The paper was published in the proceedings of the 11th Australia New Zealand Conference on Geomechanics and was edited by Prof. Guillermo Narsilio, Prof. Arul Arulrajah and Prof. Jayantha Kodikara. The conference was held in Melbourne, Australia, 15-18 July 2012.

Numerical Modelling of Soft Soil Improvement using Wick Drains

H. Y. Liu[1] and D. J. Williams[2]

[1]Centre for Geomechanics Research, School of Engineering, The University of Tasmania, TAS 7001, Australia; PH (061) 3-6226 2113; FAX (061) 3-6226 7247; email: Hong.Liu@utas.edu.au
[2]Golder Geomechanics Centre, School of Civil Engineering, The University of Queensland, QLD 4072, Australia; PH (061) 7-3365 3642; FAX (061) 7-3365 4599; email: D.Williams@uq.edu.au

ABSTRACT

Large-scale tank consolidation experiments are conducted in laboratory and simulated using a pore-fluid and stress coupling finite element method. Porous elastic and modified critical state plasticity model are used in the simulation to understand the mechanics of wick drain in improving soil consolidation. The numerical study improves the understanding of the effects of onion ring and smear effects on the soft soil settlement during consolidation using wick drains.

Keywords: soil consolidation, soil improvement, wick drain, numerical modelling, coupling method

1 INTRODUCTION

To successfully construct any structures on soft soil, especially in costal and low lying areas, the soft soil must be improved to increase the bearing capacity and reduce post construction settlement ensuring both the short term and long term stability of the structure. Surcharge is one of the most successful soil improvement techniques. However, for low lying, thick, soft soil deposits with low permeability, it may take months or even years to achieve the desired primary degree of consolidation (>95%) simply by the surcharge method. In this case, wick drain has been used in combination with surcharge worldwide in many soft soil improvement projects (Indraratna, 2010). Wick drains are usually installed by means of a mandrel, which, unfortunately, significantly remoulds the subsoil, especially in the immediate vicinity of the mandrel. Correspondingly, a distributed annulus of soil with reduced lateral permeability and increased compressibility may be developed around the wick drain, which is usually referred to as the smear effect (Chai and Miura, 1999). Moreover, during the consolidation process, a ring of soil with higher degree of consolidation and low permeability is formed around the wick drain in the radial direction, which is referred to as the onion ring, or the so-called moving boundary and the reduction of discharge capacity of vertical drain (Chen et al., 2007). The effectiveness of the wick drain is influenced by the smear effect and the onion ring. To investigate the effectiveness of the wick drain in the soft soil improvement, a series of large-scale laboratory tank consolidation experiments have been conducted at the Golder Geomechanics Centre, University of Queensland. This paper simulates the mechanism of the wick drain in the large-scale laboratory tank consolidation tests using a pore-fluid and stress coupling model.

2 PORE-FLUID AND STRESS COUPLING METHOD

In the pore-fluid and stress coupling model, the soil is considered as a multiphase material and the effective stress principle is adopted to describe its behaviour (Budhu, 2011)

$$\sigma' = \sigma - [\chi u_w + (1-\chi)u_a] \tag{1}$$

where σ' is the effective stress, σ is the total stress, χ is a factor depending on the degree of saturation, u_w is the pore water pressure, and u_a is the pore air pressure. χ is 1 when the soil is fully saturated, 0 when the soil is dry, and between 0 and 1 in unsaturated soils when its value depends on the degree of saturation of the soil. Only the fully saturated case is considered in this paper.

The coupling model attaches the finite element mesh to the solid phase of the soil and ground water can flow through this mesh. A continuity equation is introduced for the ground water equating the rate of increase in mass of ground water stored at a point to the rate of mass of ground water flowing into the point within the time increment. In the analysis, the backward difference operator is used to integrate the continuity equation (ABAQUS, 2011). The accuracy of the time integration is governed by the maximum pore water pressure change allowed in an increment. The flow of ground water is described by Darcy's law. The analysis will terminate when a specified time period is completed, or the steady-state condition is firstly reached.

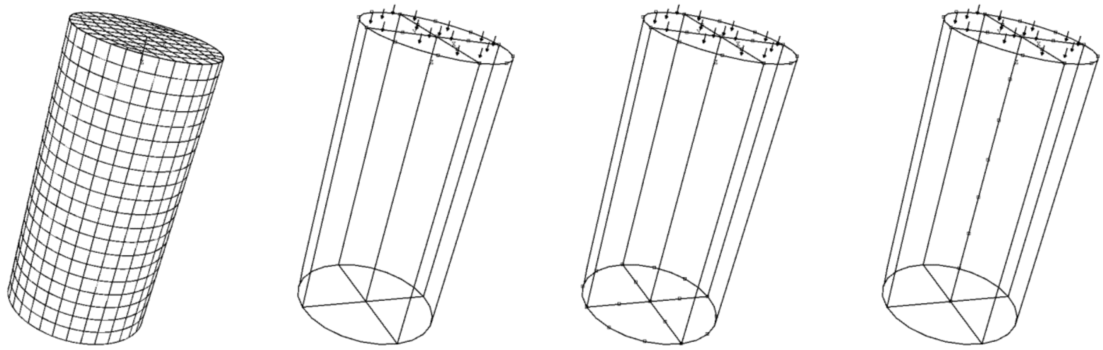
The elastic behaviour of soil is described using a porous elastic model. It is a nonlinear, isotropic elasticity model in which the pressure stress varies as an exponential function of volumetric strain (ABAQUS, 2011).

$$\frac{\kappa}{1+e_0} ln\left(\frac{p+p_t^{el}}{p_0+p_t^{el}}\right) = 1 - J^{el} \quad (2)$$

where κ is the logarithmic bulk modulus, e_0 is the initial void ratio, p is the equivalent pressure stress, p_0 is the initial value of the equivalent pressure stress, J^{el} is the elastic part of the volume ratio between the current and reference configurations, and p_t^{el} is the elastic tensile strength of the material. The plastic behaviour of the soil is described using a modified critical state plasticity model, which is based on the yield surface (ABAQUS, 2011).

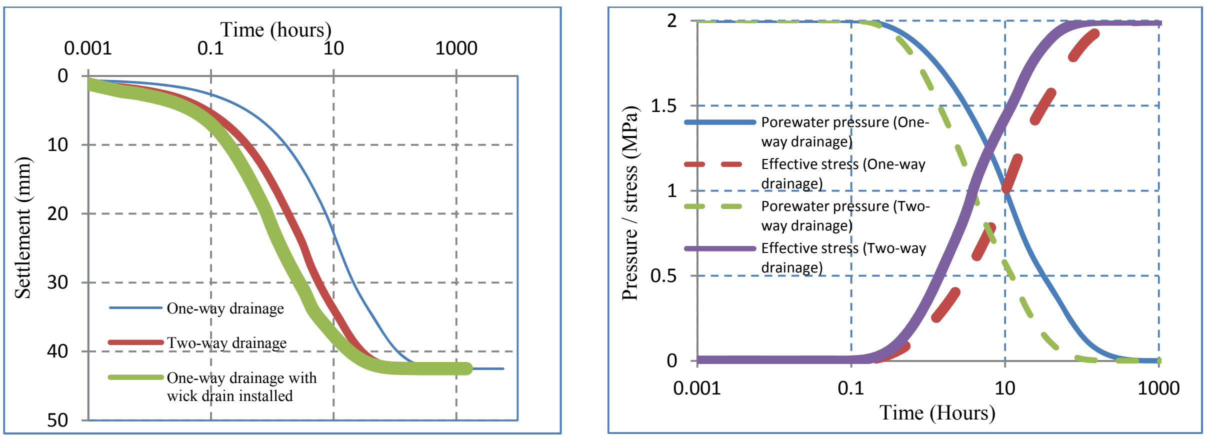
$$\frac{1}{\beta^2}\left(\frac{p}{a} - 1\right)^2 + \left(\frac{t}{Ma}\right)^2 - 1 = 0 \quad (3)$$

where p is the equivalent pressure stress, M is a constant that defines the slope of the critical state line, β is a constant that is equal to 1 on the dry side of the critical state line ($t > Mp$) but may be different from 1 on the wet side of the critical state line, $a = p_c/(1+\beta)$ is the size of the yield surface, and t is the deviatoric stress measure. The size of the yield surface is defined by a. The evolution of this variable, therefore, characterizes the hardening or softening of the material.



a) Finite element mesh   b) One-way drainage   c) Two-way drainage   d) One-way drainage with wick drain

Figure 1. Numerical model showing the finite element mesh, drainage and surcharge

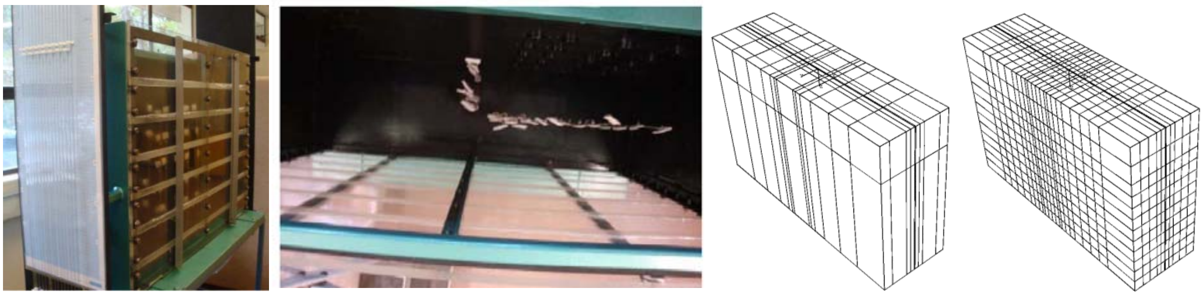


a) Settlement - time curve   b) Pore pressure and effective stress variations

Figure 2. Modelled results of 1D soil consolidation using the pore-fluid and stress coupling method

3 MODELLING OF 1D SOIL CONSOLIDATION FOR CALIBRATION OF THE PORE-FLUID AND STRESS COUPLING METHOD

The 1D soil consolidation is firstly modelled and the results are compared with the famous Terzaghi's 1D soil consolidation theory to verify the pore-fluid and stress coupling method. Fig. 1 depicts the numerical models with various drainage conditions. In order to simulate the perfect drain condition (infinite discharge capacity and no smear), the pore pressures at the drain boundary, i.e. the top surface in one-way drainage, both the top and bottom surfaces in two-way drainage, and both the top surfaces and the centre line in the one-way drainage with wick drain, are set to zero. The smear zone and onion ring are excluded. Fig. 2 depicts the modelled relationship between settlement and time curve and variation of the pore pressure and effective stress at the centre of the models. It can be seen from Fig. 2 a) that the final settlements in the three cases are the same but the rate of consolidation increases in the order of one-way drainage, two-way drainage, and one-way drainage with wick drain. Moreover, as shown in Fig. 2 b), at the beginning of soil consolidation, the applied load is carried by the pore pressure and the effective stress is zero. As the soil consolidates, the excess pore pressure decreases but the effective stress increases. The rate of variation of the pore pressure and effective stress in two-way drainage is faster than that in one-way drainage. Thus, it is obvious the results from the pore-fluid and stress coupling method are consistent with our general understanding of soil consolidation. Moreover, the results from the one-way drainage and the two-way drainage are compared with the Terzaghi's 1D soil consolidation theory and it is found that they agree well with each other. Thus, the pore-fluid and stress coupling method should be able to model soil consolidation either with or without wick drains.



a) Lab tank b) Top view of the lab tank c) Geometry model d) Numerical model

Figure 3. Finite element model for the laboratory tank consolidation experiment

4 MODELLING OF SOIL CONSOLIDATION USING WICK DRAINS IN LABORATORY TANK CONSOLIDATION EXPERIMENTS

4.1 Laboratory tank consolidation experiments

Large-scale laboratory tank consolidation experiments were conducted to investigate the effectiveness of wick drains and the laboratory apparatus is shown in Fig. 3 a) and b). The dimensions of the tank are 1000 mm length, 300 mm width and 750 mm height (Fig. 3 a). There are 8 sets of stand pipes with 5 of them along the length direction and 3 of them along the width direction (Fig. 3b). Each stand pipe set has 5 tubes vertically at 100 mm intervals. Each tube opening has geotextible filters which prevent clogging of the small tubes. Each tube inside the tank connects to a glass tube outside the tank measuring pore water pressure (Fig. 3 a). Twenty plugs are prefabricated in the back wall to collect moisture content samples (Fig. 3 a). The top surface is open enabling settlement to be measured and surcharge to be applied (Fig. 3 b). The details of the laboratory tank consolidation apparatus were explained in the Bachelor of engineering honours students' theses by Liu (2009) and Ramlackhan (2009). Four tests have been conducted till this moment and are listed in Table 1.

4.2 Numerical models

The consolidation of soft clay using wick drain in the large-scale laboratory tank consolidation experiments under a combined self weight and surcharge preloading are modelled using the pore-fluid and stress coupling method mentioned above through the finite element software ABAQUS (2011).

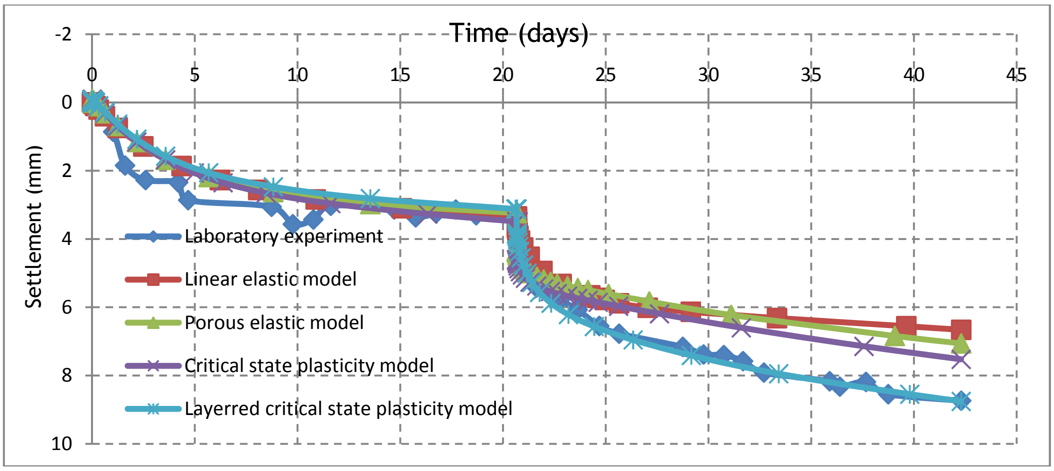
The numerical tests corresponding to each laboratory test are listed in Table 1. The geometry model and finite element model used in the numerical tests are depicted in Fig. 3 c) and d), respectively. The geometry of the numerical model is the same as that of the laboratory consolidation tank except that the height is 650 mm instead of 750 mm since the soil is only filled to 650 mm during the laboratory tank consolidation experiments. The numerical model is partitioned according to the locations of the stand pipe set in the laboratory experiments for the convenience of comparisons between measured and modelled pore water pressures. The 3D twenty-node hexahedral elements with pore pressure in corner nodes are used in the 3D finite element mesh.

Table 1: Summary of laboratory tank consolidation experiments and numerical models

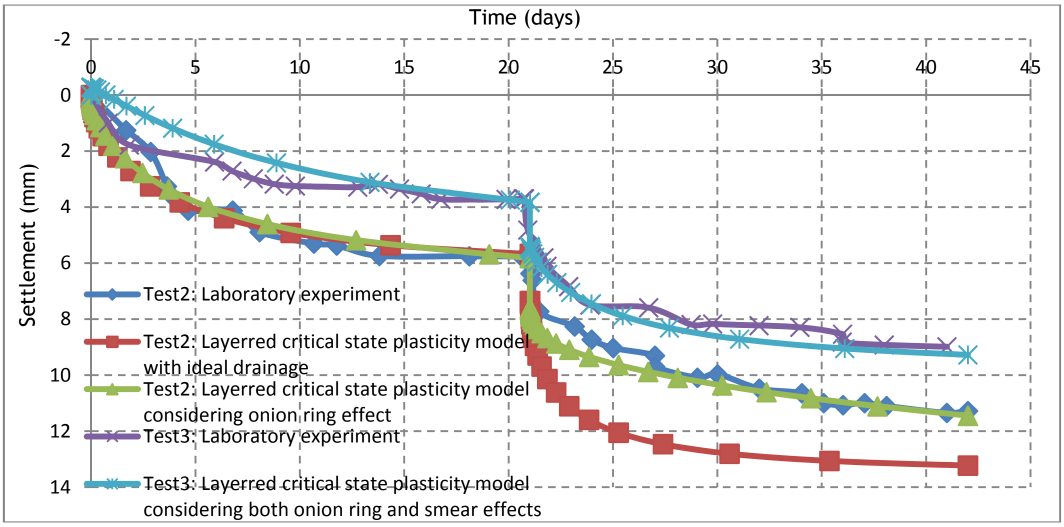
Laboratory experiments	Numerical modelling
Test1: 1D consolidation only	Model1: 1D consolidation only
Test2: consolidation with preinstalled wick (water layer placed)	Model2-1: consolidation with ideal drainage (water layer placed)
	Model2-2: consolidation considering "onion ring" effect
Test3: single wick drain inserted in	Model3: consolidation considering both "smear effect" and "onion ring effect"
Test4: 1D consolidation with water layer	Model4-1: 1D consolidation using linear elastic model
	Model4-2: 1D consolidation using porous elastic model
	Model4-3: 1D consolidation using critical state plasticity model
	Model4-4: 1D consolidation using layerred critical state plasticity model

Test 4 is firstly modelled and used as the benchmark for the modelling of other tests. In Test 4 in the laboratory, the clay is mixed with distilled water to achieve water content $w = 70.4\%$ using a hand held concrete mixer. Correspondingly, the initial void ratio is set as $e = wG_s = 0.704 \times 2.7 = 1.9008$ in the numerical model. In the laboratory, the consolidation tank is filled with 5 consecutive 130 mm layers to reach a total clay depth of 650 mm. During the sample preparation, compaction of clay in the bottom layers is unavoidable by the self weight of clay in upper layers, which affects the void ratio and permeability. Correspondingly, the initial void ratio and permeability are assumed to be linearly distributed, which are 1.9008 and 1.0×10^{-8} m/s, respectively, in the up most layer and 1.8549 and 2.34×10^{-10} m/s, respectively, in the bottom layer in the linear elastic, porous elastic and modified critical state plasticity models to be introduced later. Nonlinear distributions of void ratio and permeability are used in the layerred critical state plasticity model. In the linear elastic model, the elastic modulus is assumed to be constantly 250 kPa and the Poisson's ratio is 0.3. In the porous elastic model, the so-called logarithmic bulk modulus κ, i.e. the slope of the unloading-reloading line in $e - \ln(p')$ plot is assumed to be 0.05. In the modified critical state plasticity model, the so-called logarithmic plastic bulk modulus, i.e. the slope of the normal consolidation line in $e - \ln(p')$ plot is assumed to be 0.23 according to the laboratory test, the so-called stress ratio at critical state M, i.e. critical state line slope in the $q - p'$ plot is assumed to be 0.84.

As shown in Fig. 3 c) and d), the actual rectangular wick drain cross-section is considered in the finite element analysis. According to Chai and Miura (1999), the most wick drain sizes available in market have widths between 90-100 mm and thicknesses of 3-7 mm. Thus, the typical size of wick drain 100 mm x 4 mm is chosen in this study. In the finite element model, the displacement in the Y direction is restrained on the front and back surfaces, that in the X direction is fixed on the left and right surfaces, that in the Z direction is fixed on the bottom surfaces, and the top surface is free to move, where the settlement in the Z direction is measured. In Test 4, the top surface is the drainage surface and all of other surfaces are impermeable. Since 10 mm water layer is placed in the top surface preventing cracking and unsaturation there in the laboratory test, a pore water pressure $u = 10\text{mm} \times 10000\,\text{N/m}^3 = 100\text{Pa}$ is assumed in the top drainage surface. In Test 2 and 3, the pore water pressure is assumed to be zero in the case of ideal drainage. The loading procedure is divided into four steps. The first step checks the geostatic equilibrium under the self weight and the specified boundary conditions. The second step is the soil consolidation step under the self weight. The consolidation time is set as the same as that in the laboratory test. Thus, the soil may not completely consolidate since more time is needed to achieve complete consolidation. The third step is a geostatic step again, which checks the equilibrium under the applied surcharge (4.68 kPa in Test 1 and 3kPa in Tests 2 and 3). The fourth step is used to model the soil consolidation under the applied surcharge. It is found that convergence is easier achieved if the surcharge and consolidation steps are separate.



a) Test 4: consolidation test with 10 mm water layer at top drainage surface



b) Test 2 and 3: consolidation tests with prefabricated and inserted, respectively wick drains

Figure 4. Comparisons between laboratory and numerical results

4.3 Results and discussions

The modelled time-settlement curves and their comparisons with those from the corresponding laboratory test are shown in Fig. 4. It should noted that the results from Test 1 are excluded from this study since the height of the sample and the boundary conditions in Test 1 are different from the other tests. As mentioned above, Test 4, i.e. the ordinary soil consolidation test without wick drain is firstly modelled and used as the benchmarks for modelling of the other tests. Fig. 4 a) depicts the relationship between the time and surface settlement for Test 4 predicted by the linear elastic model, the porous elastic model, the modified critical state plasticity model and the layerred critical state plasticity model. It is obvious the layerred critical state plasticity model predicts the best time-settlement curve compared with that from the laboratory experiment.

The layerred critical state plasticity model is then used to simulate Test 2, i.e. the consolidation test with prefabricated wick drain. The results are shown in Fig. 4 b). It is found that the predicted settlement is much bigger than that from the laboratory experiment, especially during the consolidation stage caused by the applied surcharge. According to previous studies (e.g. Chen et al., 2007), the onion ring may be formed around the wick drain as radial consolidation takes place through the wick drain. With the wick drain installed in the sample, excess pore water must move radial into the wick drain. In the zone close to the wick drain, the excess pore water will move faster than that in the zone relatively far from the wick drain due to longer drainage path. The dissipation of excess pore water in the zone close to the wick drain increases the degree of consolidation there causing the permeability of the zone to decrease. The drop in permeability makes it harder for the excess pore water in the zone relatively far from the wick drain to dissipate, which is referred as moving boundary and the reduction of discharge capacity of wick drain by Chen et al. (2007). However, in the numerical modelling, the onion ring effect is completely ignored since ideal drainage is assumed at the wick drain and the radial permeability is assumed to be constant during the consolidation process. Another numerical model is then built, where the permeability is reduced in the zone (100mm) around the wick drain and the discharge capacity of the wick drain is reduced. The corresponding time-settlement curve is depicted in Fig. 4 b) again. Clearly, the result predicted by the new numerical model considering the onion ring is in acceptable agreement with the laboratory observations.

In Test 3, the wick drain is inserted into the sample, which remoulds the soil around the wick drain and cause the smear zone (Chai and Miura, 1999). There have been various arguments about the size and effect of the smear zone (Hansbo, 1979; Jamiolkowski et al., 1983; Indraratna and Redana, 1998). However, most of them agree that the radius of the smear zone is about 2.5 times the equivalent radius of the mandrel and the lateral permeability within the smear zone is 61%~92% of the outer undisturbed zone. Fig. 4 b) depicts the numerical and experimental time-settlement curve for Test 3. Both the onion ring and smear effects are considered in the layerred critical state plasticity model. With the complex of the onion ring and smear effects in the mind, the modelled curve reasonably agrees well with that from the laboratory experiment. The unit cell theory was proposed by Hansbo (1979) for radial consolidation using wick drain. However, in the theory, the smear zone is represented by a circular shape based on the equal area assumption and a single value of hydraulic conductivity is used in the smear zone. Unfortunately, experimental results indicate that the degree of disturbance, and therefore the coefficient of consolidation in the smear zone, varies with the radial distance from the drain (Chai et al., 2001). Thus more robust radial consolidation theory is needed and it is impossible to compare the results from this study with the theory proposed by Hansbo (1979).

5 CONCLUSION

The large-scale laboratory tank consolidation experiments are simulated using the pore-fluid and stress coupling finite element method with the modified critical state plasticity model. The onion ring and smear effects are studied and it is found that the models with nonlinear variation of void ratio and permeability best fit the experimental results. Throughout this study, the accuracy of settlement prediction in the soft soil improvement using wick drain is significantly improved.

6 ACKNOWLEDGEMENTS

The authors would like to thank Mr Xin Liu and Mr Reshamsing Ramlackhan for conducting the laboratory experiments in this study during the period of completing their bachelor of engineering honours theses at Golder Geomechanics Centre, the University of Queensland.

REFERENCES

ABAQUS. (2011). "ABAQUS 6.10 manuals." Simulia.

Chai, J.C., and Miura, N. (1999). "Investigation on some factors affecting vertical drain behaviour." Journal of Geotechnical and Geoenvironmental Engineering ASCE,125 (3), 216-226

Chai, J.C., Shen, S.L., Miura, N., and Bergado, D.T. (2001). "Simple method of modelling PVD improved subsoil." Journal of Geotechnical and Geoenvironmental Engineering ASCE, 127(11), 965-972

Chen, Y.M., Tang, X.W, and Jia, N. (2007). "Consolidation of sensitive clay with vertical drain." International Journal for Numerical and Analytical Methods in Geomechanics 31, 1695-1713

Hansbo, S. (1979). "Consolidation of clay by band-shaped prefabricated drains." Ground Engineering 12(5), 16-25

Indraratna, B. (2010). "EH Davis Lecture: Recent advances in the application of vertical drains and vacuum preloading in soft soil stabilisation." Australian Geomechanics 45(2), 1-53

Indraratna, B., and Redana, I.W. (1998). "Laboratory determination of smear zone due to vertical drain installation." Journal of Geotechnical and Geoenvironmental Engineering 124(2), 180-184

Jamiolkowski, M., Lancellotta, R., and Wolski, W. (1983). "Pre-compression and speeding up consolidation, general report." in: Proceeding of the 8th European Conference for Soil Mechanics and Foundation Engineering, Balkema, Rotterdam. The Netherlands, 1201-1226

Liu, X. (2009). "Consolidation of saturated fine grained soils using wick drains." Bachelor of engineering honours thesis, the University of Queensland, Brisbane, Australia

Ramlackhan, R. (2009). "Consolidation of saturated fine grained soils using wick drains." Bachelor of engineering honours thesis, the University of Queensland, Brisbane, Australia