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Finite Element Modeling of Embankment Resting on Soft Ground Stabilized with Prefabricated Vertical Drains

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ABSTRACT: This paper presents the numerical modelling of embankment resting on soft soil improved by the use of prefabricated vertical drains (PVDs). The study has been validated with the field measurements of settlements and excess pore pressures for a trial embankment at the Krishnapatnam Ultra Mega Power Project (KUMPP) in Nellore, Andhra Pradesh, India. The paper elaborately highlights the intricate effect of various parameters such as the drain spacing, reduction of permeability due to smear, and the efficiency of floating drains. Two-dimensional finite element modelling was carried out using PLAXIS 2D. In the analysis, classical axisymmetric solution for consolidation by vertical drains has been converted into an equivalent two-dimensional plane strain analysis. The comparatives reflect the agreements and differences between the field measurements and the results obtained from the numerical model. Based on the results, the state of smear prevailing in the field has been identified. The numerical study suggests that the optimal length of the partially penetrating drains (75-80% of the full penetration) would be efficient in aiding sufficient vertical consolidation of the soft soil site, thus making its usage more economical.

KEYWORDS: Preloadng **e**mbankment, Prefabricated vertical drains (PVDs), Smear effect, Floating PVDs, Soil constitutive models

1. INTRODUCTION

The Increase of population and accompanying demands for development and infrastructures have resulted in the utilization of soft soil sites primarily comprising of compressible soils. Present day land reclamation also dictates the construction to be carried out on the soft marine clays. Soft soil possessing low strength and high compressibility needs to be improved to avoid excessive settlement and prevent stability failure. The coastal belt of India comprises of very soft marine clays, where lots of construction activities are ongoing. Many ground improvement methods have been proposed and are used on soft soil sites to improve its bearing capacity and minimize the anticipated settlement (Bergado et al., 1994). The methods can be categorized as mechanical stabilization, hydraulic modification, chemical stabilization and the inclusion of confinement materials, such as geosynthetics, into the soil (Hausmann, 1990; Xanthakos et al., 1994; Schaefer et al., 1997). One of the most popular methods of soft soil improvement, in the recent years, has been the use of prefabricated vertical drains (PVD) in combination with preloading (Barron, 1948: Atkinson and Eldred, 1981; Wood, 1982; McGown and Hughes, 1981; Mesri, 1981; Bergado et al., 1994, 1997; Chai et al., 2001; Xiao, 2001; Bo et al., 2003; Hein, 2008; Giridhar Rajesh et al., 2014).

Preloading refers to the process of compressing the soft soil sites using a controlled vertical stress prior to the placement of the actual construction load. Preloading is one of the most effective and economical methods to reduce post-construction settlement and improve the bearing capacity of the soft soil. Prefabricated vertical drains, when installed in soft soils, provide artificial drainage paths that serve the purpose of shortening the drainage length and accelerating the rate of primary consolidation settlement (Kjellman, 1948; Casagrande and Poulos, 1969; Hansbo, 1979, 1981; Patty, 1995). The consolidation time depends on the rate of outflow of water from the soil matrix (Carillo, 1942). The installation of vertical drain results in the development of horizontal gradient within the soft substrata, which leads the pore-water flow through the vertical drain and subsequently into and out through a freely draining material.

It has been observed that few reports are present of the usage of PVD in Indian soft soil sites. Soft soils located in different parts of the world have diverse behavior, and each type needs special treatment due to their unique characteristics (Lin et al., 2000). This paper reports the finite element modelling of an embankment resting on PVD improved soft soil to provide a detailed illustration of various issues related to the chosen problem. A detailed study has been conducted, and the results from the numerical model are validated with the field measurements of settlements and excess pore pressures for a trial embankment at the Krishnapatnam Ultra Mega Power Project (KUMPP) in Nellore, Andhra Pradesh, India. The understandings from this case study can be used for other similar instances.

2. SITE AND SUBSOIL CONDITIONS

Krishnapatnam Ultra Mega Power Project (KUMPP), located in Nellore, Andhra Pradesh, is prevalently located on a soft soil site. Heavy installations such as the power plant, cooling towers, coal stockyard and other facilities are constructed for the project, and are to be supported by the soft substrata. In order to develop the ground improvement scheme to be adopted for the entire site, three areas were preliminarily chosen as the trial sites. In the trial areas, vertical band drains were installed, soil instrumentations were deployed, and preloading was carried out with the aid of trial embankments.

This paper presents the details of one of the test embankment site (TA-02). The detailed plan view of the trial test embankment area is shown in Figure 1.

The preloading embankment had a square base area of 50 m x 50 m (excluding side slopes), height of 4 m with 1V:2H side slopes. The test embankment site has been equipped with 5 settlement gauges (SG) and 2 piezometers (PM) at various locations. A field monitoring program was followed to measure the settlements and excess pore-pressures (Radhakrishnan, 2011). In order to determine the subsoil properties beneath the test embankment, 1 borehole stratigraphy (BH) and 2 static cone penetration tests (SCPT) have been conducted at the site.

Soil investigation in the area has shown that the average subsoil profile comprises of medium dense silty sand up to a depth of 6-7m below the existing ground level (EGL) overlying soft moderate to high compressible silty clay up to an approximate thickness of 14 m. Below the soft clay layer, there exists a stiff to very stiff clay which is considered as a competent foundation material. The subsoil data beneath the test embankment TA-02 has been provided in Table 1.

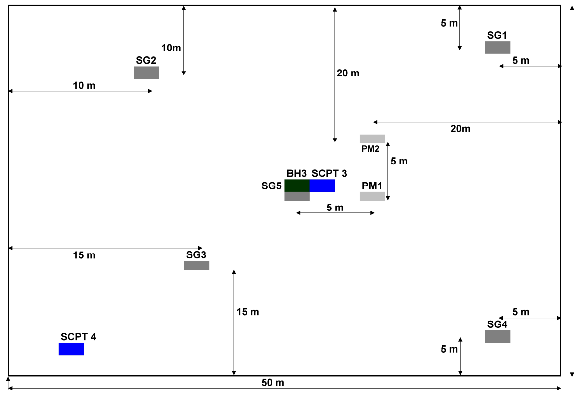

Figure 1 Schematic view of the TA-02 trial area; SG: Settlement gauge; SG-5 at center of the plot; PM1: Piezometer @ 15 m depth; PM2: Piezometer @ 10 m depth; BH: Bore hole; SCPT: Static cone penetration test location

Table 1 Summary of soil profile and average soil properties

Property	Medium Dense Sand	Soft Clay	Stiff Clay
Average stratum thickness	6-7 m	13-14 m	>20m
Bulk density (kN/m^3)	17	15	17
Undrained cohesion (c_u, kPa)	-	15-30	>50
LL (%)	-	60-72	-
PL (%)	-	20-38	-
Compression index (C_c)	-	0.3 -0.6	-
Coefficient of consolidation (c_v, m^2/yr)	-	0.7- 1.2	-
Initial void ratio	-	0.8- 1.2	-

3. PVD INSTALLATION AND STAGE LOADING

The PVD was installed using a hydraulic drain stitcher using a steel mandrel and a disposable drain shoe. The mandrel was rectangular in cross section with outside dimensions of 150 mm by 45 mm. The PVDs, having a dimension of 100 mm by 4 mm, were installed at a spacing of 2.5 m in a triangular pattern. The maximum depth of band drain installation was 20 m below GL. The upper silty sand layer being medium dense, the drain stitcher had difficulty penetrating this layer. The machinery had to be suitably modified prior to band drain installation. A sand drainage blanket, 300 mm thick, was spread above the installed band drains to allow easy flow of the discharged pore water from the drains during consolidation. The earth preload was place in layers of 300 mm thickness and was compacted to field requirement. The stages of preloading are as shown in the Figure 2.

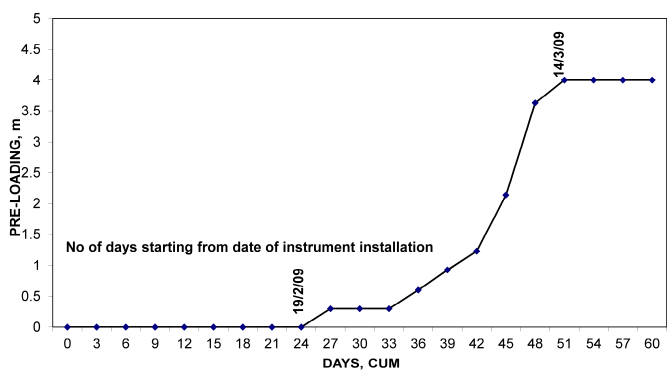

Figure 2 Preloading plan at the test site

4. FINITE ELEMENT MODELING OF PVD SUPPORTED EMBANKMENT

In order to carryout FEM analysis, PLAXIS 2D v2012 numerical modeling software was used. PLAXIS allows for an automatic generation of unstructured 2D finite element mesh with options for global and local mesh refinements. In the current study, medium coarse mesh is used for the analysis. As the embankment is symmetric, only half-section of the embankment is considered for the FEM analysis. The finite element model has been setup in plane strain condition with 15-noded triangular elements. A typical FEM model with the mesh structure is as shown in Figure 3.

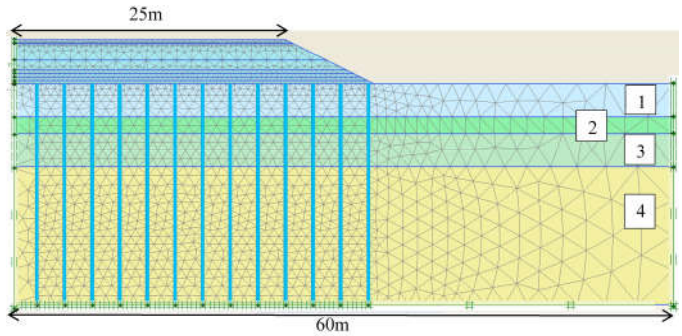

Figure 3 Typical finite element model with preloading embankment supported on PVD incorporated soft soil and its mesh structure; Soil layers 1: Very dense sand of 3m thickness; 2: Medium dense sand of 1.5m thickness; 3: Very dense sand of 3m thickness; 4: Soft clay of 12.5m thickness

PLAXIS allows various constitutive models to represent the behaviour of geomaterials. The prime difference between those models is the efficacy with which the stress-strain behaviour of the soil can be properly represented. In this study, three different constitutive modelling schemes have been used to predict the settlements and pore-pressures, and establish the efficacy of these models in representing the behavior of the embankment resting on PVD incorporated soft soil. The modelling schemes chosen for this study are:

Model 1. All soils are modeled using Mohr-Coulomb (MC) model

Model 2. Clay is modeled using Soft Soil Creep (SSC) Model, and remaining is modeled using MC model

Model 3. Clay is modeled using Hardening soil (HS) model and the remaining soil layers are modeled using MC model

4.1 Material Properties for Model 1

The MC model requires five parameters; Elastic modulus (E), Poisson's ratio (ν), friction angle (φ), cohesion (c), and dilatancy angle (ψ). In this case study, the Undrained-B method is used for the modeling the clay behavior, which works with the direct input of the undrained shear strength, $c = c_u$, $\phi = \phi_u = 0$, $\psi = 0$. The summary of material properties is shown in Table 2.

4.2 Material Properties for Model 2

In this scheme, clay is modeled using the SSC material model. The basic parameters required in SSC model are the modified compression index λ^*, modified swelling index κ^*, and the modified creep index μ^*. For an approximate estimate of the model parameters, the ratio λ^* / μ^* is considered in the range 15-25 and the ratio λ^* / κ^* is considered in the range of 5-10 (Almeida and Ferreira, 1993; Gunduz, 2010). The model parameters chosen for Model 2 are summarized in Table 3.

Table 2 Summary of material properties used in Model 1 [adopted from Radhakrishnan (2011)]

Parameter	Embankment	Very dense sand	Medium dense sand	Very dense sand	Clay
Material Type	MC	MC	MC	MC	MC
Material behavior*	D	D	D	D	U
γ_{unsat} (kN/m^3)	17	17	17	17	15
γ_{sat} (kN/m^3)	19	20	20	20	17
k_x (m/day)	2	0.0173	0.0173	0.0173	1.6x 10^{-05}
k_y (m/day)	2	0.0043	0.0043	0.0043	4.0x 10^{-06}
Elastic modulus (E_{ref}) (kPa)	30000	40600	42130	40600	1059
Poisson's ratio (v)	0.3	0.25	0.25	0.25	0.35
Cohesion (c_{ref}) (kPa)	1	3	3	3	25#
Friction angle (φ°)	30	41.5	36	41.5	0#
Dilatancy angle (ψ°)	0	0	0	0	0

*D: Drained, U: Undrained, γ_{unsat}: Soil unit weight above phreatic level, γ_{sat}: Soil unit weight below phreatic level, #: Effective values

Table 3 Soil parameters for Model 2 for KUMPP site

Parameter	Clay
Material model	SSC
Drainage type	Undrained (A)
Soil unit weight above phreatic level (γ_{unsat}) (kN/m^2)	15
Soil unit weight below phreatic level (γ_{sat}) (kN/m^2)	17
Horizontal permeability (without smear) (k_x) (m/day)	1.60x10^{-05}
Vertical permeability (without smear) (k_y) (m/day)	4.00x10^{-06}
Effective Cohesion (c') (kPa)	2
Effective Friction angle (φ') (°)	26.55
Dilatancy angle (ψ) (°)	0
Modified compression index (λ*)	0.1035
Modified swelling index (κ*)	0.01294
Modified creep index (μ*)	5.17x10^{-03}

4.3 Material Properties for Model 3

In this modeling scheme, clay is modeled using Hardening soil model. PLAXIS uses three different stiffness moduli in the HS model to describe the hyperbolic stress-strain curve. Those are the stiffness from the standard drained triaxial test (E_{50}), tangent stiffness from oedometer test (E_{oed}), and the stiffness modulus for unloading/reloading (E_{ur}). For this case study, due to non-availability of other data except E_{oed}, it has been assumed that $E_{ur} \approx 3E_{50}$ and $E_{oed} \approx E_{50}$. The summary of materials properties used in this model is presented in Table 4.

4.4 Modelling of Vertical Drains

The vertical draining function of the PVD can be modeled by application of the drain element having infinite permeability. During the consolidation analysis, the drain element works by prescribing zero excess pore pressure in all nodes that belong to a drain. Drains can be activated and deactivated in calculation phases making it possible to account for the delayed installation of vertical drains.

Table 4 Soil parameters for Modelling scheme 3 for KUMPP site

Parameter	Clay
Material model	HS
Drainage type	Undrained (A)
Soil unit weight above phreatic level (γ_{unsat}) (kN/m^2)	15
Soil unit weight below phreatic level (γ_{sat}) (kN/m^2)	17
Horizontal permeability (without smear) (k_x) (m/day)	1.60x10^{-05}
Vertical permeability (without smear) (k_y) (m/day)	4.0x10^{-06}
Effective Cohesion (c') (kPa)	0
Effective Friction angle (φ') (°)	26.55
Dilatancy angle (ψ) (°)	0
E_{50} (kPa)	1700
E_{oed} (kPa)	1700
E_{ur} (kPa)	5100

Most finite element analysis of soft embankment ground is based on plane strain assumption. However, in the actual case, the consolidation around the vertical drain is supposed to be axisymmetric. To obtain a realistic 2-D finite element analysis, an equivalent between axisymmetric and plane strain model should be established. The conversion of the axisymmetric unit cell to plane strain band drain has been achieved as per the technique proposed by Indraratna et al. (1997), and the same is depicted in Figure 4.

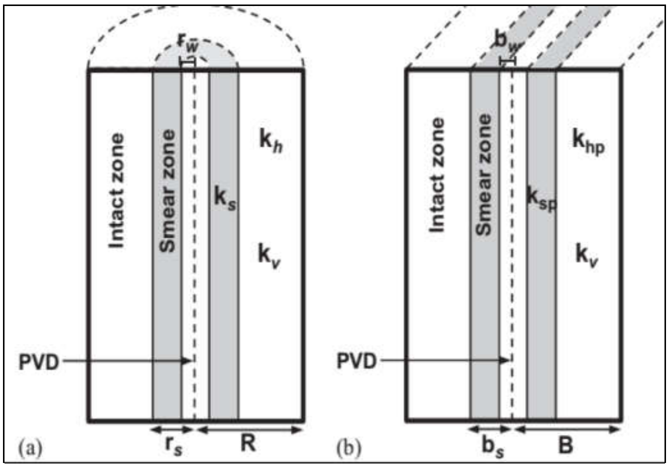

Figure 4 Conversion of axisymmetric unit cell to plane strain condition (as per Indraratna *et al*., 1997)

The equivalence between axisymmetric and plane strain conditions prior to FEM modeling has been carried out based on the permeability equivalency procedure proposed by Hird *et al.* (1995) as expressed by the following equation:

$$\frac{k_{pl}}{k_{ax}} = \frac{2}{3\left[\ln(n) + \left(\frac{k_{ax}}{k_s}\right)\ln(s) - \frac{3}{4}\right]} \tag{1}$$

where, k_{pl} is the horizontal permeability of undisturbed zone in plane strain unit cell, k_{ax} is the horizontal permeability of undisturbed zone in axisymmetric unit cell, k_s is the horizontal permeability of smear zone in axisymmetric unit cell, n is the influence ratio r_e/r_w, s is the smear ratio r_s/r_w., r_e is the radius of influence zone, r_w is the equivalent radius of vertical drain, and r_s is the radius of smear zone (Onoue et al., 1991; Indraratna and Redana, 1998, 2000; Sathananthan, 2005).

4.5 Modelling of the Staged Construction

Staged construction is the most important type of loading input. In PLAXIS, it is possible to change the geometry and load configuration by deactivating or reactivating the loads, volume clusters or structural objects as created in the geometry input. It enables an accurate and realistic simulation of various loading, construction and excavation processes. The modeling of drains and staged construction process adopted in FEM analysis is shown in Figure 5.

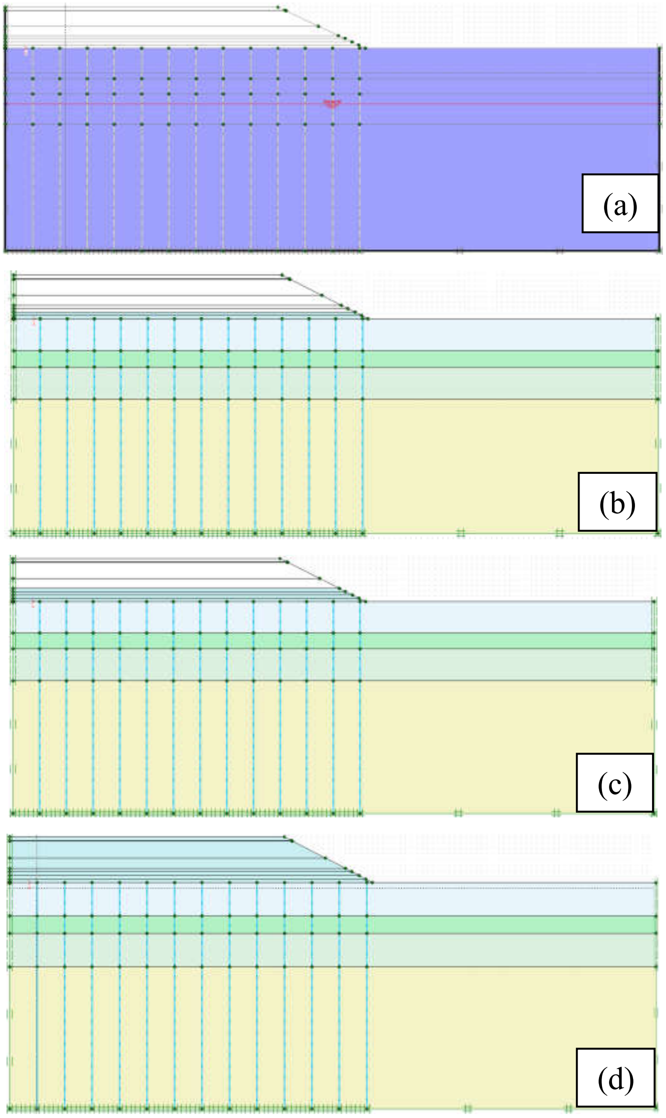

Figure 5 Staged constructions representing the progressive development of the preload embankment (a) Installation of PVDs (b) Embankment filling up to a height of 0.6 m (c) Embankment filling up to a height of 1.23 m (d) Full height embankment filling

4.6 Modelling of Consolidation Process

The decay of excess pore pressures with time can be computed using a consolidation analysis. A consolidation analysis requires the input of permeability coefficients in the various soil layers. A consolidation analysis introduces the dimension of time in the calculations. In order to correctly perform a consolidation analysis, a proper time step must be selected. Automatic time stepping procedures makes the analysis robust and easy to use. Consolidation analysis is supported by three main possibilities:

1. Consolidate for a predefined period, including the effects of changes to the active geometry (staged construction).
2. Consolidate until all excess pore pressures in the geometry have reduced to a predefined minimum pore pressure.
3. Consolidate until a specified degree of consolidation.

4.7 Modelling of Drainage

In principle, all model parameters in PLAXIS are meant to represent the effective soil response. An important component that affects the soil behavior is the presence of pore water. Pore pressures significantly influence the soil response. To incorporate the soil water interaction, different types of drainage modeling can be done. In the drained behavior, no excess pore pressures are generated. This is the case for dry soils subjected to a low rate of loading, and also for high permeability soils ensuring full drainage. Undrained behavior is used for saturated soils where pore water cannot freely flow through the soil skeleton. PLAXIS introduces three different ways of modeling undrained behavior which is termed as Undrained A (UA), Undrained B (UB) and Undrained C (UC) respectively. The details of the undrained analyses can be obtained from PLAXIS Manual (PLAXIS, 2012).

5. RESULTS AND DISCUSSIONS

All the settlements in this study are measured at the center of the embankment, and excess pore pressures are measured at a distance of 5 m from the center; both being measured at a depth of 15 m from the ground surface (not from the top of the embankment). Figure 6 depicts the deformed shape of the embankment after the consolidation settlement period.

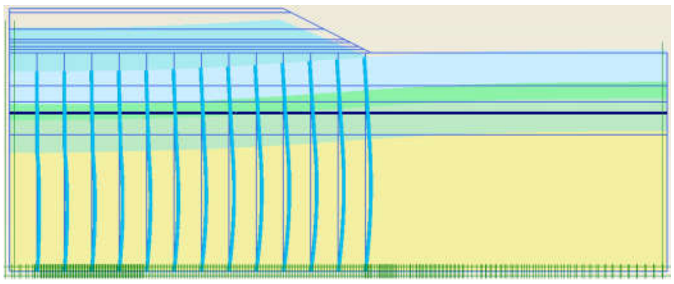

Figure 6 Post-consolidation deformed shape of the embankment resting on PVD incorporated soft soil

5.1 Effect of Application of PVDs

In order to portray the effect of PVDs, two different models of embankment resting on soft soil have been analyzed, one with PVD and the other without PVD. It is understandable that PVDs will be associated with quick dissipation of generated excess pore water pressure, and hence, in turn; will affect the time taken to achieve the ultimate settlement.

5.1.1 Settlement profiles

Figure 7 shows that the rate of settlement in embankment without PVD is very slow as there are no accelerated drainage paths for the escape of pore-water. In the case of embankment with PVD, as there is an artificial drainage path created for the escape of water, the rate of settlement is noticeably fast. The time taken to achieve the ultimate settlement (530 mm) due to PVD installations is around 1157 days, while, by that time, the settlement in the embankment without PVD is meager 142 mm.

5.1.2 Excess Pore Pressure profiles

Stage construction of an embankment, using specific fill thickness during a particular time interval, leads to the rise of the pore-pressure. At the same time, the PVDs embedded within the soil starts functioning to dissipate the pore-pressure generated in the vicinity, although the functioning may not be to the full extent of its depth. This result in a reduction of the total pore pressure generated at the end of the preloading period. It is noted from Figure 8 that in the absence of PVD, the developed pore-pressure does not reduce substantially, even after sufficient time of consolidation, due to the inherent low permeability of the soil leading to low dissipation of pore-water.

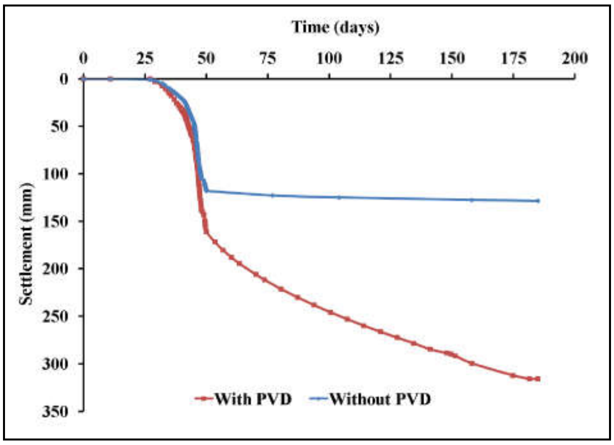

Figure 7 Effect of application of PVD on the time-dependent settlement of embankment on soft soil

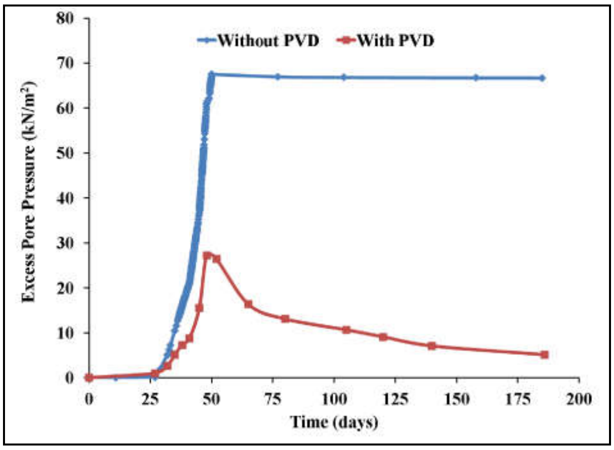

Figure 8 Effect of application of PVD on the excess pore-pressure of embankment on soft soil

Figures 9-12 illustrate the distribution of pore-pressures in the soft soil foundation, with and without the incorporation of PVDs. The results are presented in terms of the excess pore pressure at the end of preloading and the end of consolidation phases. It is observed from Figure 9 that the maximum pore-pressure reached in most part of the foundation beneath the embankment is approximately 90 kPa, which does not decrease significantly even after a period of consolidation for 186 days. As evident from Figure 10, the pore-pressure decreases to 70 kPa, that is not substantial at all. Figure 11 illustrates the effect of the application of PVD on the generated pore-pressure after the preloading is completed. It is noted that in the major part of the soft soil beneath the embankment, the pore-pressure generated is approximately 65 kPa, which is significantly lower than the case without PVDs. Moreover, the presence of PVD also resulted in significant reduction of the pore-water pressure as evident to be in the range of 40 kPa (Figure 12). A larger concentration of pore-pressure is observable in the left boundary; however, this is only due to the presence of boundary effect on the obtained result.

5.2 Effect of Smear

Smear in the adjacent area of the vertical drain is primarily caused by the disturbance generated during the installation of the vertical drain with the aid of a mandrel and drain stitcher (Akagi, 1977). The smear effects due to disturbance during installation were investigated by varying the ratio of soil horizontal permeability to the smeared soil permeability (k_h/k_s).

The results obtained from the effect of consideration of smear are compared with the field data in an attempt to identify the amount of smear which might have actually occurred in the test site due to the installation of the drains.

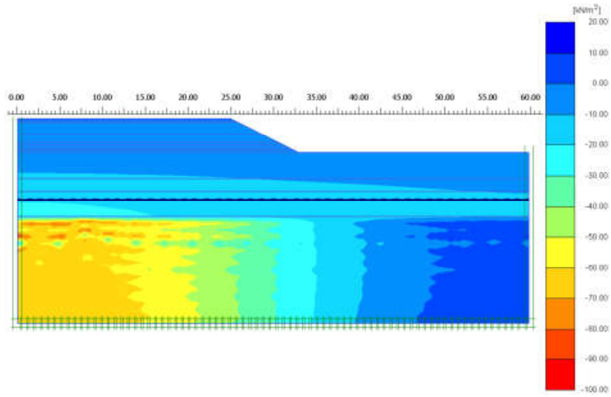

Figure 9 Distribution of pore pressure within the soft soil at the end of preloading (No PVDs)

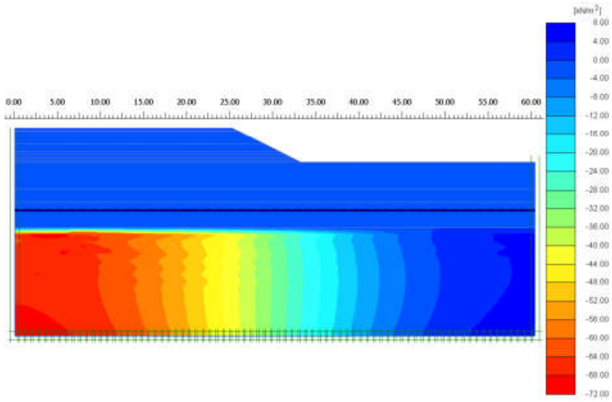

Figure 10 Distribution of pore pressure within the soft soil at the end of 186 days (No PVDs)

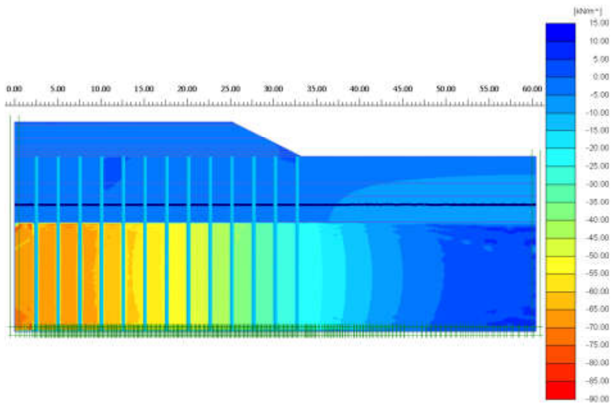

Figure 11 Distribution of pore pressure within the soft soil at the end of preloading (With PVDs)

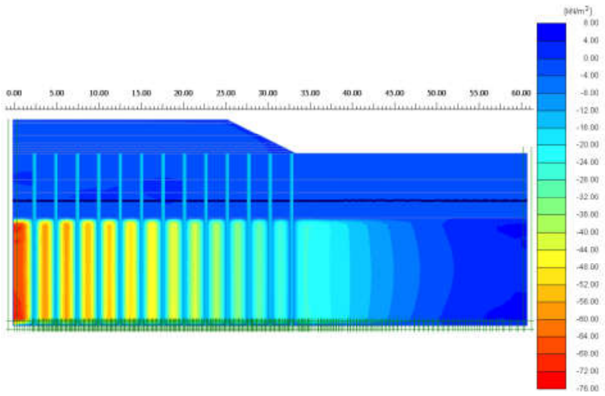

Figure 12 Distribution of pore pressure within the soft soil at the end of 186 days (With PVDs)

5.2.1 Settlement profiles

For investigating the effect of smear, k_h/k_s have been varied over a range of 1-10. Figure 13 depicts the effect of decreasing the permeability of the smeared soil on the settlement of the embankment. It is observed that as the degree of smear increases, the rate of settlement vividly decreases. More the smear, lesser is the smear zone permeability, and hence more is the magnitude of k_h/k_s. A larger magnitude of smear creates more disturbances in the zone adjacent to the drain, and results in lowering of the soil permeability, and hence leads to lowered rate of dissipation of pore-water pressure, and thus, decreased rate of settlement.

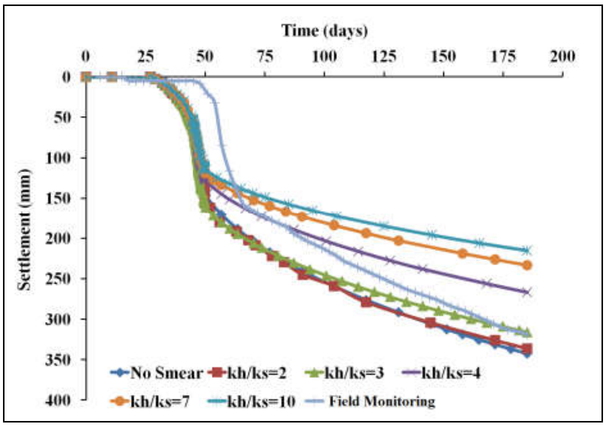

Figure 13 Effect of smear on the time-dependent settlement of embankment on soft soil

Figure 14 shows a histogram representation of the sensitivity of the degree of smear on the settlement attained after particular time duration. It is observable that in the range of 2-3, k_h/k_s ratio does not significantly affect the settlement attained after a particular time (186 days for this case). However, as the ratio exceeds 3, there is a drastic reduction in the settlement. This is attributed to the significant decrease in the degree of consolidation as the permeability of soil in the smeared zone gets noticeably decreased. Fig. 13 exhibits that as k_h/k_s ratio exceeds 4, the predicted settlement is severely underestimated. Gopalan (2010) has also reported the similar findings.

Figure 15 illustrates the effect of smear on the time required to reach the ultimate settlement. It is noticeable that the effect of smear becomes significantly prominent when the k_h/k_s ratio exceeds 4. For k_h/k_s=3, the settlement profile is nearly matching the field settlement curve; although, the settlements predicted by the FEM analysis are slightly lower (less than 10%).than the field monitored settlement. A study by Arulrajah *et al.* (2004) on the Changi reclamation project in Singapore also reports similar findings. This could be attributed to the mismatch of the soil behavior due to the inefficacy of the chosen soil model to represent its coupled and complex behaviour. Moreover, the model falls short in explaining the change occurring during simultaneous staged loading and consolidation.

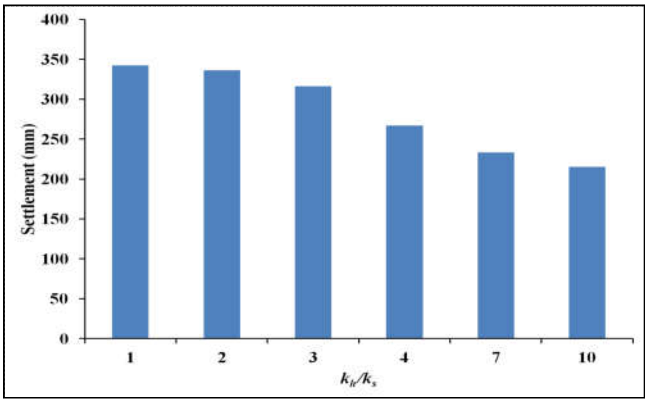

Figure 14 Sensitivity of k_h/k_s on predicted settlement after a particular time duration (186 days for this case)

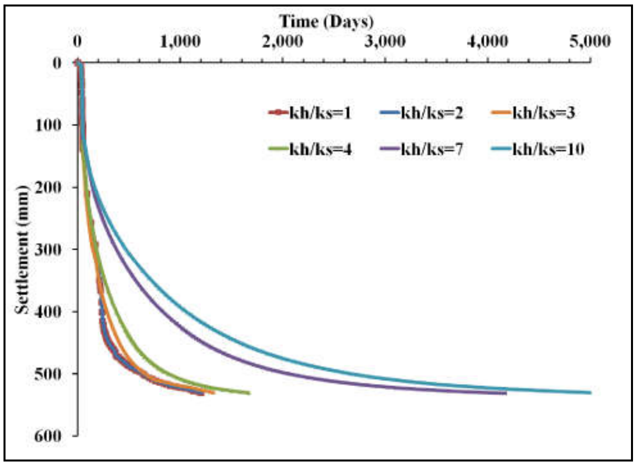

Figure 15 Time-settlement curve for different k_h/k_s ratios

5.2.2 Excess pore pressure profiles

The excess pore pressures obtained from FEM are compared to the excess pore pressures of field monitoring data. There is a large variation between field monitored value and FEM predicted value. There are drastic variations of pore pressure between the recording points merely separated by 1 m, which can be observed from the Figure 16. At a distance of 6.18 m from the center of the embankment, the values predicted by FEM and that obtained from field monitoring are found to be similar. In the field, the piezometer was inserted at 5 m from center to a depth of 15 m. However, it has been mentioned in the literature (Radhakrishnan, 2011) that due to the presence of the dense sandy layer, there had been significant difficulty in the installation of drains. It is not very surprising that the piezometer also suffered similar fate during its placement and installation. When it has been inserted from the top of the embankment, there is always a possibility of tilt, which might have made the piezometer reach a different point than which is marked in the top plan view of the position of the instrumentation.

The spacing between the drains has been 2.5 m, and even a shift of the base of the piezometer by 40-50 mm may result in recording lower pore-pressure values. As a result, with respect to the insertion point marked in the top of the embankment, the pore-pressure values recorded would have been different. It is understandable that, if at all, a proper match with the pore-pressures is required, the proper nodal location in the FEM model should be chosen. Fig. 17 depicts the variation of the excess pore-pressure in between two drains. Within a drain spacing of 2.5 m, the pore-pressure varied in the range of 0 kPa (at the drain – free flow) to 60 kPa (mid-way between two drains). Hence, any small tilt in the installation of the piezometer will result in a significantly different record in the pore-pressure data collected from the field.

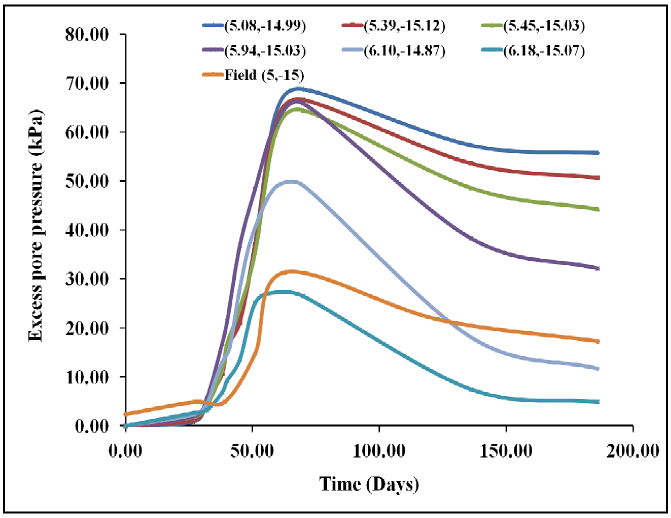

Figure 16 Variation of excess pore pressure with time at different points in the embankment

5.3 Effect of PVD Spacing

It is quite customary that spacing of the PVD will significantly affect the dissipation of the generated excess pore-pressure and hence the settlement of the embankment (Chai and Miura, 1999), which is quite evident from the pore-pressure gradient shown in Figure 17. Three different models (of same geometry and soils layers as that of Figure 3) have been analyzed with varied drain spacing particularly (a) 1m c/c (b) 2.5 m c/c, and (c) 2.5 m c/c at the central portion of the embankment and 4m c/c near the periphery and side slopes (Figure 18).

Figure 19 depicts the effect of drain spacing on the settlement of the embankment. It is observed that as the spacing decreases, the rate of settlement increases, primarily due to the accommodation of more number of drains and greater dissipation of pore-water pressure. Since the embankment produces uniform load on the underlying foundation, more pore-water pressures develop towards the central of the embankment. The generation of pore-pressure is comparatively lower in the peripheral regions. Hence, as noted, a decrease of spacing towards the periphery and side slopes, does not significantly affect the rate of settlement.

5.4 Effect of Soil Constitutive Models

In this study, three different models have been chosen to represent the soft clay layer in order to predict settlements. The clay layer has been modeled by MS, HS and SSC model respectively in separate models, while in all the three models, rest of the material properties are maintained identical. Effects of the chosen constitutive models are judged on the basis of the settlement profiles as depicted in Figure 20.

Ideally, SSC model should be able to better represent the behavior of long-term deformation of clayey soil under compression or consolidation. However, it is observed from Figure 20 that the settlements predicted by SSC and HS models are substantially higher than that obtained from the MC model. This anomaly can be attributed to the determination and usage of the derived material model parameters. Owing to the lack of proper field data, the material parameters of the HS model were calculated based on the assumption of $E_{ur} \approx 3E_{50}$ and $E_{oed} \approx E_{50}$. Similarly, in case of SSC model, the assumption of $\lambda^* / \kappa^* = 8$ and $\lambda^* / \mu^* = 20$ have been used.

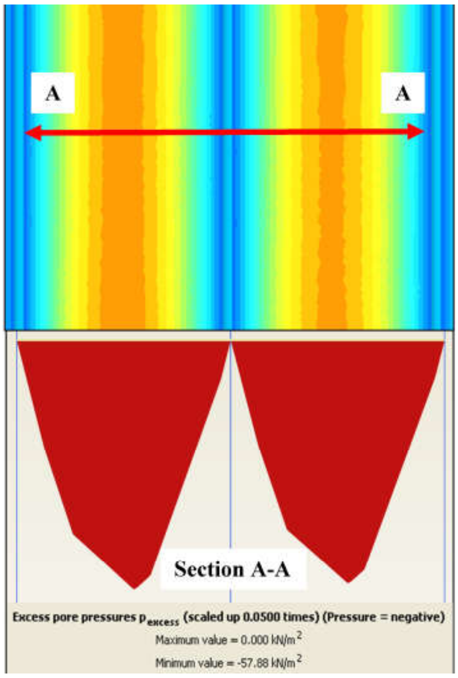

Figure 17 Variation of excess pore pressure in between two PVDs

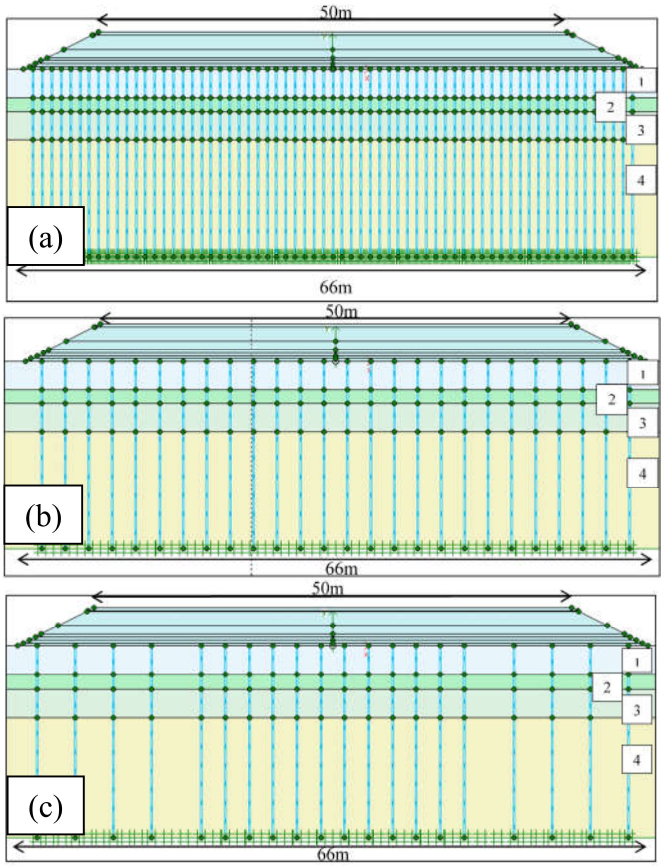

Figure 18 FEM Model with various PVD spacings (a) 1 m c/c (b) 2.5 m c/c (c) 2.5 m c/c at the center and 4 m c/c at the periphery; Soil layers 1: Very dense sand of 3m thickness; 2: Medium dense sand of 1.5m thickness; 3: Very dense sand of 3m thickness; 4: Soft clay of 12.5m thickness;

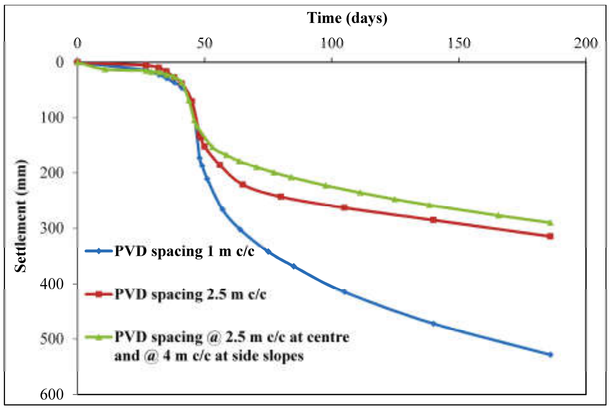

Figure 19 Effect of PVD spacing on the rate of settlement

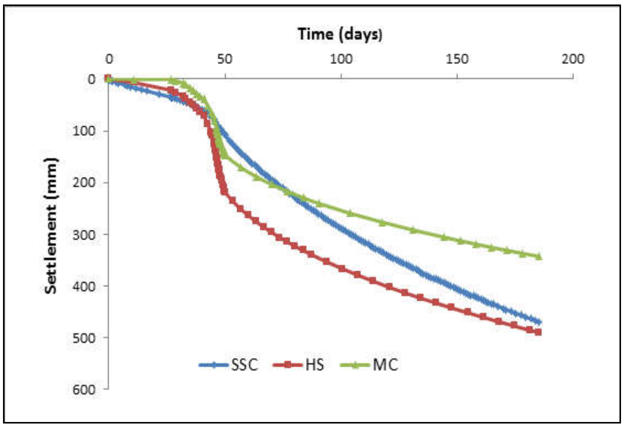

Figure 20 Time-dependent settlement based on the use of different constitutive models for clay

In a practical situation, it is very normal that these relationships do not hold true for the site subsoil under investigation, and the adoption of these parameters might actually result in a different soil constitutive behavior than that predominant in the site. This may lead to the observed discrepancy in the results.

5.5 Application of Floating PVDs

In order to explore this possibility of floating PVDs for the present study, models have been analyzed by varying the ratio of length of drain (L) to the thickness of soft soil stratum (H) in the range of 0.5-1. Figure 21 shows a typical FEM model for partially penetrating drains with L/H=0.7.

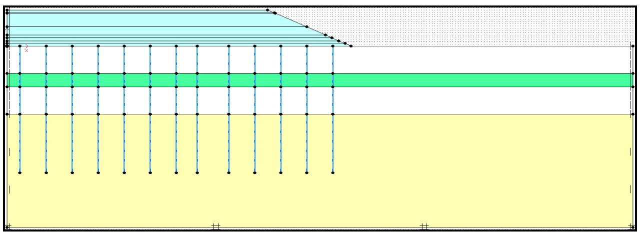

Figure 21 Typical FEM model for floating PVDs (L/H=0.7)

The effect of the provision of various configurations of floating drains is studied through the settlement profile observed from FEM models as shown in Figure 22. It is observed that as the L/H ratio decreases, the rate of settlement decreases due to the smaller drain length. Without significantly affecting the rate of settlement, drain length can be reduced by 10-15% of the full penetration length, as observed in the present study. As shown in Figure 22, the degree of consolidation also does not get substantially changed if the said reduction is utilized. A study by Ikhya and Schweiger (2011) on land reclamation project for Cirebon power plant in Indonesia also reported similar findings.

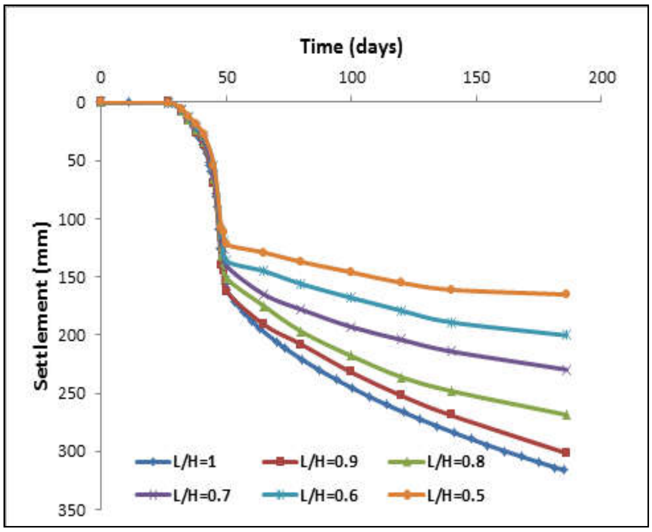

Figure 22 Variation of time dependent settlement for various length of floating PVDs

Figure 23 illustrates the distribution of pore pressure for a typical floating drain configuration (L/H=0.7) for the stages just after preloading, and after 186 days of consolidation, respectively. It is observed that the application of floating PVDs is effective in reduction of the pore pressure within the depth of penetration of the drains.

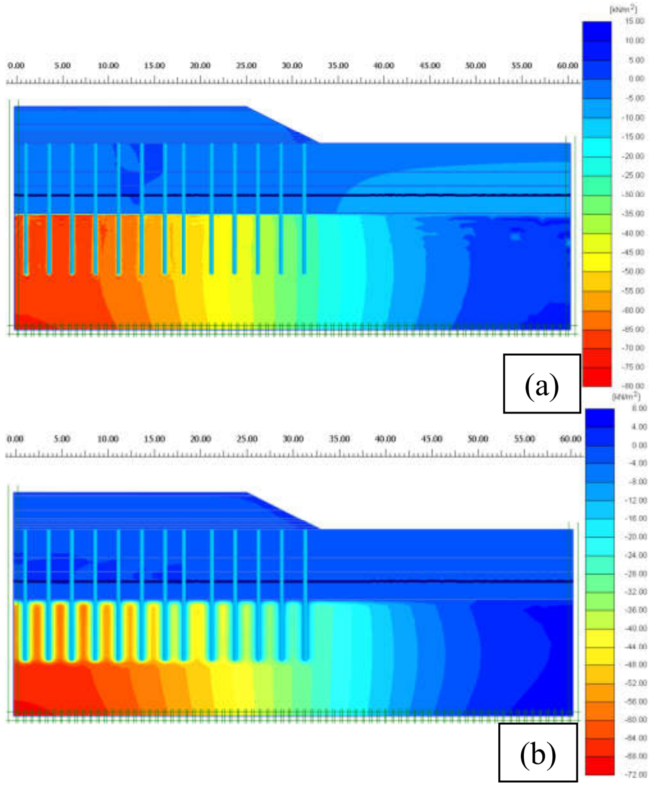

Figure 23 Generation of excess pore pressure for floating PVDs (a) at the end of preloading (b) at the end of 186 days of consolidation

However, beyond that depth, an accumulation of excess pore pressures are noticeable. It is understandable that the effect of the pore-pressure generation, its dissipation, and change in the degree of consolidation beyond the floating PVDs will have insignificant effect if the soft soil stratum is of large thickness. Hence, for deeply buried soft soil stratum, floating PVDs can serve as a better alternative in comparison to the fully penetrating drains, as it would help in the substantial reduction of the project cost (Indraratna and Rujikiatkamjorn, 2008).

6. CONCLUSIONS

The principal intent of this study was to predict the settlement of an embankment over soft ground, treated with preloading and PVD, through finite element analysis. Apart from the primary target, effect of various other issues namely the drain spacing, degree of smear, choice of soil constitutive model and the application of floating PVDs have been explored and investigated.

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

1. The settlement predicted from FEM analysis and the actual values measured in the field for the PVD treated ground are in reasonable agreement with each other. The settlements predicted by the FEM analysis are slightly lower than the field monitored settlement. This could be due to the decrease in soil permeability with an increase of effective stress, which has not been properly accounted in the analysis. Moreover, simultaneous loading and consolidation could not be modeled using the available soil constitutive models. Apart from that, uncertainty in the determination of the soil model parameters from the available field and laboratory experiment data, which fall short in proper determination of model parameters in many cases, might also result in the mismatch between the field measurements and FEM predictions.
2. The sensitivity study of the degree of smear, in terms of the k_h/k_s ratio, indicated that higher the ratio, the lower is the rate of settlement. As k_h/k_s ratio exceeds 3, the predicted settlement is underestimated significantly. The value of k_h/k_s can be taken as 2-3 for this particular case study. This particular case study reveals that the effect of smear is small for efficient vertical drains.
3. Without significantly affecting the rate of settlement, the drain length can be reduced by 15-20% with respect to the full-length penetration.
4. The number of drains used in the side slopes can be decreased for obtaining an optimum and economical design of PVD installations.
5. Pore-pressure distribution in between the drains varies drastically. Hence, any discrepancy in the installation of the piezometers in the field can lead to substantially different data.

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