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Soft ground improvement by vacuum-assisted preloading

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SOFT GROUND IMPROVEMENT BY VACUUM-ASSISTED PRELOADING

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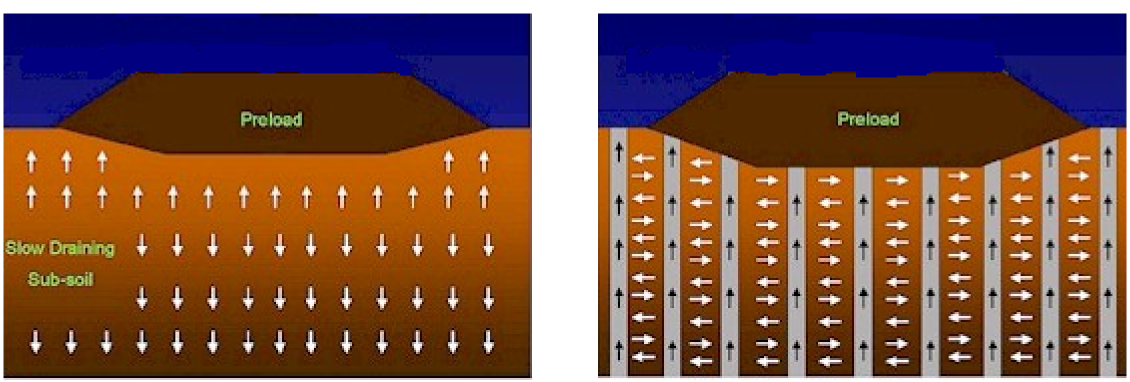
ABSTRACT

This paper describes the behaviour of soft soil foundation stabilized with vacuum-assisted preloading at the New Bangkok International Airport, Thailand. An analytical solution considering the variation of soil permeability and compressibility and a finite element analysis based on an equivalent plane strain model developed by the authors are employed to investigate the performance of the test embankment. The converted equivalent plane strain parameters are incorporated in the finite element code *ABAQUS.* The associated settlement, excess pore pressure and lateral movement are predicted and compared with the available field measurement. The data indicate that the efficiency of the prefabricated vertical drains depends on the magnitude and distribution of vacuum pressure as well as on the extent of air leak protection provided in practice. The height of sand surcharge and consolidation time are significantly reduced in comparison with the conventional method of surcharge alone. The effectiveness of this method, its economies and its merit potential are also discussed.

1. INTRODUCTION

Many coastal regions of Australia and Southeast Asia contain very soft clays, which possess poor geotechnical properties such as high compressibility and very low bearing capacity (Indraratna et al., 1992). The construction of highway and railway embankments on normally consolidated soft soil deposits has been affected by the excessive differential settlements, lateral displacements in the absence of an appropriate ground improvement prior to construction. To prevent the unfavorable conditions, the application of preloading with prefbricated vertical drains (PVDs) prior to the construction has been popularly employed in many large scale projects (Hansbo, 1981; QDMR, 1991; Indraratna and Redana, 2000; Chu et al., 2004) . This method accelerates the consolidation by providing a shorter drainge path (Figure 1). The gained shear strngth of the foundation can be achieved due to rapid excess pore pressure dissipation. It is also well-known that the high outward lateral movement causing the embankment instability can also be reduced via this method.



(a) (b)

Figure 1 Effect of vertical drain on drainage path; (a) without vertical drains and (b) with PVDs (http://www.americandrainagesystems.com/wick_drains.htm)

In the case of hydraulic fill with low bearing capacity in land reclamation projects where the height of surcharge is restricted due to the low shear strength of soft soil, vacuum-assisted consolidation is an ideal method for ground improvement (Shang et al., 1998; Indraratna et al., 2004; Chu and Yan, 2005). The application of vacuum pressure as apparent surchage load with PVDs can be used as a replacement of high embankment fill (Choa, 1990). Vacuum preloading method was first introduced by Kjellman (1952) to improve the strength of soft soil. The applied negative vacuum pressure is propagated along the PVDs length to deep subsoil layer, resulting in an increase in lateral hydraulic gradients ($i = \frac{1}{\gamma_w}\left\{ \frac{\partial u}{\partial r} + \frac{\partial u_{vac}}{\partial r} \right\}$) and effective stresses in soil where u and u_{vac} are excess pore pressure generated by preloading and suction pressure generated by vacuum pump, resprectively. Consolidation can be accelerated without increasing excess pore pressure (Cognon et al., 1994; Qain et al., 1992).

In this paper, the New Bangkok International Airport located about 30km from the city of Bangkok, Thailand was analysed (AIT, 1995). The site was occupied by ponds for fish farming and for agricultural purposes in the past. The area is often flooded during the rainy season and the soil water content is very high. Therefore, soft clays, mainly of marine or deltaic origin, often present substantial construction problems, which require ground improvement techniques to prevent excessive settlement and lateral movement. Soft soil improvement by prefabricated vertical drains (PVDs) combined with preloading is one of the most successful applications to this site (Indraratna and Redana, 2000). The multi-staged embankment construction, applied to prevent the embankment instability, increases the consolidation time significantly. In this regard, vacuum preloading combined with surcharge was considered as a feasible alternative to shorten the consolidation time and to reduce the height of surcharge.

Comprehensive analytical solution for soil consolidation under vacuum condition incorporating non-linear soil properties (e.g. compressibility and permeability) for vacuum preloading with vertical drains is introduced, under both axisymmetric and *equivalent* plane strain conditions. The analytical and numerical predictions are compared to the field data (e.g. settlements, lateral displacements and excess pore pressures). Finally, the performance of vacuum preloading method is evaluated by comparing to the conventional method of surcharge alone.

2. BACKGROUND

The vacuum preloading method was firstly proposed by Kjellman (1952) for cardboard wick drains. It has been used extensively to accelerate the consolidation process for improving soft ground such as Philadelphia International Airport, USA and Sunshine Motorway, Australia (Holtan, 1965 and QDMR, 1991). Recently, the PVDs system has also been included to distribute vacuum pressure to deep subsoil layer and thereby increase the consolidation rate (e.g. Chu et al. 2000). The increase in the effective stress in soil mass for vacuum application is caused by the vacuum application instead of the conventional surcharge. Therefore, the extent of surcharge fill can be decreased to achieve the same amount of settlement. The effective stress related to suction pressure increases equiaxially, and the corresponding lateral movement is compressive, inducing horizontal movement towards the embankment centreline. Consequently, the risk of shear failure can be minimized even at a higher rate of embankment construction. Yan and Chu (2003) showed that the cost of soil improvement by vacuum preloading reduces over one-third that by conventional surcharge alone. The effectiveness of this system relies on: (a) reliability (airtight) of membrane, (b) effectiveness of the seal between the ground surface and the membrane edges and, and (c) soil conditions and the location of ground water level (Cognon et al., 1994).

In saturated soils, the total stress (σ) at any point within the soil mass is the combination of the effective stress (σ') and the pore pressure (u) (Terzaghi, 1943). Thus, the total stress at any point within the soil mass can be written as:

$$\sigma' = \sigma - (+u_{\Delta p}) \tag{1}$$

Under the surcharge load alone, the effective stress is gained by the dissipation of positive excess pore water pressure after the load application. In contrast, the effective stress is increased by the applied negative pore pressure ($-u_{vac}$) under the vacuum condition. Equation (1) can be rewritten based on the vacuum and fill preloading as:

$$\sigma' = \sigma - (+u_{\Delta p}) - (-u_{vac}) \tag{2}$$

It can be seen that the effective stress increases by negative suction, thereby, reducing the risk of shear failure. The performance of this system depends on the vacuum condition under the airtight membrane (Indraratna et al. 2004). The intensive pore pressure measurement under membrane should be performed to verify the reliability of the vacuum system.

3. THEORETICAL CONSIDERATIONS

The triangular grid installation pattern of PVDs and axisymmetric unit cell are illustrated in Figures 2a and 2b, respectively. Field experience has shown that when vacuum pressure is applied in the field through prefabricated vertical drains (PVDs), the suction head along the drain length may decrease with depth, thereby reducing the efficiency (Indraratna et al. 2004). Pore pressure measurements along the drain in the large-scale consolidometer at the University of Wollongong clearly indicated that the vacuum head decreases down the drain length even in short vertical drains. In order to study the effect of vacuum loss, the vacuum pressure distribution along the soil-drain boundary is considered to vary linearly from $-p_0$ at top of the drain to k_1p_0 at the bottom of the drain, where k_1 is a ratio between vacuum pressure at the bottom and the top of the drain (Figure 2c).

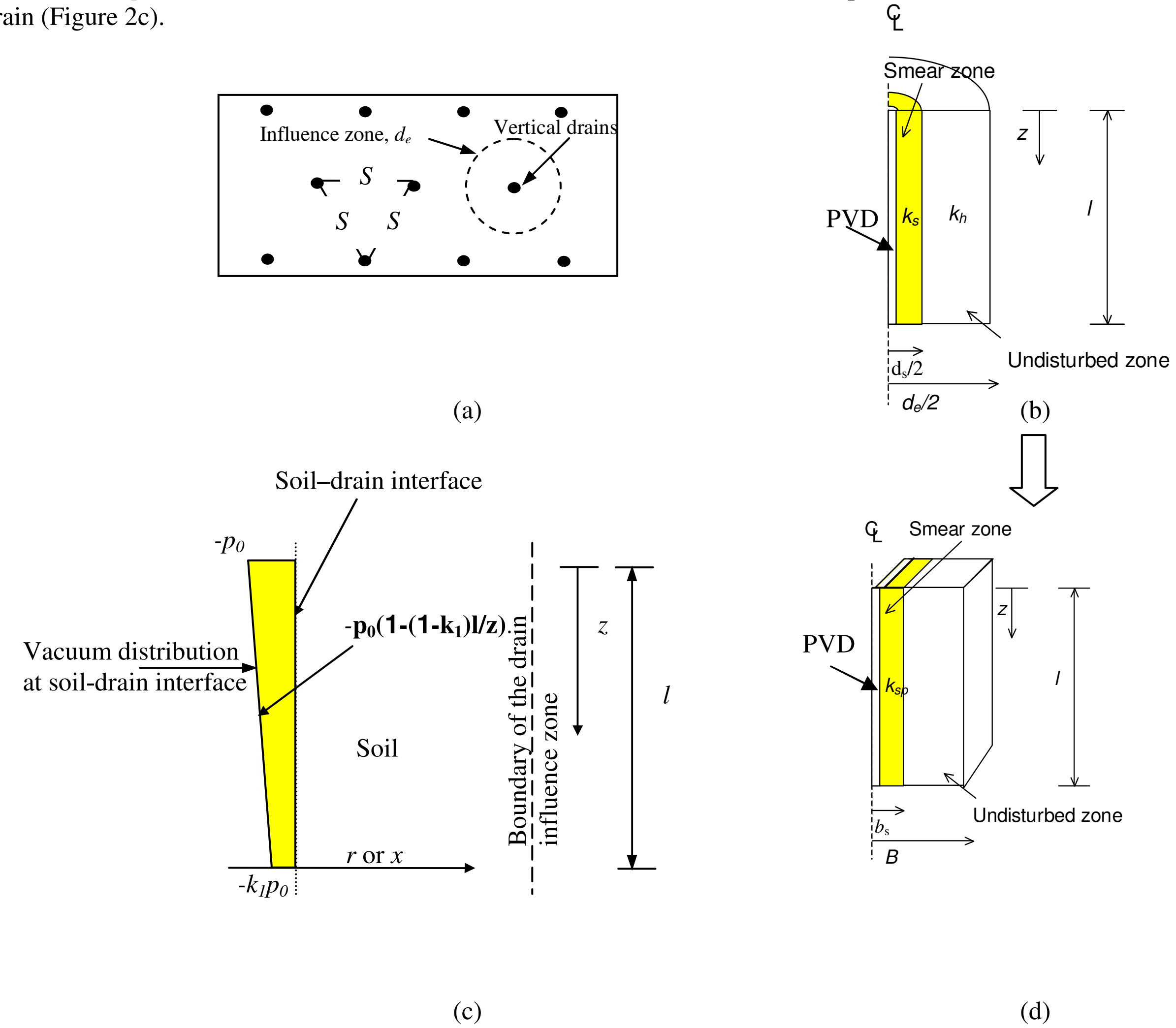


Figure 2 PVDs system (a) vertical drain installation layout, (b) axisymmetric unit cell, (c) negative pore water pressure along drain boundary based on laboratory test, (d) plane strain unit cell (Indraratna et al. 2005a).

3.1. SOLUTION FOR AXISYMMETRIC CONDITION

The unit cell theory is usually employed in the analysis of radial consolidation of soil around a single drain at the location of the embankment centreline where the lateral displacement is negligible (Barron, 1948; Hansbo, 1981). The solutions are based the nonlinear compressibility and permeability. Indraratna et al. (2005a) proposed a comprehensive analytical solution for a unit cell under vacuum condition considering the constant soil compressibility and soil permeability.

The dissipation rate of average excess pore pressure $\bar{u}_t$ at any time factor (T_h) can be derived based on Indraratna et al. (2005a) and expressed as:

$$\bar{u}_t = \left(\Delta p + p_0 \frac{(1+k_1)}{2}\right)\exp\left(\frac{-8T_h *}{\mu}\right) - p_0 \frac{(1+k_1)}{2} \quad (3)$$

In the above expression,

$$T_h * = P_{av} T_h \quad (4)$$

$$P_{av} = 0.5\left[1 + \left(1 + \Delta p/\sigma'_i + p_0(1+k_1)/2\sigma'_i\right)^{1-C_c/C_k}\right] \quad (5)$$

$$T_h = c_h t / d_e^2 \quad (6)$$

$$\mu = \ln\frac{n}{s} + \frac{k_h}{k'_h}\ln s - 0.75 \quad (7)$$

where, μ = a group of parameters representing the geometry of the vertical drain system and smear effect, $n = d_e/d_w$, s = d_s/d_w, d_e = equivalent diameter of cylinder of soil around drain = $2r_e$, d_s = diameter of smear zone and d_w = diameter of drain well= $2r_w$, k_h = average horizontal permeability in the undistrubed zone (m/s), k'_h = average horizontal permeability in the smear zone (m/s). Δp = preloading pressure, T_h is the dimensionless time factor for consolidation due to radial drainage, C_c = the compressibility indices, C_c = the permeability change index k_1 = ratio between vacuum pressure at the bottom and at the top of vertical drain, σ'_i = initial in-situ effective stress, p_0 = applied vacuum pressure at the top of the drain, c_k = permeability change index), z = depth, l = equivalent length of drain, q_w = well discharge capacity.

The average degree of consolidation (U_h) can now be evaluated conveniently by the equation:

$$U_h(\%) = \frac{1-\bar{u}}{1-\bar{u}_\infty} \times 100 \quad (8)$$

where, $\bar{u}_\infty$ can be calculated by Equations 3 to 7 when $t \rightarrow \infty$.

The settlement (strain) based on average degree of consolidation is defined by:

$$\rho = \rho_\infty U_h \quad (9)$$

where, ρ_∞ is the ultimate settlement when $t \rightarrow \infty$.

When the value of C_c/C_k approaches unity, the solution converges to that of Indraratna and Redana (2000) hence:

$$R_u = \left[1 + \left(p_0/\bar{u}_o\right)\left((1+k_1)/2\right)\right] \exp\left(-8T_h/\mu\right) - \left(p_0/\bar{u}_o\right)\left((1+k_1)/2\right) \quad (10)$$

3.2. SOLUTION FOR PLANE STRAIN CONDITION

The vacuum pressure distribution with a plane strain unit cell in compliance with the axisymmetric condition is shown in Figure 2d. The average excess pore pressure ratio $(R_{up} = \Delta p / \bar{u}_0)$ for plane strain condition is repeated by (Indraratna et al. 2005a):

$$R_{up} = \left[1 + \left(p_{0p}/\bar{u}_o\right)\left((1+k_1)/2\right)\right]\exp\left(-8T_h/\mu_p\right) - \left(p_{0p}/\bar{u}_o\right)\left((1+k_1)/2\right) \quad (11)$$

and

$$\mu_p = \alpha + \frac{k_{hp}}{k_{sp}}(\beta) + (\theta)\left(2lz - z^2\right),\ \alpha = \frac{2}{3}\frac{(n-s)^3}{n^2(n-1)},\ \beta = \frac{2(s-1)}{n^2(n-1)}\left[n(n-s-1) + \frac{1}{3}\left(s^2 + s + 1\right)\right],\ \theta = \frac{2k_{hp}}{Bq_w}\left(1 - \frac{1}{n}\right) \quad (12)$$

3.3. EQUIVALENT PLANE STRAIN CONVERSION PROCEDURE

It is impractical to conduct an analysis (even with the most powerful computers), employing an individual axisymmetric zone around every drain when there are thousands of vertical drains installed in large ground improvement projects. Therefore, two-dimensional (2D) plane strain conversion is most convenient in terms of computational effectiveness compared to an extensive 3-D analysis that can lead to considerably more time required for convergence, even with a smaller number of drains. Complex mesh descretization often leads to poor convergence. Therefore, a permeabiltiy matching procedure for vertical drain modeling transforms relevent parameters from the true axisymmetric condition (3D) to the *equivalent* plane strain (2-D) condition (Indraratna et al 2005a). The detailed derivation can be found in Indraratna et al. (2005a). The relationships between axisymmetric and plane strain permeability coefficients and vacuum pressure in this method are given below:

$$k_{hp}/k_h = \left[\alpha + \beta\, k_{hp}/k_{sp} + \theta\left(2lz - z^2\right)\right] / \left[\ln(n/s) + (k_h/k'_h)\ln(s) - 0.75 + \pi z(2l - z)(k_h/q_w)\right] \quad (13)$$

Now, by neglecting the smear effect and well resistance for relatively short vertical drains, the equivalent permeability in the undisturbed zone can be derived as:

$$k_{hp}/k_h \approx 0.67/[\ln(n) - 0.75] \quad (14)$$

By rearranging Eq. (13), the equivalent permeability within the smear zone can be determined by (Indraratna and Redana, 2000):

$$k_{sp}/k_{hp} = \beta / \left(k_{hp}\left[\ln(n/s) + (k_h/k'_h)\ln(s) - 0.75 + \pi z(2l - z)(k_h/q_w)\right]/k_h\right) - \alpha \quad (15)$$

The equivalent vacuum pressure under plane strain is:

$$p_{0p} = p_0 \quad (16)$$

where, μ = a group of parameters representing the geometry of the vertical drain system and smear effect, $n = d_e/d_w$, $s = d_s/d_w$, d_e = equivalent diameter of cylinder of soil around drain, d_s = diameter of smear zone and d_w = diameter of drain well, k_h = average horizontal permeability in the undistrubed zone (m/s), k'_h = average horizontal permeability in the smear zone (m/s). p_0= vacuum pressure at the top of the drain, q_w = discharge capacity, l = drain length, Subscript p denotes parameter for plane strain condition.

4. APPLICATION TO CASE HISTORY

4.1. EMBANKMENT DETAILS AND NUMERICAL SIMULATION

Figure 3 shows the shear strength and compressibility ratio of sub-soil layers. The minimum undrained shear strength (C_u) of topmost weathered clay is about 18 kPa at a depth of 1 m. This value decreases to 8-15 kPa in the very soft underlying clay layer, which is highly compressible. The compression ratio of the soft clay layer varies from 0.3-0.5, whereas the weathered crust has a compressibility ratio of about 0.2. The soil layer at the crust is much less compressible and highly overconsolidated (OCR ~ 2.4-26) due to compaction, aging, desiccation and oxidation process.

In this study, the soil profile at the site has been divided into 5 sublayers. The subsoil is relatively uniform, consisting of a top weathered crust (2 m depth) overlying very soft to soft dark gray layers extending from 2 m to 10.5 m depth. A 3.5 m thick medium clay layer underlies the soft clay layer. Underneath the medium clay layer, a light-brown stiff clay layer can be found at 14 m to 21 m depth. The groundwater level varies from 0.5 m to 1 m depth. The initial piezometric level is lower than the theoretical hydrostatic pore pressure at 6.0 m below the ground level, due to the excessive withdrawal of groundwater. Test embankment, TV2, was constructed on soft Bangkok clay with PVDs. The soft soil foundation was stabilised by 12 m long PVDs (Figure 4). A water and air tight LLDPE geomembrane liner was placed on top of the drainage system. The geomembrane liner was sealed by placing the edges of the bottom of the perimeter trench and covered with 300 mm layer of seal bentonite and submerged under water. The PVDs with 50mm equivalent drain diameter were installed in a triangular pattern with 1 m spacing.

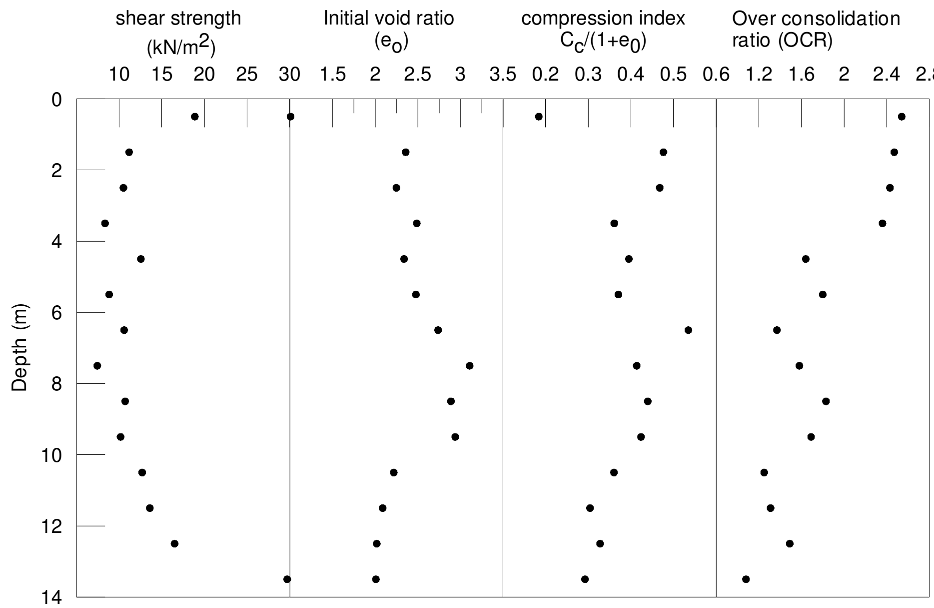


Figure 3 Average strength and compressibility indices (After Sangmala, 1997)

Surface settlement plates, subsurface multipoint extensometers, vibrating wire electrical piezometers and inclinometers were installed to monitor the behaviour of the embankments. The surface settlement plates were placed directly on top of the geomembrane. At the edges of each embankment, an inclinometer was installed. The vibrating wire piezometers were installed under the test embankment at 3 m depth intervals together with the sensors for the multipoint piezometer. At the dummy area, observation wells and stand pipe piezometers were also installed. The settlement, excess pore water pressure and lateral movement were monitored for about 150 days.

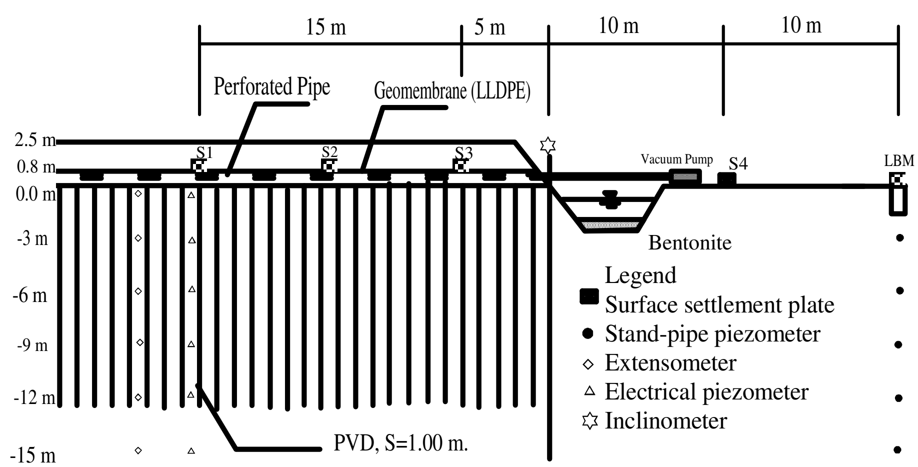
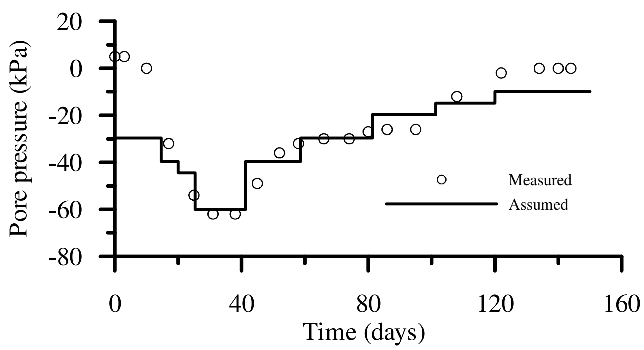
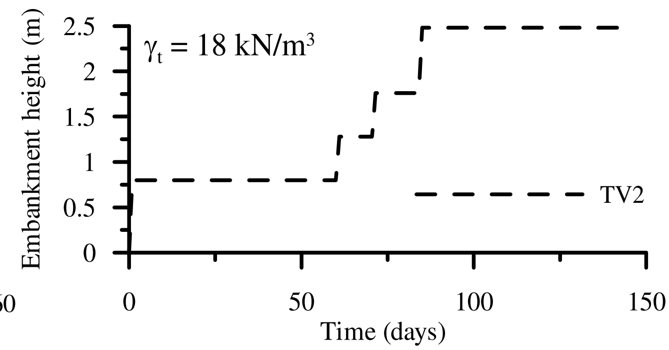


Figure 4 Cross section and location of monitoring system beneath Embankment TV2

A vacuum pump capable of generating 70 kPa suction pressure was employed. After 40 days, there were discrepancies between the measured and applied vacuum pressure. The suction head in the field decreased because of possible air leaks. Figure 5a illustrates the measured pore pressure at surface of Embankment TV2 and the assumed vacuum pressure value adjusted based on the field measurements. After 45 days of vacuum pressure application, the embankment load was applied in 4 stages upto a height of 2.5 m (the unit weight of surcharge fill equals to 18 kN/m^3). The stage of loading is illustrated in Figure 5b.





(a) (b)

Figure 5 (a) Measured and assumed pore pressure, (b) construction schedule.

4.2. SINGLE DRAIN ANALYSIS USING PROPOSED ANALYTICAL MODEL

In the field, at the embankment centerline, the condition of 1-D consolidation assumed in the proposed analytical model can be justified. The soil parameters, the in-situ effective stress and the soil permeability for soft Bangkok clay subsoils are shown in Table 1. The relevant soil properties were obtained from CK_oU triaxial tests (AIT, 1995). The slope of e-logk_h (C_k) can be determined by (Tavenas et al., 1983, Indraratna et al. 2005b):

$$C_k = 0.5e_0 \quad (17)$$

In the analysis, each subsoil layer was divided into 12 sub-layers (approximately 1m thick) to obtain a more accurate effective stress distribution with depth. The value of soil compressibility indices (C_c or C_r) are related to the actual stress state, where the current effective stress must be considered in association with the pre-consolidation pressure of soil at that particular depth (Indraratna et al., 1994). The values of k_h/k_s and d_s/d_w for this case study were assumed to be 2 and 6, respectively.

The embankment loading was simulated using an instantaneous loading at the upper boundary. Settlement predictions were carried out at the embankment centerline using Equations (3)-(10) and (17). At the beginning of the subsequent stage, the initial in-situ effective stress and soil permeability were calculated based on the final degree of consolidation of the previous loading stage. As the computation of consolidation settlement at the centerline is uncomplicated and follows the 1-D consolidation theory, the use of an EXCEL spreadsheet formulation for this purpose is sufficient. For the first stage loading, where the effective pre-consolidation pressure (p'_c) is not exceeded, the value of recompression index (C_r) may be used. In particular, the surface crust is heavily over-consolidated (up to about 2 m depth). Once p'_c is exceeded, the value of compression index (C_c) follows the normally consolidated line as indicated by the values in Table 1.

The following 4 models were analysed:

Model A: Application of 2.5m height of surcharge earth fill load alone (Figure 5b, i.e., no vacuum application),

Model B: Application of 2.5m height of surcharge and time dependent vacuum pressure simulating vacuum loss (leakage) after 1month according to the measured vacuum pressure under membrane (Figure 5a). However, the soil compressibility and permability is assumed to be constant (Hansbo, 1981). m_v was calculated based on the slope of the compression curve for a given effective stress range obtained from the oedometer test.

Model C: Similar to Model B. The nonlinear variation of soil compressibility and permeability proposed by the Authors is employed. OCR was used to determine the effective preconsolidation pressure when the in-situ effective stress is known. If the effective stress of soil is less than effective preconsolidation pressure, c_r will be used otherwise c_c.The detailed procedures of the calculation were mentioned elsewhere by Indraratna et al. (2005b).

Model D: Similar to Model C. There is no vacuum loss (no Leakage) after 1 month in contrast to Figure 5a.

Figure 6 compares the predicted surface centreline settlement with the measured data. As expected, the predicted results based on the proposed solutions agree well with the measured results, whereas the prediction based on the constant k_h overestimates settlement after 80 days, because, the actual soil permeability decreases considerably at higher stress levels. It was verified that the combined vacuum application and the PVDs system accelerates consolidation, while the vacuum pressure performs as an additional surcharge load. As shown in Figure 6, ‘no leakage’ condition (i.e. no vacuum loss) gains more settlements, whereas the prediction without any vacuum application yields less settlement. This is because vacuum pressure acts as an additional sucharge load without increasing excess pore pressure.The efficiency depends entirely on preventing airleaks and the distribution of vacuum pressure along the length of the drain. It is noted that the ultimate settlement can be obtained after 170 days.

Table 1 Selected soil parameters for single drain analysis

Depth (m)	C_r	C_c	k_h ($\times 10^{-9}$ m/s)	e_0	γ (kN/m^3)	p'_c (kPa)
0.0-2.0	0.06	0.37	30.1	1.8	16	58
2.0-8.5	0.08	1.6	12.7	2.8	15	45
8.5-10.5	0.05	1.7	6.02	2.4	15	70
10.5-13.0	0.03	0.95	2.56	1.8	16	80
13.0-15.0	0.01	0.88	0.60	1.2	18	90

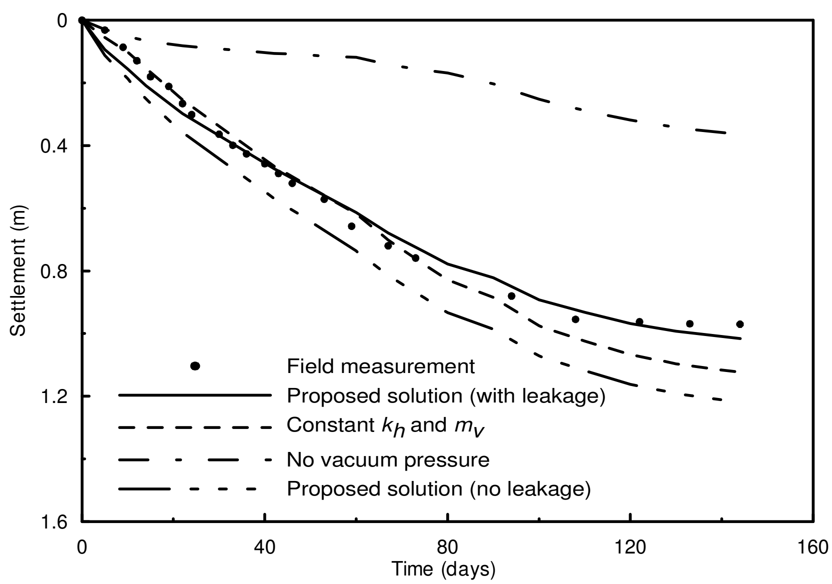


Figure 6 Surface settlement predictions at the centerline

4.3. MULTI-DRAIN ANALYSIS USING EQUIVALENT PLANE STRAIN TECHNIQUE

The numerical analysis was based on the modified Cam-Clay model (Roscoe and Burland, 1968) and the equivalent plane strain Equations 13-16 developed by the author, which are incorporated in the finite element code, *ABAQUS* (Hibbitt, Karlsson, and Sorensen, 2004). The adopted parameters of 5 subsoil layers are listed in Table 2. The critical-state soil properties were determined by Asian Institute of Technology (AIT, 1995). The finite element mesh, which contains 8-node bi-quadratic displacement and bilinear pore pressure elements, is shown in Figure 7. Because of symmetry, it was sufficient to consider one half of the embankment for the numerical analysis.

The following 4 models were numerically examined under the plane strain multi-drain analysis:

Model A: Conventional analysis (i.e., no vacuum application),
Model B: Vacuum pressure is adjusted according to field measurement and reduces linearly to zero at the bottom of the drain (k_1= 0),
Model C: No vacuum loss (i.e. vacuum pressure was kept constant at -60kPa after 40 days); vacuum pressure varies linearly to zero along the drain length (k_1= 0),
Model D: Constant time-dependent vacuum pressure throughout the layer (k_1=1)

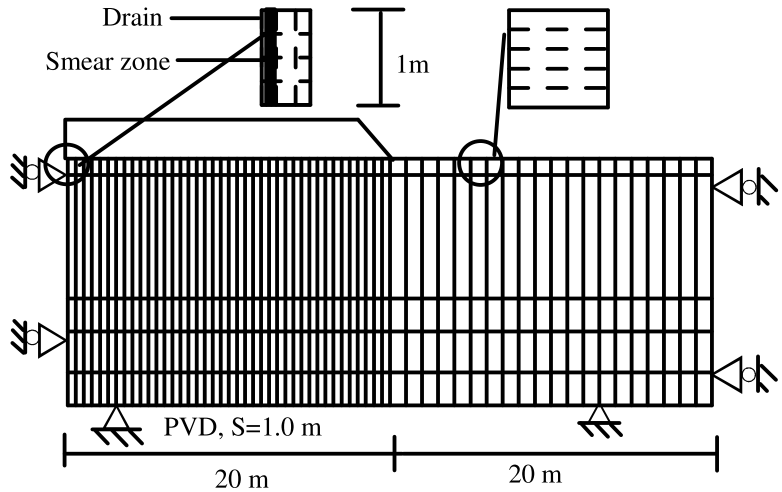


Figure 7 mesh discretization (Indraratna and Rujikiatkamjorn, 2004).

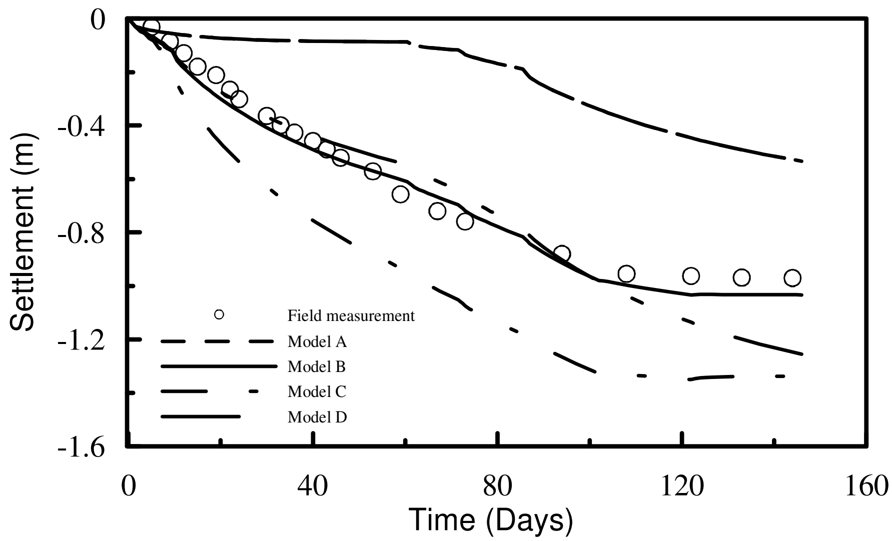
Table 2 Selected soil parameters in plane strain finite element analysis.

Depth (m)	λ	κ	ν	e_0	γ kN/m^3	k_v 10^{-9} m/s	k_h 10^{-9} m/s	k'_h 10^{-9} m/s	k_{hp} 10^{-9} m/s	k'_{hp} 10^{-9} m/s
0.0-2.0	0.3	0.03	0.30	1.8	16	15.1	30.1	89.8	6.8	3.45
2.0-8.5	0.7	0.08	0.30	2.8	15	6.4	12.7	38.0	2.9	1.46
8.5-10.5	0.5	0.05	0.25	2.4	15	3.0	6.0	18.0	1.4	0.69
10.5-13.0	0.3	0.03	0.25	1.8	16	1.3	2.6	7.6	0.6	0.30
13.0-15.0	1.2	0.10	0.25	1.2	18	0.3	0.6	1.8	0.1	0.07

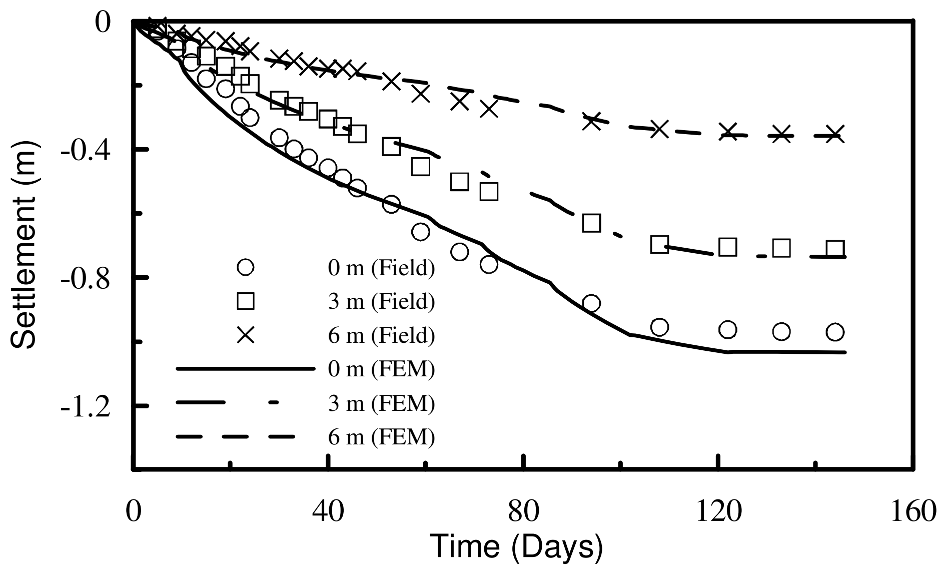
The predicted and measured surface settlement and the subsurface settlement at the centreline of the embankment are shown in Figure 8. Clearly, Model B predicts the settlement of this embankment very well. Comparing all the different vacuum pressure conditions, Models A and D give the lowest and highest settlement, respectively. A vacuum application combined with a PVD system can accelerate the consolidation process significantly. With vacuum application most of primary consolidation is achieved around 120 days, whereas conventional surchage (same equivalent pressure) requires more time to reach primary consolidation (after 150 days). It is also apparent that greater settlement can be attained, if any loss of vacuum pressure can be maintained (Model C). Field measurements indicate that the pattern of vacuum distribution and the extent of vacuum loss directly influences soil consolidation behaviour. The accuracy of the numerical predictions is governed by correct assumptions of the time-dependent vacuum pressure distribution with soil depth.

The predicted and measured excess pore pressures are shown in Figure 9a. The field data plot is closest to Model B, suggesting that the writer's assumption of linearly decreasing vacuum pressure along the drain length is justified. Excess pore pressure generated from the vacuum application is significantly less than from the conventional case, which enables the rate of construction of a vacuum-assisted embankment to be higher than conventional construction.

The predicted and measured lateral displacements (at the end of surcharge construction) are shown in Figure 9b. The observed lateral displacements do not seem to agree with the vacuum pressure models. In the middle of the very soft clay layer (4-5m deep), the predictions from Models B and C are closest to the field measurements. Nearer to the surface, the field observations do not agree with the 'inward' lateral movements predicted by Models B and C. The discrepancy between the finite element models and the measured results is more evident in the topmost weathered crust (0-2 m). Vacuum preloading generates an inward lateral movement of soft soil towards the embankment centreline (i.e. negative displacement in Figure 9b) and minimises the risk of bearing capacity failure due to rapid embankment. However, this inward movement may cause tension cracks in adjacent areas, hence, lateral movement at the borders of the embankment and its affects on adjacent structures should be carefully monitored. It is noted that consolidation time was decreased by 3 months and height of embankment was reduced from 4m to 2m, to achieve the same amount settlement by conventional surcharge alone.

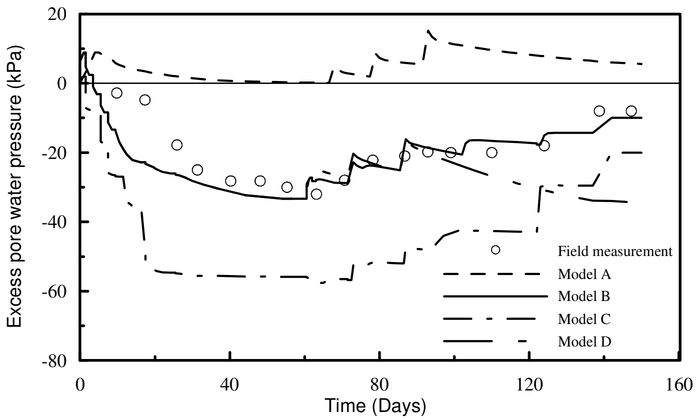


(a)

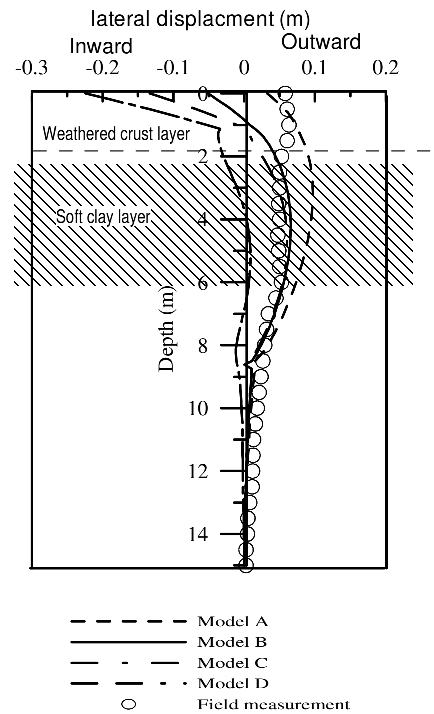


(b)

Figure 8 Measured and predicted settlements (a) surface settlement of embankment TV2, (b) consolidation settlement of embankment TV2 (Model B).



(a)



(b)

Figure 9 Pore pressure and lateral movement (a) variation of excess pore water pressure at 3m below the surface and 0.5m away from centerline, (b) Calculated and measured lateral displacements distribution with depth at the embankment toe (20m away from the centreline).

5. CONCLUSIONS

A system of prefabricated vertical drains (PVDs) combined with vacuum preloading is an effective method for accelerating soil consolidation. In this study, a revised analytical model for vacuum preloading incorporating the compressibility indices (C_c and C_r) was proposed, and the variation of horizontal permeability coefficient (k_h) was represented by the e-logk_h relationship. The solution was employed to evaluate the performance of soft clay beneath embankment TV2 using spreadsheet software. In addition, a multi-drain equivalent plane strain model was executed to evaluate the performance of soft clay beneath embankment TV2. The settlements with depth, excess pore water pressures, and lateral movements of the soft clay foundation were analysed and compared with field observations when considering the actual field condition such as the varations of vacuum pressure, soil compressibility and permeabiltiy. It was shown that the assumption of vacuum pressure distribution along the drain length is realistic. The effectiveness of vacuum system depends on the air leak protection in the field.

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