Modelling of combined vacuum and surcharge preloading with vertical drains

Modélisation combinant la consolidation sous vide et le pré chargement avec drains verticaux

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ABSTRACT

In this study, 2D and 3D numerical analyses were conducted to predict embankment behaviour on soft clay foundations stabilised by Combined Vacuum and Surcharge Preloading. To determine the equivalent plane strain permeability, a revised conversion procedure for plane strain condition considering linear permeability variation in the smear zone is proposed. The equivalent permeability coefficients are then incorporated in finite element codes, employing the modified Cam-clay theory. The advantages and limitations of vacuum application through vertical drains are discussed through two selected case histories from China and Australia and the predictions are compared with the available field data. Apart from realistic 3D numerical modelling, it is demonstrated that the equivalent plane strain analysis can now be used as a predictive tool with acceptable accuracy.

RÉSUMÉ

Des études numériques en 2D et 3D ont été entreprises pour prédire le comportement d'un remblai sur argile molle améliorée par consolidation sous vide combinée avec un préchargement. La perméabilité équivalente en déformation plane est déterminée à l'aide d'une méthode de conversion révisée qui suppose une variation linéaire de la perméabilité dans la zone remaniée. Les coefficients de perméabilité équivalente sont alors introduits dans les codes à éléments finis incorporant le modèle Cam Clay Modifié. L'analyse de deux cas d'études, choisis de la Chine et de l'Australie, a permis de discuter les avantages et limitations de la consolidation sous vide à travers la comparaison entre les prédictions et les mesures en place. A défaut d'une modélisation numérique 3D réaliste, il a été prouvé que l'étude en déformation plane équivalente constitue un moyen de prédiction de précision acceptable.

Keywords : Numerical modelling, Soft soils, Vertical drains, Vacuum Consolidation

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 & Redana 2000). 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 (Indraratna et al. 2004, Kelly et al. 2008).

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

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-2204

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).

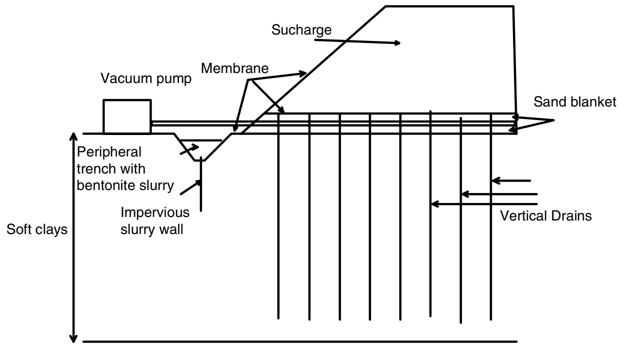

Figure 1. Schematic diagram of PVDs incorporating preloading system for Membrane system

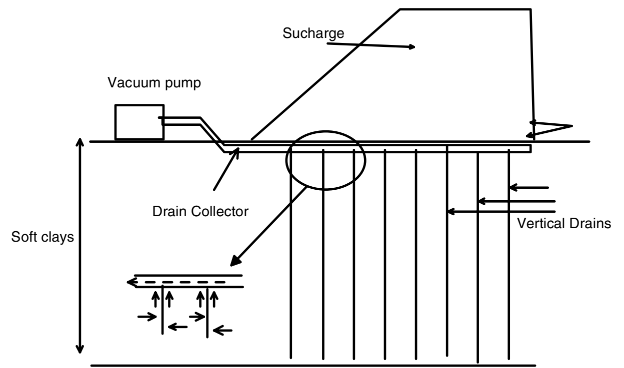

Figure 2. Schematic diagram of PVDs incorporating preloading system for Membraneless system

In this paper, a revised conversion procedure for plane strain condition considering linear permeability variation in the smear zone is proposed. The equivalent permeability coefficients are incorporated in finite element codes, employing the modified Cam-clay theory. Both 2D and 3D numerical analyses are conducted to predict embankment behaviour on soft clay foundations. Two selected case histories from China and Australia are discussed and analysed, and the predictions are compared with the available field data.

2 THEORETICAL APPROACH

2.1 *Axisymmetric condition*

A rigorous radial consolidation theory incorporating both the smear effect and well resistance was proposed by Hansbo (1981). Application of vacuum pressure with only a surcharge load along the surface (i.e. no vertical drains) was modelled by Mohamedelhassan and Shang (2002) based on one-dimensional consolidation. Indraratna et al. (2005) have proposed the radial consolidation theory for vacuum preloading based on constant permeability in the smear zone. However, laboratory testing conducted using large-scale consolidometer by Onoue et al. (1991), Walker and Indraratna (2006) and Sharma and Xiao (2000) suggests that the permeability in the 'smear zone' decreases towards the drain (Fig. 3). To obtain more accurate predictions, the linear permeability in the smear zone will be incorporated in the formulation proposed earlier by Indraratna et al. (2005).

Based on Fig. 3, the variation of permeability can be expressed by (Walker and Indraratna, 2006):

$$k'_h = k_0\left[A\,r/r_w + B\right] \tag{1}$$

where, $\kappa = k_h/k_0$, $A = (\kappa-1)/(s-1)$, $B = (s-\kappa)/(s-1)$, $n = r_e/r_w$ and $s = r_s/r_w$. The diameter of the vertical drain, drain influence zone and smear zone are d_w, d_e and d_s, respectively.

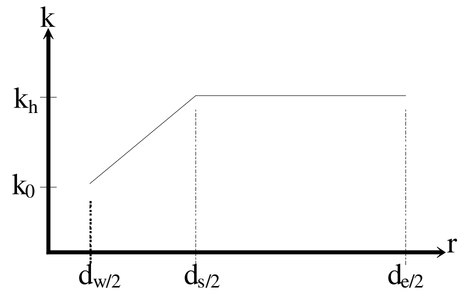

Figure 3. Linear variation of permeability in the smear zone

Using a similar procedure and considering the variation of permeability in the smear zone, the dissipation rate of average excess pore pressure ratio $(R_u = \bar{u}_t/\Delta p)$ at any time factor (T_h) can be expressed as:

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

$$T_h = c_h t / d_e^2$$

$$\mu_L = \ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{\kappa(s-1)}{s-\kappa}\ln\left(\frac{s}{\kappa}\right) \tag{2b}$$

where, p_0 is the applied vacuum pressure, Δp is the surcharge loading, c_h is the horizontal coefficient of consolidation.

2.2 *Conversion Procedure for Equivalent Plane Strain Analysis*

Indraratna et al. (2005) shows that, based on the appropriate conversion procedure by matching the degree of consolidation at a given time step, plane stain multi-drain analysis can be employed to predict soft soil behaviour improved by vertical drain and vacuum preloading.

Using the geometric transformation in Fig. 4, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability is obtained by:

$$\frac{k_{s,ps}}{k_{h,ps}} = \beta \Big/ \left(\frac{k_{h,ps}}{k_h}\ln\left(\frac{n}{s}\right) - \frac{3}{4} + \frac{\kappa(s-1)}{s-\kappa}\ln\left(\frac{s}{\kappa}\right) - \alpha\right) \tag{3a}$$

$$\alpha = 0.67 \times (n-s)^3 / n^2(n-1) \tag{3b}$$

$$\beta = \frac{2(s-1)}{n^2(n-1)}\left[n(n-s-1) + \frac{1}{3}\left(s^2+s+1\right)\right] \tag{3c}$$

The subscription *ps* is for plane strain condition.

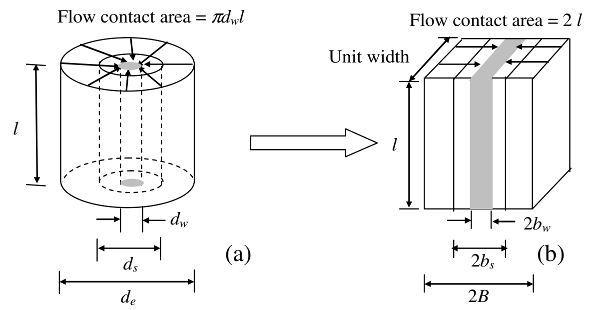

Figure 4. Unit cell analysis: (a) axisymmetric condition, (b) equivalent plane strain condition (after Indraratna et al., 2005)

3 APPLICATION TO A CASE HISTORY

3.1 *Tianjin Port, China*

Tianjin Port is about 100 km from Beijing, China, as reported by Rujikiatkamjorn et al. (2008). Due to the expansion of the port, construction of a new quay on reclamation land was required for a new cargo space facility. The site was reclaimed using clay slurry taken from the seabed formed the first top 3-4m of the soil deposit. The soft muddy clay beneath the reclaimed soil was approximately 5m, followed by the soft muddy clay layer at a depth of 8.5-16m. Stiff silty clay (6m thick) was present beneath the soft muddy clay layer. The soil profile and its related soil properties are shown in Fig. 5, where the groundwater level is at the ground surface.

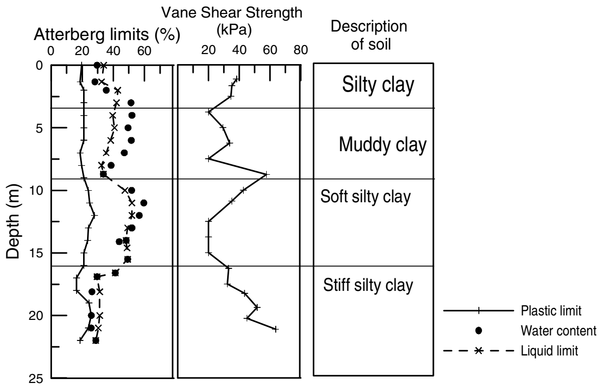

Figure 5. Soil properties and profile at Tianjin port (adopted from Rujikiatkamjorn et al. 2008)

The facility covers an area of 7500 m^2. As the undrained shear strength of the top soft soil is very low, the vacuum preloading method with 80 kPa suction was adopted for improving the soil. Figure 6 presents the vertical cross-section and the location of field instrumentation, which included the settlement gauges, pore water pressure transducers, multi-level gauges, inclinometers and piezometers. PVDs (100 mm × 3 mm) of 20m length were installed at 1m spacing in a square pattern in all three sections. The parameters s and κ were assumed to be 5 and 3, respectively.

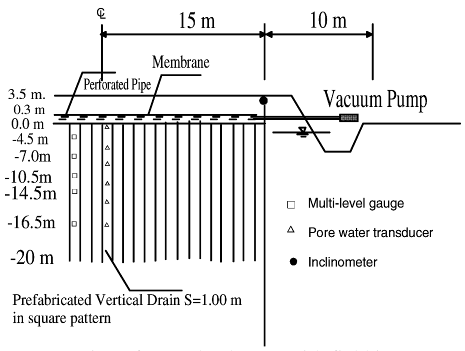

Figure 6. Cross-section of an embankment with field instrumentation

A finite element code (ABAQUS v.6.5.1) coupled with Biot consolidation theory was used to simulate the 3D multi-drain analysis (Hibbitt, Karlsson, and Sorensen, 2005). The 3D finite element mesh representing a quarter of embankment consists of 90000 C3D8RP solid elements (8-node tri-linear displacement and pore pressure) (Fig. 7). In contrast, 2D finite element mesh which is consisted of 14400 C2D8RP solid elements (8-node displacement and pore pressure) were employed for the equivalent plane strain condition via Equation (3) (Fig. 8). Table 1 presents the soil parameters used in the analysis.

Figure 9 shows a comparison between the predicted and measured field settlements at the embankment centreline with the loading history. The settlement predictions from 3D and 2D analyses are almost the same confirming the validity of the conversion procedure. Figure 10 illustrates the comparison between the measured and predicted lateral movements at the toe of the embankment after 6 months. The negative lateral displacement denotes an inward soil movement towards the centreline of the embankment due to the presence of vacuum application. The predictions from the 2D and 3D analyses agree well with the measured data.

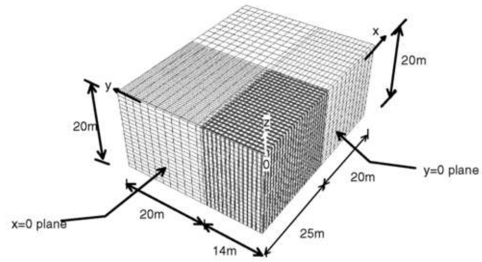

Figure 7. 3D FE mesh (adopted from Rujikiatkamjorn et al. 2008)

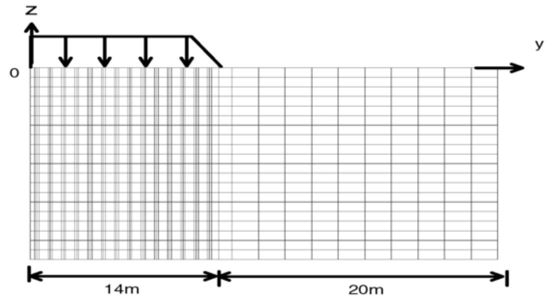

Figure 8. 2D Finite element mesh (from Rujikiatkamjorn et al. 2008)

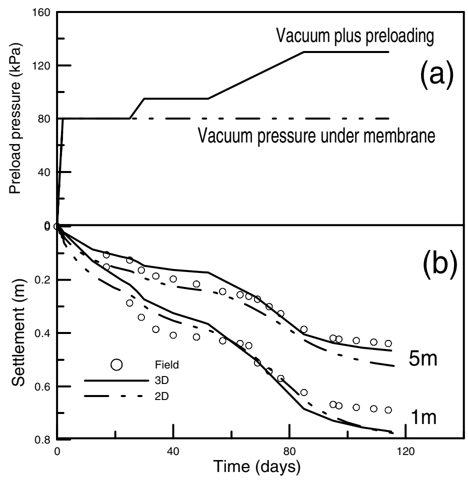

Figure 9. (a) Loading history and (b) Consolidation settlements

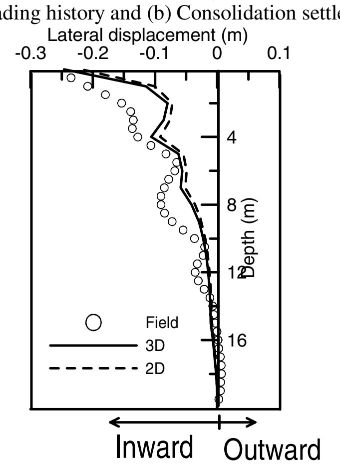

Figure 10. Lateral displacements at embankment toe at 168th day

Table 1 Soil parameters used in the analysis

Depth (m)	λ	κ	γ kN/m^3	e_0	$k_{h,ax}$ 10^{-10} m/s
0.0-3.5	0.12	0.03	18.3	1.1	20
3.5-8.5	0.14	0.03	18.8	1.0	40
8.5-16.0	0.20	0.04	17.5	1.35	20
16.0-20.0	0.10	0.02	18.5	0.9	5

3.2 *Pacific Highway, Ballina, Australia*

The Pacific Highway is being upgraded to support the transportation demand between Sydney and Brisbane, Australia. To avoid the town of Ballina, the bypass route has to traverse a floodplain consisting of very soft, highly compressible, saturated marine clays up to a depth of 40m. Vacuum assisted surcharge load in conjunction with PVDs was selected to shorten the required consolidation time and to stabilize the deeper subsoil layers. To investigate the effectiveness of this approach, a trial embankment at the southern approach to Emigrant Creek, north of Ballina was built with the installation of 30 mm diameter circular drains @1.5m spacing in a triangular pattern. As the soft clay layers fluctuate between 7m to 24m, the design embankment height was varied from 4.3m to 11.2m to limit the post construction settlement. A vacuum pressure of 70 kPa was applied as a negative pressure at the drain interface. The geotechnical parameters of 3 subsoil layers obtained from standard oedometer tests are listed in Table 2. A smear radius to drain radius ratio (s) of 5 and a ratio of smeared to undisturbed soil permeability (κ) of 5 were applied in the analysis (Kelly et al. 2008).

Table 2 Soil parameters used in the analysis

Depth (m)	λ	κ	γ kN/m^3	e_0	$k_{h,ax}$ 10^{-10} m/s	OCR
0.0-0.5	0.57	0.06	14.5	2.9	10	2
0.5-15.0	0.57	0.06	14.5	2.9	10	1.7
15.0-24.0	0.48	0.048	15.0	2.6	3.3	1.1

The construction history and measured settlement at the settlement plate SP-12 are shown in Fig. 11. At this location, the clay thickness was taken to be 24m based on the CPT data. The finite element analysis is similar the Tianjin Port case history described in the previous section. The predictions from 2D and 3D analyses agree well with the measured data. It can be seen that the settlement rate increases significantly after vacuum application. It is estimated that more than 2 years will be satisfied the design criteria (i.e. limit the post construction settlement if the surcharge alone were to apply at this site). In terms of embankment stability, the ratio of lateral to vertical displacement (stability index) was less than 0.13 when the surcharge fill height was 5.3m and it increased to 0.3 once the fill thickness reached 7m. This clearly shows the beneficial effects of vacuum consolidation. Staged construction can be shortened in the vacuum consolidation area compared to the standard surcharge filling methods.

4 CONCLUSIONS

In this paper, a revised conversion procedure for plane strain condition considering linear permeability variation in the smear zone has been proposed. A three-dimensional and two-dimensional multi-drain finite element analyses (ABAQUS) were executed to evaluate the soil consolidation of under combined vacuum and surcharge (fill) loading. The predicted results from equivalent 2D (plane strain) and 3D analyses were similar. From a practical point of view, the height of surcharge fill can be reduced with the application of vacuum preloading to achieve the same desired rate of consolidation thereby reducing the risk of unacceptable lateral yield and associated embankment stability.

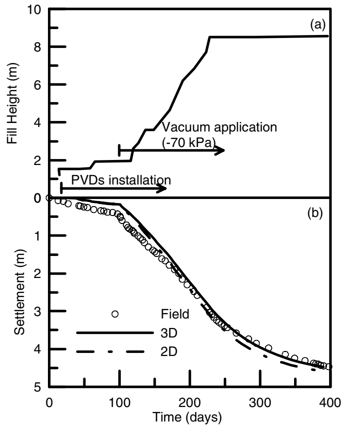

Figure 11. (a) Loading history and (b) Consolidation settlements for Settlement plate SP-12

ACKNOWLEDGEMENTS

The Authors gratefully acknowledge the financial assistance from Australian Research Council. The authors gratefully appreciate the provision of the data from Coffey Geotechnics, Royal Traffic Authority (RTA, NSW) and Austress Menard.

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