APPLICATION OF VACUUM CONSOLIDATION FOR SUSTAINABLE TRANSPORT INFRASTRUCTURE

Cholachat Rujikiatkamjorn
Senior Lecturer, Centre for Geomechanics and Railway Engineering, University of Wollongong, ARC Centre of Excellence in Geotechnical Science and Engineering, Australia
(email: cholacha@uow.edu.au)
Buddhima Indraratna
Professor, Director, Centre for Geomechanics and Railway Engineering, University of Wollongong, ARC Centre of Excellence in Geotechnical Science and Engineering, Australia
(email: indra@uow.edu.au)

Abstract

A system of vertical drains combined with vacuum preloading to accelerate soil consolidation by promoting radial flow is an effective method. In this paper, the analytical modelling of vertical drains incorporating vacuum preloading in an axisymmetric condition is proposed. The distribution of vacuum pressure along the drain and variations in volume compressibility, and permeability of soil based on large scale laboratory observations are included in the proposed solutions. These analytical predictions are compared with a case history taken from Thailand where the entire embankment was stabilised with vertical drains and subjected to vacuum preloading. This analysis employing the writer's model indicates an improvement in the predictions in relation to the field observations. The data show that the efficiency of the prefabricated vertical drains depends on the magnitude and distribution of vacuum pressure, and the extent to which air is prevented from leaking.

Keywords: Consolidation, Vertical Drains, Vacuum preloading

1. Introduction

Many coastal regions of Australia and Asia contain very soft clays, which have unfavourable soil properties such as, low bearing capacity and high compressibility. In the absence of adequate ground improvement, excessive settlement and lateral movement may affect the stability of buildings, port and transport infrastructure built on such soft ground (Indraratna & Chu 2005). The constraints of limited space, tight construction schedules, environmental and safety issues, maintenance costs and the longevity of earth structures have continued to demand unfailing innovation in the design and construction of essential infrastructure on soft clays. Preloading method in conjunction with prefabricated vertical drains (PVD) to improve the performance of soft clays is usually an economical solution (Hansbo 1981; Indraratna et al. 2005). However, there can be a significant delay in consolidation time due to the very low soil permeability, low soil shear strength and the lack of efficient drainage in very deep soil layers. The installation of PVD, followed by the application of vacuum pressure would accelerate the dissipation of pore water pressure (Bergado et al. 2002; Indraratna et al. 2005). It is expected that with an airtight membrane placed over the surface, the applied vacuum pressure will propagate along the ground surface and down the PVDs, consolidating and strengthening the soil within the PVD stabilised zone (Chu et al. 2000; Carter et al 2005). Also, the thickness of the surcharge fill may be reduced by several meters, if sufficient vacuum pressure (less than atmospheric pressure) is applied and sustained, thereby reducing the risk of undrained bearing capacity failure due to the rapid construction of a high embankment. Once the soil has experienced consolidation settlement (increased shear strength), the post-construction soil settlement will be significantly less, thereby eliminating any risk of instability of the overlying infrastructure (Shang et al. 1998). Therefore, ground improvement provided by prefabricated vertical drains (PVD) combined with vacuum pressure may be an economically attractive alternative in deep soft clay sites.

Currently, two types of vacuum preloading systems can be utilised in the field.

A. Membrane system (e.g. Menard Drain System)

After installation of PVDs and placement of sand blanket, horizontal drains will be installed in the transverse and longitudinal directions. Afterwards, these drains can be connected to the edge of a peripheral bentonite slurry trench, which is typically sealed by an impervious membrane (Fig. 1). The membrane is then laid over the sand blanket in order to ensure an airtight region above the PVDs. The vacuum pumps are then connected to the discharge system extending from the trenches. A major advantage of this system is that the vacuum head propagates along the soil surface and down the PVDs within the airtight domain, inducing rapid dissipation of pore water pressure towards the PVDs and the surface. However, an obvious drawback is that the efficiency of the entire system depends on the ability of the membrane to prevent any air leaks to sustain a sufficient suction head over a significant period of time.

B. Membraneless system (e.g. Beau Drain System)

When an area has to be separated into a number of sections to assist the installation of the membrane, the vacuum preloading can only be performed one section after another. One way of overcoming this problem is to attach the vacuum system directly to each individual PVD using a tubing system. In this arrangement, each individual drain is connected directly to the drain

collector (Fig. 2), where each drain acts independently. However, the requirement of extensive tubing for hundreds of drains can affect the installation time and cost (Seah, 2006).
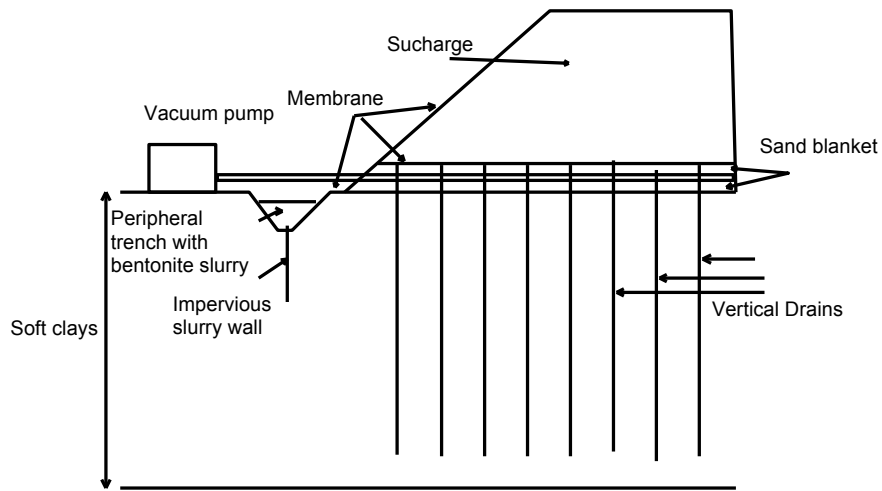

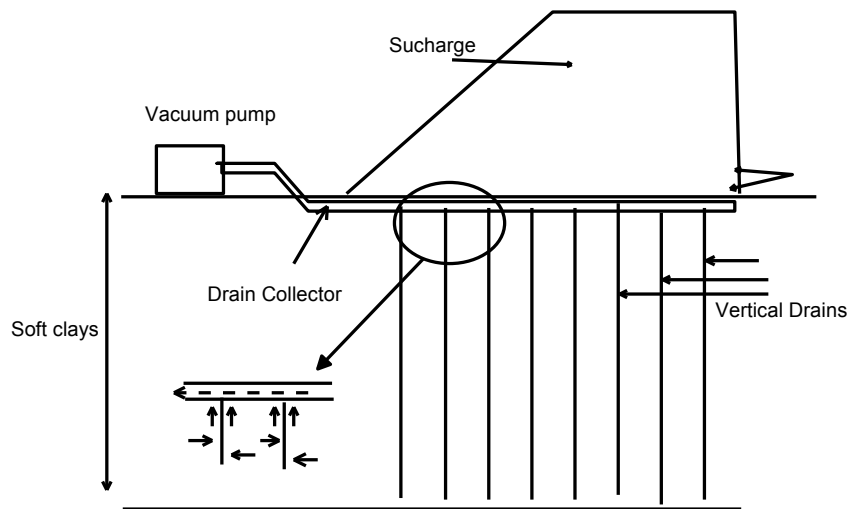Figure 1 Schematic diagram of PVDs incorporating preloading system for Membrane system

Figure 2 Schematic diagram of PVDs incorporating preloading system for Membraneless system

In this paper, an analytical solution for radial consolidation considering linear permeability variation in the smear zone is proposed. A selected case history from Thailand is discussed and analysed, and the predictions are compared with the available field data.

2. Theoretical considerations

In order to analyse the behaviour of vertical drains, the unit cell theory representing a single drain surrounded by a soil annulus in axisymmetric condition (3D) was proposed by Hansbo (1981). Since the small strain theory is employed in the analysis, a constant coefficient of volume compressibility (m_v) and a constant coefficient of horizontal permeability (k_h) were assumed for a given stress range. In contrast, for a relatively large applied stress range, it is known that both soil permeability and soil volume compressibility coefficients decrease during the consolidation process. The stress state in relation to preloading (surcharge) and effective preconsolidation pressure is essential to predict the actual. In this paper, the e-logσ' relationship is used to determine the compressibility indices (C_c and C_r), and the e-logk_h relationship is used to represent permeability variation. In contrast, in the conventional radial consolidation (Hansbo, 1981), the parameters m_v and k_h were not changed as a function of the void ratio for a given stress range, but they were changed for each stage of loading (e.g. multi-stage loading).

The main assumptions made in the analysis are summarised below:

- The soil is fully saturated and homogeneous, and laminar flow through the soil (Darcy's law) is adopted.

- The permeability ratio between disturbed and undisturbed zone is assumed to be constant during consolidation.

- During the consolidation process, at a given depth, the relationship between the average void ratio and the logarithm of average effective stress in the normally consolidated range (Fig. 3a) can be expressed by: $\bar{e} = e_0 - C_c \log(\sigma'/\sigma'_i)$. If the current vertical effective stress (σ') is smaller than the effective preconsolidation stress (p'_c), the recompression index (C_r) is used instead of C_c for the overconsolidated range.

- In radial drainage, the horizontal permeability of soil decreases with the average void ratio (Fig. 3b). The relationship between these two parameters can be commonly found by $\bar{e} = e_0 + C_k \log(k_h/k_{hi})$.

Based on the above assumptions, the expression for excess pore pressure ratio can be expressed by:

$$R_u = \left(1 + \frac{p_0}{\Delta p}\frac{(1+k_1)}{2}\right)\exp\left(-4[1+f]\frac{T_h}{\mu_{ax}}\right) - \frac{p_0}{\Delta p}\frac{(1+k_1)}{2} \tag{1}$$

$$P = P_{av} = 0.5[1+f] \tag{2}$$

$$f = \left(1 + \Delta p/\sigma'_i + p_0(1+k_1)/2\sigma'_i\right)^{1-C_c/C_k} \tag{3}$$

$$\mu_{ax} \approx \left[\ln\left(\frac{n}{s}\right) + \frac{k_{h,ax}}{k_{s,ax}}\ln(s) - \frac{3}{4}\right] \tag{4}$$

where, p_0 = applied vacuum pressure at the top of the drain; Δp=preloading pressure; k_1=ratio between vacuum pressure at the bottom and at the top of vertical drain; T_h =dimensionless time factor for horizontal drainage, $T_{h,ax} = \dfrac{c_h t}{d_e^2}$ or $T_{h,ps} = \dfrac{c_h t}{4B^2}$
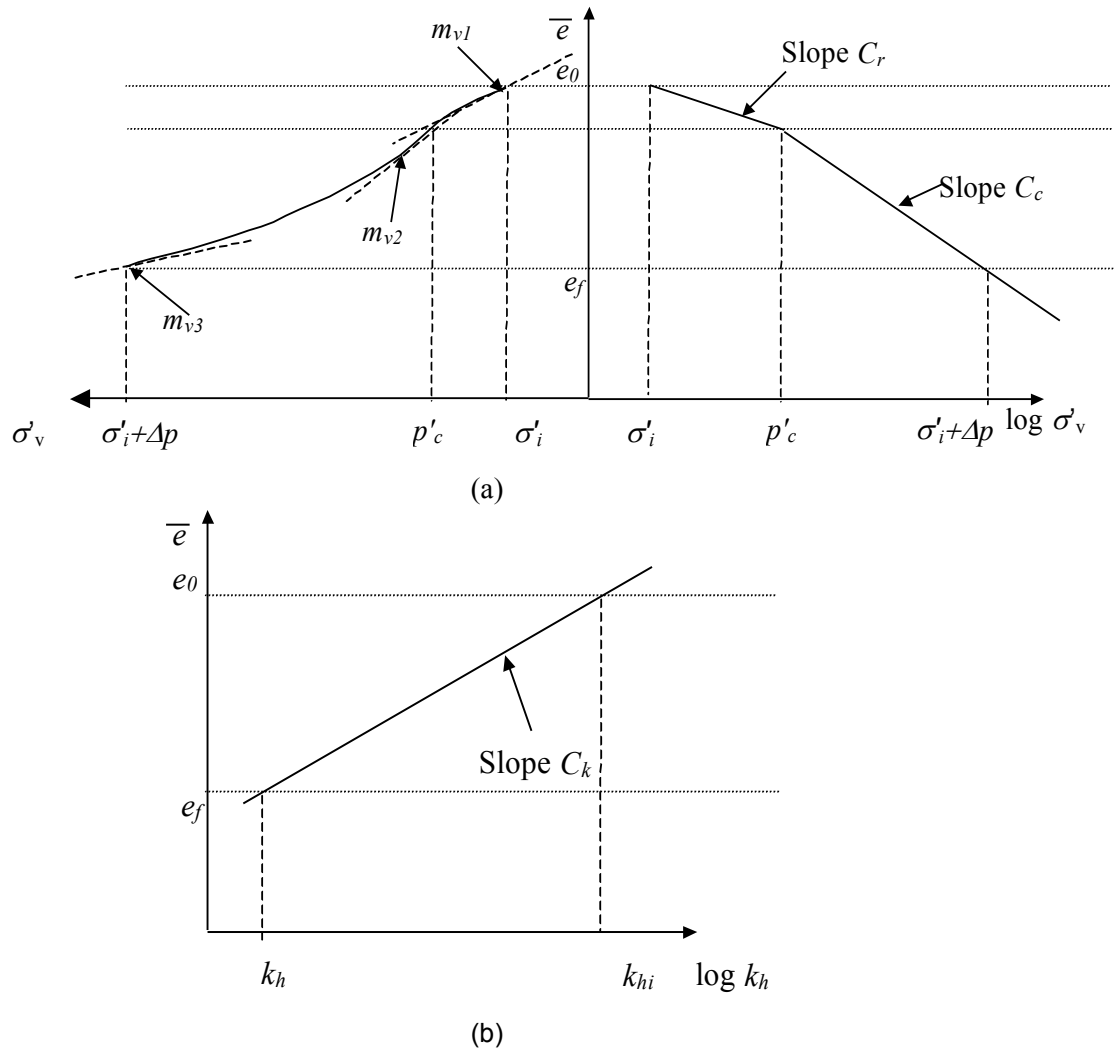

(a)

(b)

Figure 3 (a) Compression during preloading and (b) Semi-log permeability-void ratio relationship

3. Site Location and Embankment Characteristics

The Second Bangkok International Airport or Suvarnabhumi Airport is located about 30km from the city of Bangkok. The exact location of this site in the Samut Prakan province is shown in Fig. 4. In the past, the site was occupied by ponds for fish farming and used for agricultural purposes. The area is often flooded during the wet season and the soil generally retains very high moisture content. Therefore, soft clays, mainly of marine or deltaic origin, often present considerable construction problems, which require ground improvement techniques to prevent excessive settlement and lateral movement.
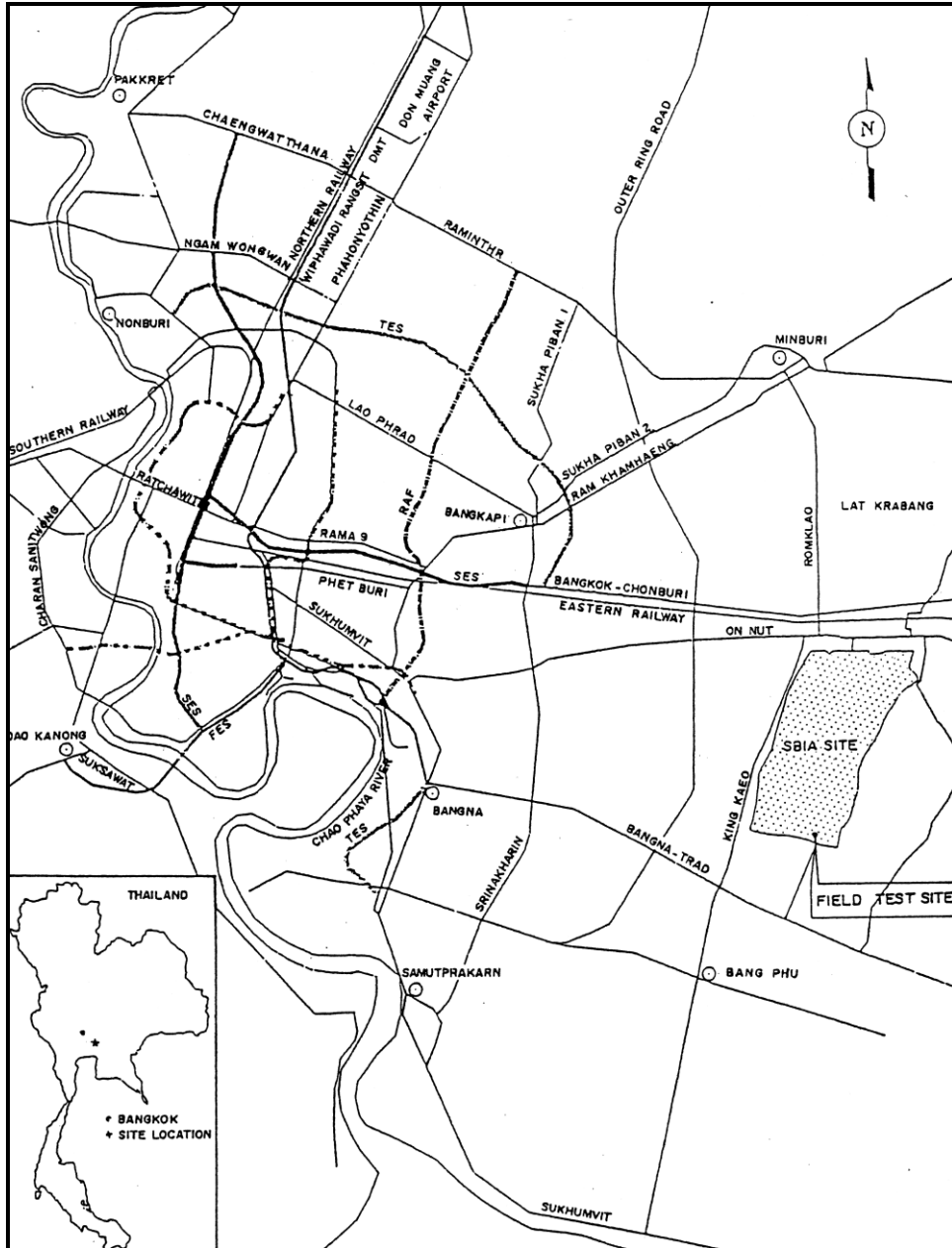
Figure 4 Location of the Second Bangkok International Airport

The subsoil profile at the site consists of 2 m thick weathered crust (highly overconsolidated clay) overlying a very soft to medium clay, which extends about 10m below the ground surface. Underneath the medium clay layer, a light-brown stiff clay layer is found at a depth of 10-21m. The ground-water level fluctuates between 0.5 and 1.5m below the surface. The soil profile with the strata properties is illustrated in Fig. 5. The water content of the very soft clay layer

varies from 80 to 100%, whereas in the lower parts of the stratification (10-14m) it changes from 50 to 80%. The plastic limits and liquid limits of the soil in each layer are similar and found to be in the range of 80 to 100% and 20 to 40%, respectively (Fig. 5).

At this airport site, several trial embankments were constructed, two of which, TV1 and TV2, were built with PVDs and vacuum application (Fig. 6). Total base area of each embankment was 40×40 m² (Asian Institute of Technology, 1995). Figures 7 and 8 present the cross sections and positions of the field instruments for embankments TV1 and TV2, respectively. For TV1 (Fig. 7), 15m long PVDs with a hypernet drainage system were installed, and for TV2 (Fig. 8), 12m long PVDs with perforated and corrugated pipes wrapped together in non-woven geotextile were used. The drainage blanket (working platform) was constructed with sand 0.3 m and 0.8 m for embankments TV1 and TV2, respectively with an air and water tight Linear Low Density Polyethylene (LLDPE) geomembrane liner placed on top of the drainage system. This liner was sealed by placing its edges at the bottom of the perimeter trench and covered with a 300mm layer of bentonite and then submerged with water. The array of instrumentation of the embankments includes piezometers, surface settlement plates, multipoint extensometers, inclinometers, observation wells and benchmarks.

Table 1 shows the vertical drain properties for embankment TV1 and TV2. The PVDs were installed in a triangular pattern at a spacing of 1m. The type of drain installed in both embankments is Mebra (MD-7007) drains (100 mm x 3 mm) having grooved polypropylene channels wrapped in a nonwoven polypropylene filter. In these embankments, the drains were installed using a mandrel, which was continuously pushed into the soil using a static weight (in lieu of vibration). This method was employed to reduce the extent of smear zone as much as possible.
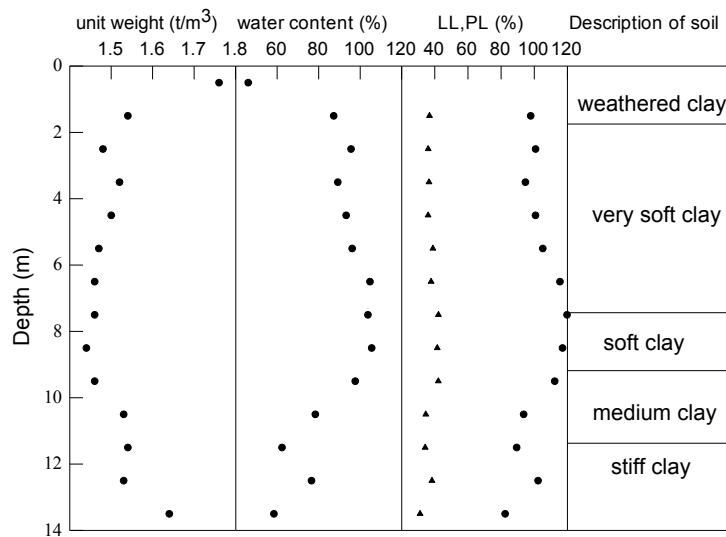

Figure 5. General soil profile and properties at Second Bangkok International Airport

Table 1Vertical drain parameters

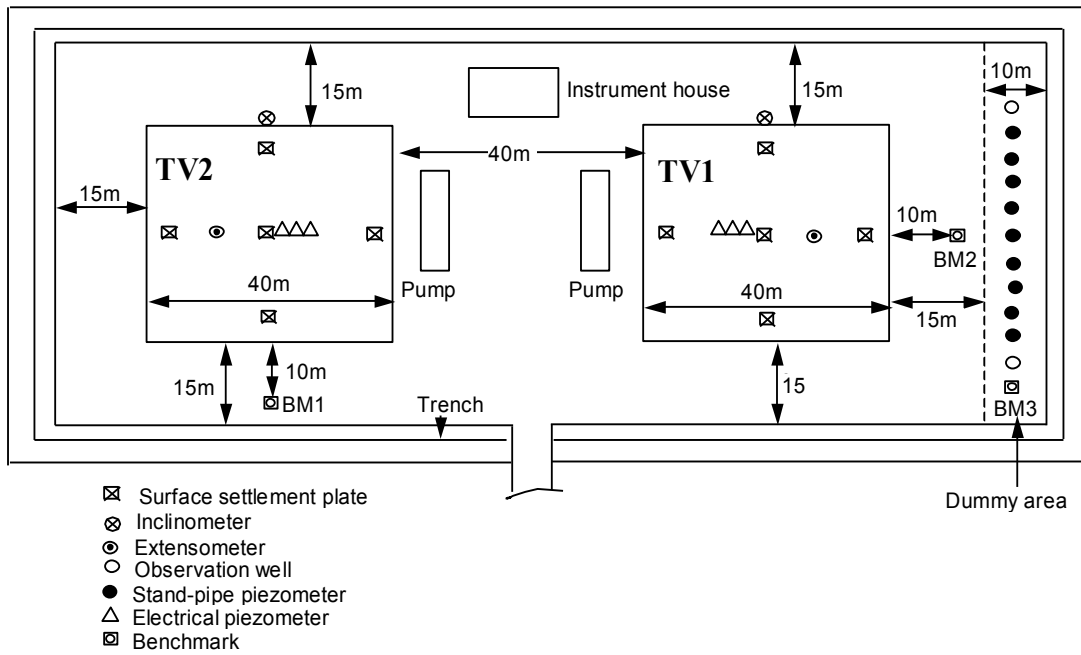| Spacing, S	1.0 m (triangular)
Diameter of drain, d_w	50 mm
Discharge capacity, q_w	50 m³/year (per drain)

Length of vertical drain	15 m for TV1 and 12 m for TV2

Surface settlement plate
⊗ Inclinometer
◉ Extensometer
○ Observation well
● Stand-pipe piezometer
△ Electrical piezometer
▣ Benchmark

Figure 6 Site plan for the test embankments at Second Bangkok International Airport

Legend
Surface settlement plate
○ Stand-pipe piezometer
◇ Extensometer
△ Electrical piezometer
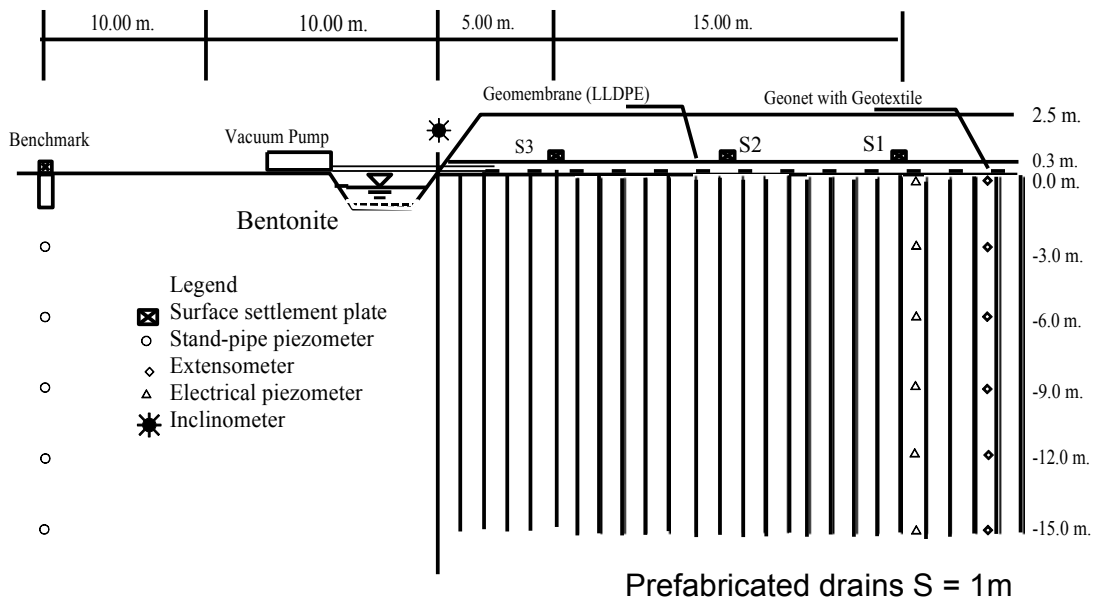✷ Inclinometer

Prefabricated drains S = 1m

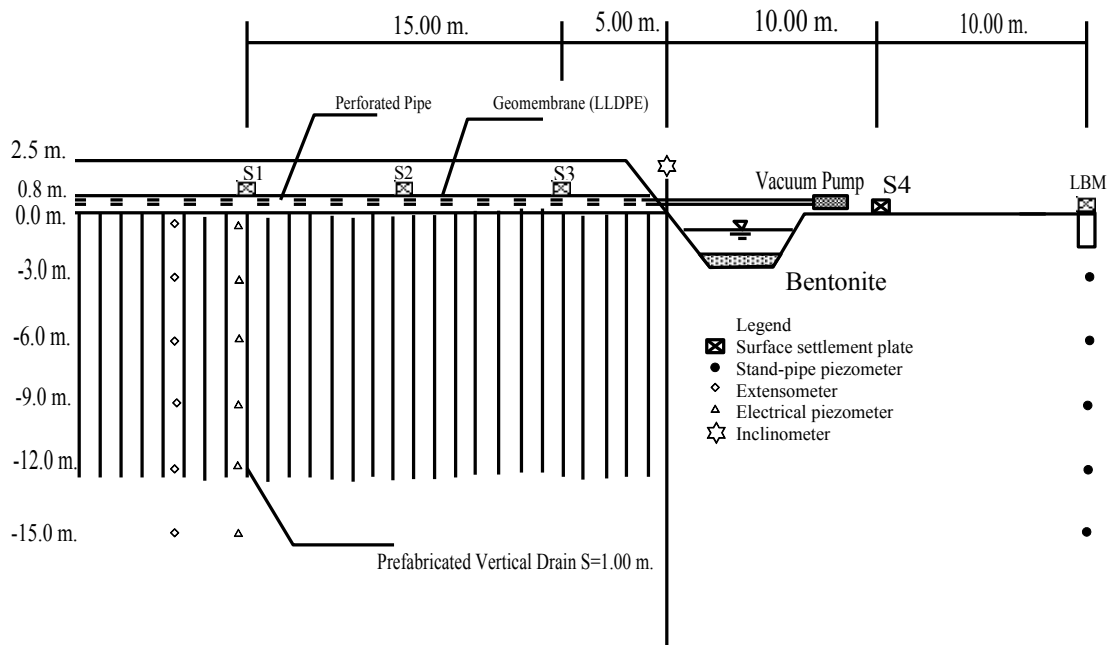Figure 7 Vertical cross section at embankment TV1 with instrumentation locations

Figure 8 Vertical cross section at embankment TV2 with instrumentation locations

achieved using the available vacuum equipment. This pressure is equivalent to a fill height of 4m. After 45 days of vacuum application, the surcharge load was applied in 4 distinct stages upto 2.5m high (the unit weight of surcharge fill equals to 18 kN/m^3) as illustrated in Fig. 9. Field instrumentations including surface settlement plates, subsurface multipoint extensometers, vibrating wire electrical piezometers and inclinometers were installed. In addition, around the dummy area, observation wells and stand-pipe piezometers were installed. The settlement, excess pore water pressure, and lateral movement, were observed for 5 months. During the application of vacuum pressure, it was found that the suction head transmitted to the soil could not be maintained at the same level throughout the vacuum pressure application period as shown in Figure 10. This fluctuation has not been uncommon in various soft clays, and has often been associated with air leaks through the surface membrane or the loss of suction head beneath the certain depth for long PVD. Intersection of natural macro-pores with drains at various depths can also lead to suction head drops.
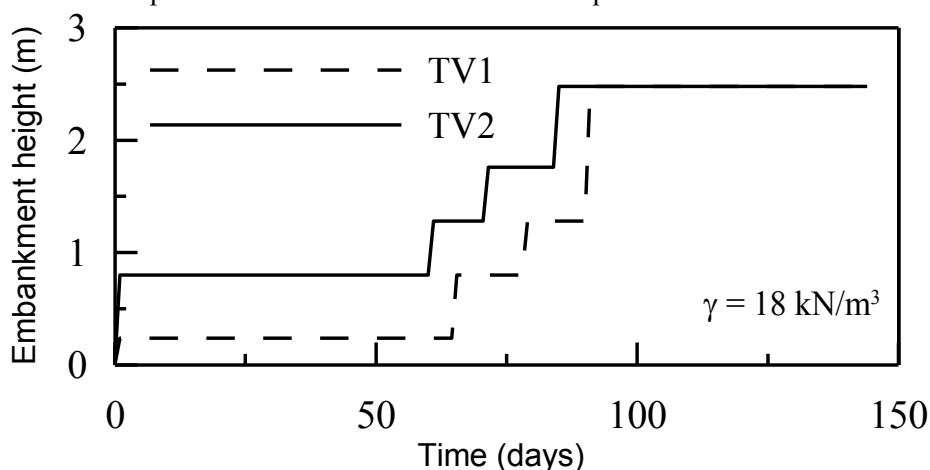

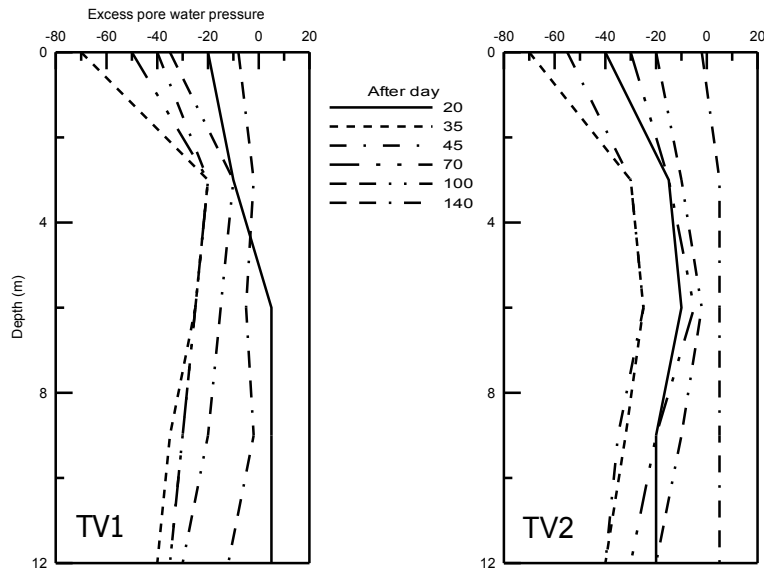Figure 9 Multi-stage loading for embankments TV1 and TV2

Figure 10 Observed variation of pore pressure with time and depth in the field for two embankments TV1 and TV2

4. Single Drain Analysis at Embankment Centreline by Proposed Analytical Model

The embankment loading was simulated by assuming an instantaneous loading at the upper boundary. Settlement predictions were carried out at the embankment centerline using the writer's analytical model (e.g. Eqs. 1-4). At the beginning of the subsequent stage, the initial in-situ effective stress and initial coefficient of horizontal consolidation (c_{hi}) were calculated based on the final degree of consolidation of the previous loading stage. As the computation of consolidation settlement at the centerline (zero lateral displacements) is straightforward and follows the 1-D consolidation theory, the use of an EXCEL spreadsheet formulation for this purpose is most convenient. The value of soil compressibility (C_c or C_r) in association with the correct working effective stress plays a very important role for predicting settlement. For Stage 1 loading, where the effective preconsolidation pressure (p'_c) is not exceeded, the value of recompression index (C_r) may be used. In particular, the surface crust is heavily overconsolidated (upto 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 2. The time-dependent vacuum pressure is assumed to vary linearly to zero along the drain length (k_l= 0, Fig. 10).

Figures 11 and 12 compare predicted surface centreline settlement with the measured data for Embankment TV1 and TV2, respectively. As expected, the predicted results based on the writer's solutions agree well with the measured results, whereas the prediction based on the constant k_h and smear zone overestimates settlement after 80 days, because, the actual soil permeability decreases significantly at higher stress levels. It was verified that the combined vacuum application and the PVD system accelerates consolidation, while the vacuum pressure acts as an additional surcharge load. As shown in Figs. 11 and 12, 'no leakage' condition gives more settlements, whereas the prediction without any vacuum application gives less settlement. It indicates that 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 2 Selected soil parameters for single drain analysis

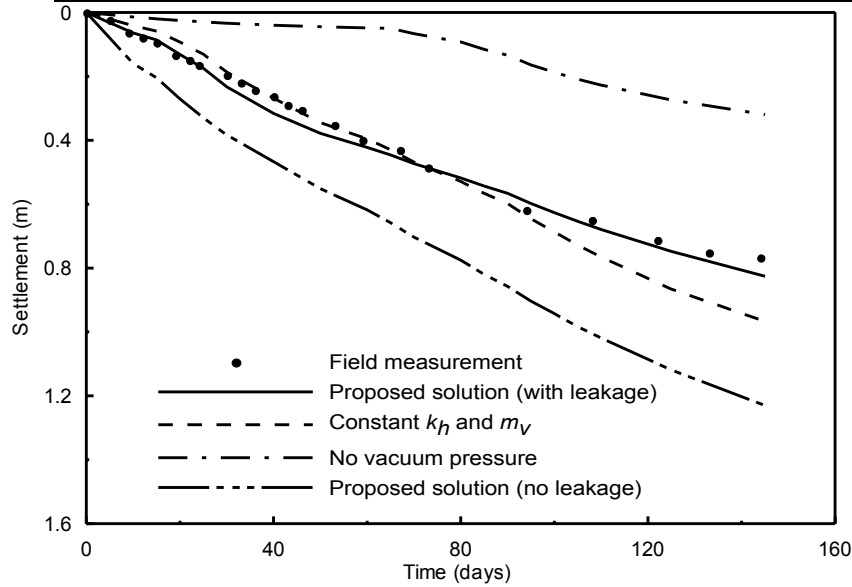| Depth (m)	C_r	C_c	k_h ($\times 10^{-9}$ m/s)	e_0	γ (kN/m³)	p'_c (kPa)
0.0-2.0	0.06	0.37	30.1	1.8	16	58
2.0-8.5	0.08	1.6	12.7	2.8	15	45
8.5-10.5	0.05	1.7	6.02	2.4	15	70
10.5-13.0	0.03	0.95	2.56	1.8	16	80
13.0-15.0	0.01	0.88	0.60	1.2	18	90

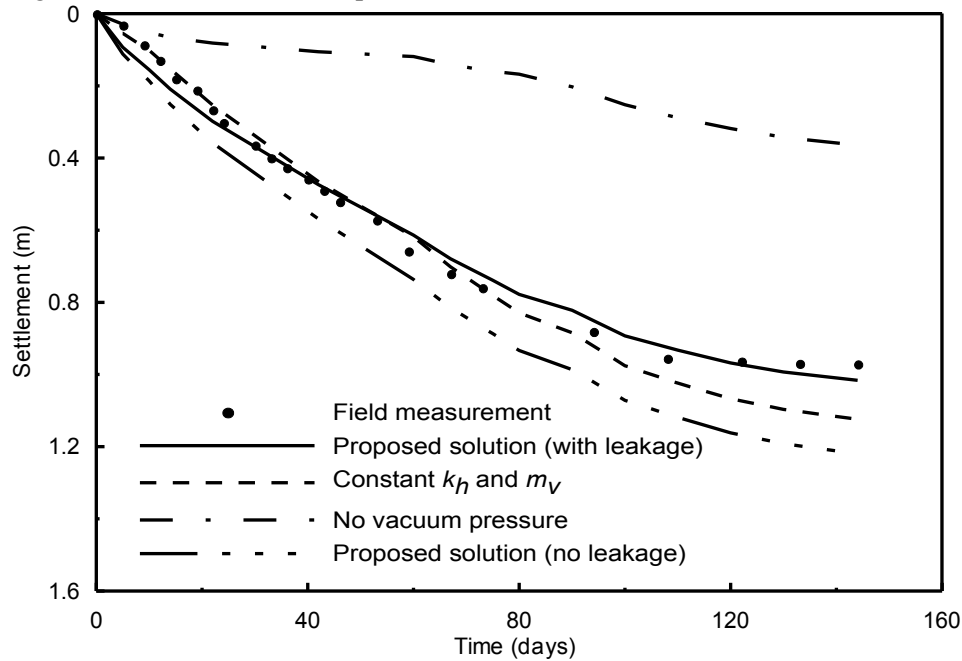Figure 11 Surface settlement predictions at the centerline of Embankment TV1

Figure 12 Surface settlement predictions at the centerline of Embankment TV2

5. Conclusions

The performance of two test embankments stabilised with vertical drains subjected to vacuum loading was investigated using an analytical analysis based on the writer's axisymmetric model. The analytical predictions of pore pressures and settlements can be easily obtained at the centreline of embankment where the assumed 'no lateral strain' condition can be justified. It has been demonstrated that the actual time-dependent vacuum pressure and the vacuum pressure distribution along the drain length (i.e. linear distribution) play a very important role in predicting the settlement and excess pore pressure variation.

Vacuum pressure may diminish with depth due to various practical limitations such as improper sealing and due to the nature of soil conditions (e.g. presence of fissures and macro-pores), as observed in field studies. Therefore, the assumption of diminishing suction values along the drain depth is justified in the finite element modelling. Vacuum must be monitored in the field for proper numerical simulations and practical designs. Further comprehensive study with 'instrumented PVDs' in the field is necessary to provide further insight to the vacuum pressure distribution with depth in the stabilisation of soft clays.

Acknowledgement

References

Bergado, D.T., Balasubramaniam, A.S., Fannin, R.J. and Holtz, R.D. 2002. Prefabricated Vertical Drain (PVD) in Soft Bangkok Clay: A Case of NBIA Project, Canadian Geot. Journal, 39: 304-315.

Carter, J.P., Chai, J.C. and Hayashi, S. (2005). Ground Deformation Induced by Vacuum Consolidation. ASCE Journal of Geotechnical and Geoenvironmental Engineering. 131(12): 187-192.

Chu, J., Yan, S.W. and Yang, H. 2000. Soil improvement by the vacuum preloading method for an oil storage station. Geotechnique, 50(6): 625-632.

Hansbo, S. 1981. Consolidation of fine-grained soils by prefabricated drains. Proc. 10th Int. Conf. on Soil Mechanics and Foundation Engineering, Stockholm, Sweden, 3: 677-682.

Indraratna B., and Chu, J. 2005. Ground Improvement – Case Histories Book (Volume 3), Elsevier, London 1115p.

Indraratna, B., Sathananthan, I., Rujikiatkamjorn C. & Balasubramaniam, A. 2005. Analytical and numerical modelling of soft soil stabilized by PVD incorporating vacuum preloading. Int. J. of Geomechanics, ASCE, 5(2): 114-124.

Seah, T.H. 2006. Design and construction of ground improvement works at Suvarnabhumi Airport. Geot. Eng, J. of Southeast Asian Geot. Society, Vol. 37, 171-188.

Shang, J. Q., Tang, M., and Miao, Z. 1998. Vacuum preloading consolidation of reclaimed land: a case study. Canadian Geotechnical Journal, 35: 740-749.