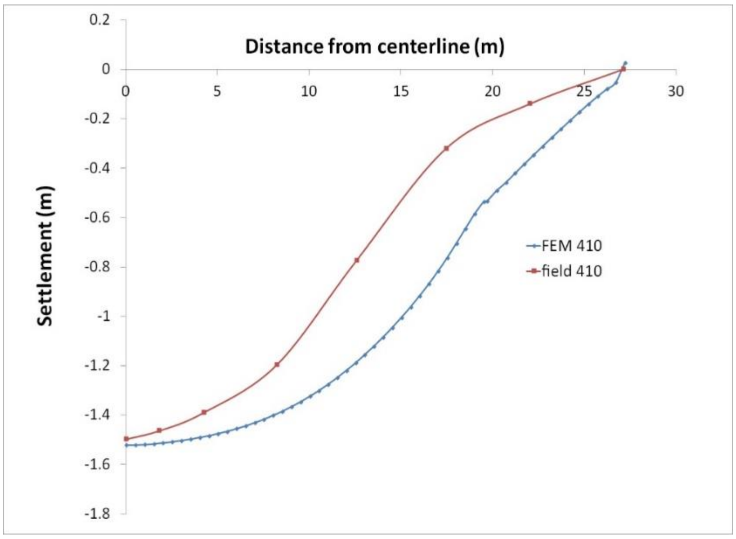


Figure 7. The FEM results of TS3 settlement vs measured data [22] (a) after 230 days (b) after 410 days at the centerline of the embankment

3.2. The omission of the hydraulic modifier function from the verified model

To investigate the effect of the hydraulic modifier function, the function was omitted from the verified models. The resultant settlement curves are shown in Figure 8. As can be seen, all the settlement results are overestimated and as the distance between the PVDs increased, the quantity of the overestimation increase dramatically. For the TS1 embankment, the settlement increased from 0.78 m to 0.87 m in 210 days. In 365 days, the settlement increased from 1.27 m to 1.44 m. For the TS2 embankment, the settlement increased from 0.93 m to 1.67 m in 230 days. In 410 days, the settlement increased from 1.36 m to 2.27 m. For the TS3 embankment, the settlement increased from 1.15 m to 1.79 m in 230 days. In 410 days, the settlement increased from 1.52 m to 2.24 m.

As the consolidation occurs, the clay structure becomes stiffer and the void ratio decreases. As the process continues for higher step loading, the clay undergoes lesser volume deformation. Concerning the definition of m_v in formula 2, the coefficient of permeability decreases like the permeability in the process of the consolidation but as can be seen in Formula 1 , the reduction of m_v, leads to the increase in c_v.

$$C_v = {}^{k}/_{\Upsilon_w} . m_v \tag{7}$$

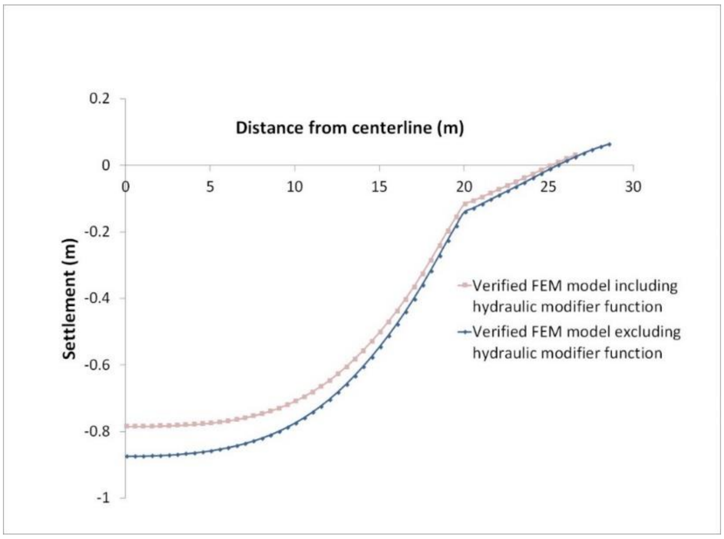
Where C_v is coefficient of the consolidation , m_v is the coefficient of compressibility, Υ_w is the unit weight of the water and k is the permeability of the soil.

$$m_v = \frac{1}{1+e_1} \frac{e_1 - e_2}{p_2 - p_1} \tag{8}$$

Where $m_v$the coefficient of compressibility in one step of loading, e is the void ratio of the soil and p is the applied stress.

By the omission of the hydraulic conductivity, since m_v in the modified cam clay model is a constant integer and an input parameter in most of the FEM programs, the consolidation process speeds up unrealistically and the resultant settlements, as a result, are far from reality. The inclusion of the hydraulic functions that calibrate the model for every step of loading is essential in modeling such problems. For the design phase, this function should be calculated from lab tests (preferably undisturbed samples) that were bored from the site and the resultant function be used as an inseparable part, of the modeling and calculations. Based on the experience of the authors, the results obtained from the Lefranc Permeability test method, are highly overestimated and can be just used for a preliminary soil investigation reports and as a result, a suitable test shall be carried out in the lab.

(a)



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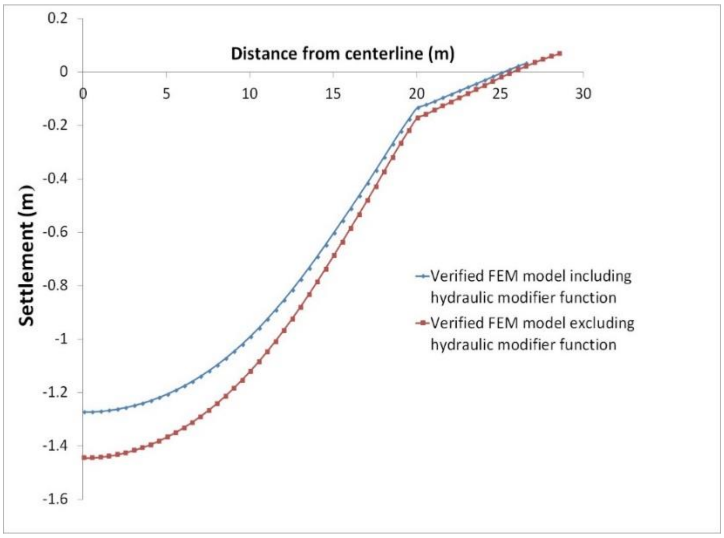
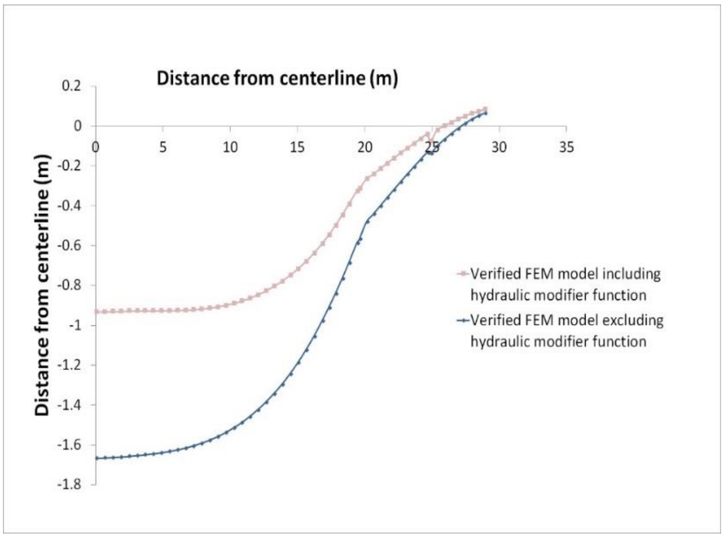


Figure 8. The settlement curve of the TS1 verified FEM model vs the settlement curve of the TS1 FEM model excluding the hydraulic function modifier (a) after 210 days (b) after 365 days

(a)



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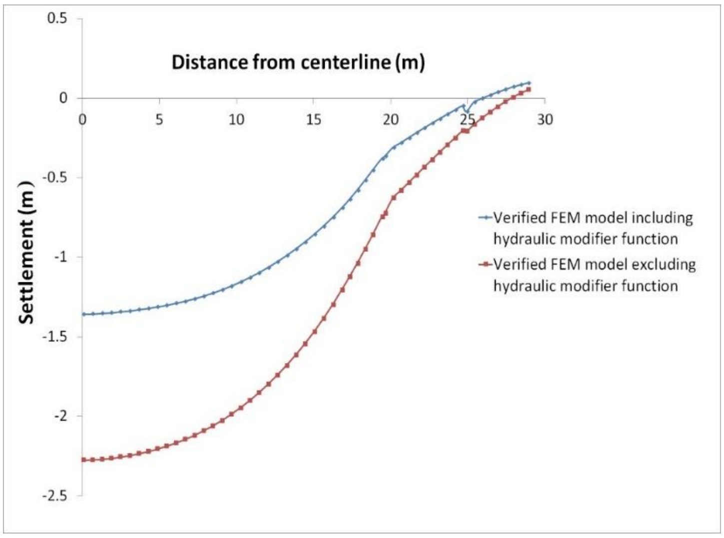
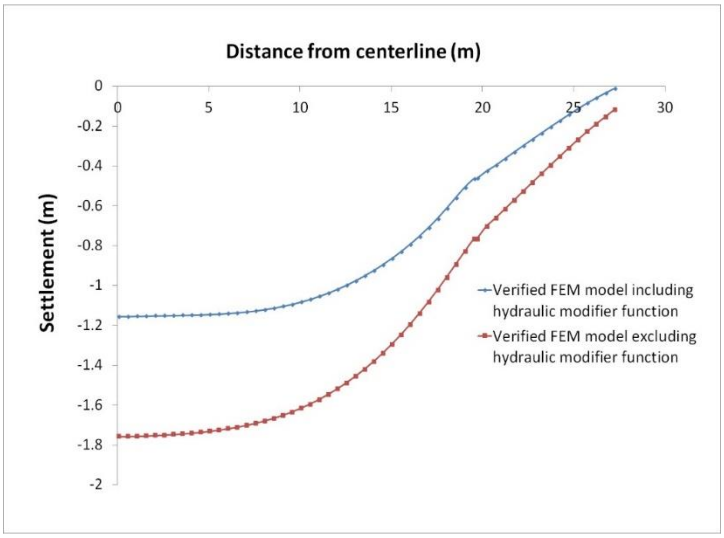


Figure 9. The settlement curve of the TS2 verified FEM model vs the settlement curve of the TS2 FEM model excluding the hydraulic function modifier (a) after 230 days (b) after 410 days

(a)



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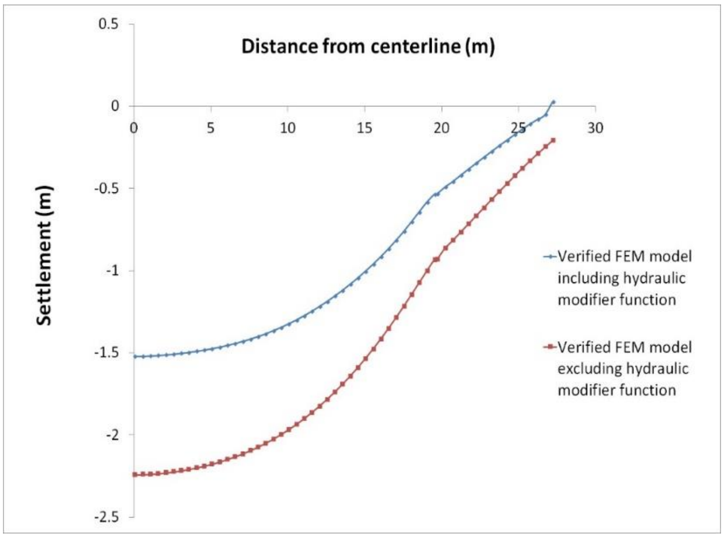

Figure 10. The settlement curve of the TS3 verified FEM model vs the settlement curve of the TS3 FEM model excluding the hydraulic function modifier (a) after 230 days (b) after 410 days

4. CONCLUSION

A case history of trial embankments as TS1, TS2, and TS3 that were built to investigate the consolidation process in Bangkok airport were modeled and verified. In the verification process, there was a difference in field data and FEM calculations curve's pass that can be attributed to the existence of sand lenses and also the delay in the consolidation of soil stratum due to the degradation of clay microscopic structure.

Since the hydraulic modifier function vs stress increment is an extremely important parameter in modeling the consolidation process, especially in soil treatment system incorporating PVDs and surcharge loading, the curves that were used in the FEM modeling procedure was presented for each layer. To clarify the importance of the hydraulic modifier function, it was omitted and then the FEM calculations were done base on verified models again. The results showed that by the omission of the function, the results were overestimated dramatically, especially as the distance of the PVDs decreased, and the overestimation of the results increased significantly.

- For the TS1 embankment, the settlement increased from 0.78 m to 0.87 m in 210 days. In 365 days, the settlement increased from 1.27 m to 1.44 m.

- For the TS2 embankment, the settlement increased from 0.93 m to 1.67 m in 230 days. In 410 days, the settlement increased from 1.36 m to 2.27 m.
- For the TS3 embankment, the settlement increased from 1.15 m to 1.79 m in 230 days. In 410 days, the settlement increased from 1.52 m to 2.24 m.

The inclusion of hydraulic functions that calibrate the model for every step of loading is essential in modeling of such problems. By the omission of the hydraulic conductivity, since m_v in the modified cam clay model is a constant integer and an input parameter in most of the FEM programs, the consolidation process speeds up unrealistically and the resultant settlements, as a result, are far from reality.

For the design phase, this function should be calculated from lab tests from preferably undisturbed samples that were bored from the site and the resultant function be used as an inseparable part of modeling and calculations. In the assessment phase, to build a model that can demonstrate the real condition of the project (during the construction or after the compilation of the project) the modifier function should be obtained by lab results from soil investigation reports or in the absence of such data's, estimated reasonably.

FUNDING/SUPPORT

Not mentioned any Funding/Support by authors.

ACKNOWLEDGMENT

Not mentioned by authors.

AUTHORS CONTRIBUTION

This work was carried out in collaboration among all authors.

ONFLICT OF INTEREST

The author (s) declared no potential conflicts of interests with respect to the authorship and/or publication of this paper.

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