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ABSTRACT

In this study, the large-scale apparatus used to examine vacuum preloading in conjunction with conventional surcharge loading. Several tests were performed to examine the effect of a vacuum and determine parameters such as the extent of smear zone and the soil permeability characteristics. The settlement and excess pore pressure associated with a combined vacuum and surcharge load indicates that applying a vacuum has specific benefits. The analytical modeling of one-dimensional consolidation by vertical drains with vacuum preloading considering both variation of soil compressibility and permeability is used to predict the soil consolidation behavior. It is shown that the analytical model can reasonably predict the laboratory behavior.

1. INTRODUCTION

Kjellman (1952) Introduced vacuum preloading method via vertical drains to improve the soft soil properties such as soil compressibility and shear strength. An effective stress in the soil mass is increased by the application of suction pressure (Qian et al. 1992). This method can accelerate the consolidation process without additional surcharge load. A system of vertical drains incorporating surcharge and vacuum preloading has been successfully applied in large highway and port projects (Indraratna et al. 2004, Chu et al. 2000). Mohamedelhassan and Shang (2002) discussed the application of vacuum pressure and its benefits, but without any prefabricated vertical drains (PVDs). In this paper, a series of large-scale testing is presented to study the effect of vacuum and surcharge load. Subsequently, an analytical solution for radial consolidation incorporating vacuum effect introduced by Indraratna et al. (2005) was employed to predict the vacuum consolidation responses. The advantages of vacuum-surcharge were discussed.

2. TEST APPARATUS AND SOIL PROPERTIES

Figure 1 illustrates a schematic diagram of the large-scale radial drainage consolidation cell at the University of Wollongong. The main body of the cell consists of two stainless steel half sections (450 mm inside diameter by 950 mm high) with flanges either side that allows them to be bolted together. The cell stands on a steel base. In order to reduce the friction effect along the boundary of the cell a 1.5 mm thick, ultra smooth Teflon sheet (friction <0.03) was inserted around the internal circumference. The surcharge loading system with a maximum capacity of 1200 kN was applied by an air jack compressor system via a 50mm thick rigid piston, while a vacuum loading system with a maximum capacity of 100 kPa was applied through a hole in the centre of the rigid piston. The instrumentation, including a Linear Variable Differential Transducer (LVDT) and miniature pore pressure transducers, were installed to monitor the consolidation. The porous stone tips were saturated under vacuum and kept there using thin, adjustable, plastic tubes. The cell can also be equipped with a specially designed mandrel which enables a prefabricated vertical drain to be inserted vertically along the central axis of the cell.

The amount of soil required for each sample for the large-scale consolidometer is about 0.14m^3 . Reconstituted commercial clay was used. The selected geotechnical properties of a typical specimen are shown in Table 1. In order to determine the soil compressibility and void ratio-permeability relationship, four different series of oedometer tests were conducted using reconstituted Moruya clay. The $e\text{-log}\sigma'$ and $e\text{-log}k_h$ relationships are illustrated by Figure 2. From these $e\text{-log}\sigma'$ and $e\text{-log}k_h$ plots, the slope of the $e\text{-log}\sigma'$ line (C_c) and the slope of $e\text{-log}k_h$ line (C_k) were found to be 0.29 and 0.45, respectively. Therefore, the corresponding C_c/C_k can be calculated as 0.64. The undrained shear strength was measured by standard consolidated undrained triaxial test.

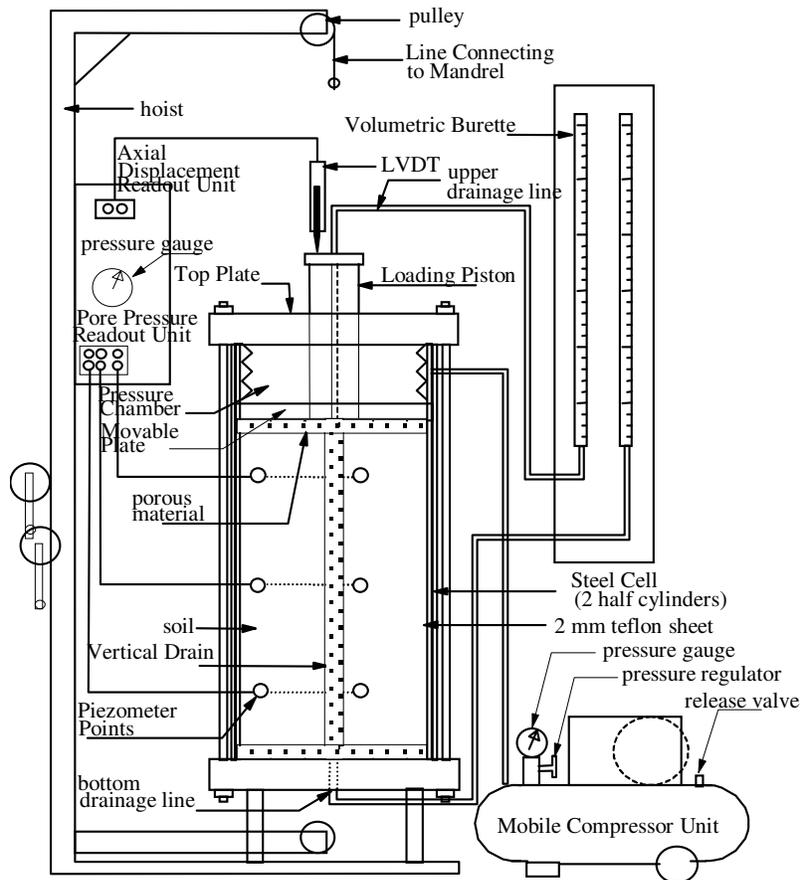


Figure 1. Large - scale consolidation apparatus. (after Indraratna and Redana, 1998).

Table 1. Soil properties of the reconstituted Moruya clay sample.

Clay content (%)	40-50
Silt Content (%)	45-60
Water content, w (%)	38
Liquid limit, w_L (%)	57
Plastic limit, w_p (%)	17
Unit weight (kN/m^3)	18.1
Specific gravity, G_s	2.65
Undrained shear strength (kPa)	7.3

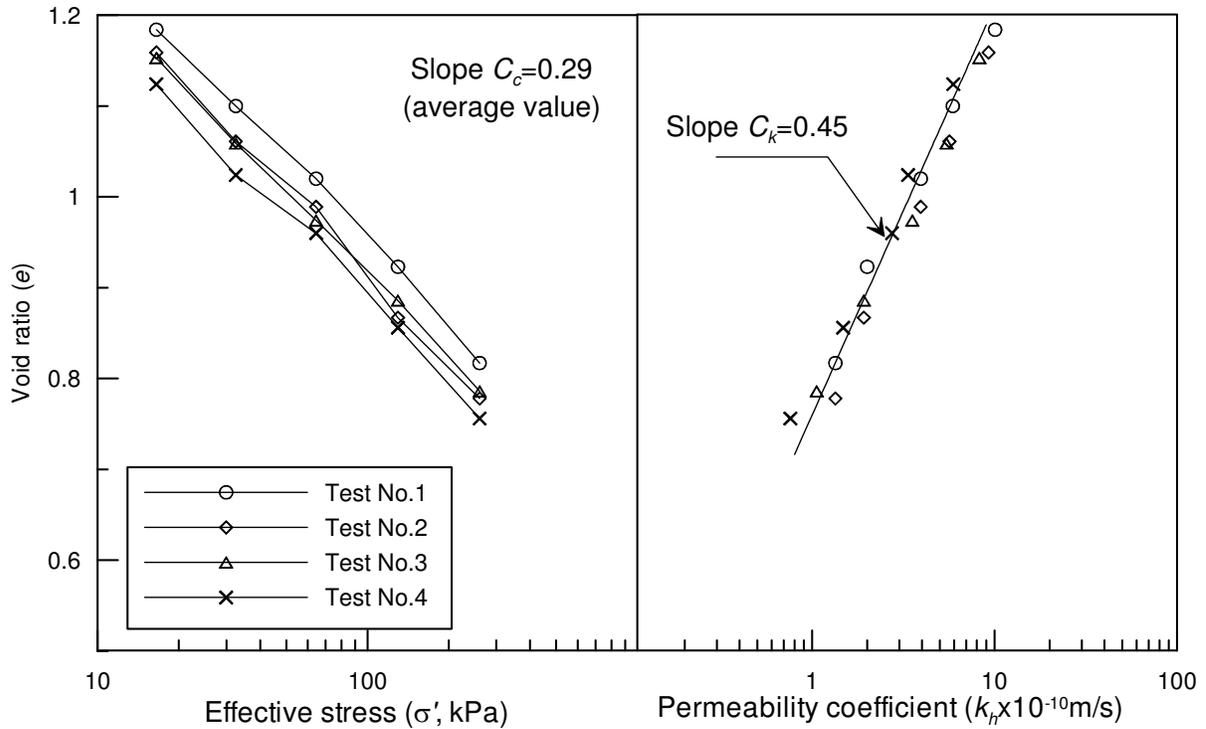


Figure 2. Typical e - $\log \sigma'$ and e - $\log k_h$ plots for Moruya clay (Indraratna et al. 2005).

3. TEST PROCEDURE

In order to investigate the vacuum preloading, a series of three different large-scale tests, including surcharge preloading (SP), vacuum pre-loading (VP), and combined surcharge and vacuum (SV) were conducted. These tests can generally be divided into three stages: preparation of reconstituted clay, drain installation, and collection of oedometer sample. Table 2 summarizes these test series in detail.

Table 2. Summary of the large-scale tests.

Series	Test No.	Applied vacuum pressure (kPa)	Applied surcharge pressure (kPa)	Preconsolidation pressure (kPa)
1	SP	0	30	20
2	VP	20	0	20
3	SV	20	30	20

The procedure for reconstituted clay was similar for all tests. The clay is thoroughly mixed into slurry using distilled water to minimize air entrapment and provide uniform specimens. In this study the water content of the reconstituted clay was approximately 58%. Subsequently the saturated mix (almost a slurry) was placed in 5 layers (about 20cm thick) in the cell. To avoid trapping air each layer was subjected to a vacuum and mild vibration for one hour. After the specimen was prepared, a filter was placed on the surface so that any excess water bleeding out could be removed.

After placing the soft, saturated clay sample, an initial consolidation pressure of 20 kPa was applied in every test to the top of the sample. A vertical band drain was installed using a specially designed rectangular mandrel, a mandrel guide, and a hoist. To ensure that the PVD was saturated it was submerged in water before installation. Six pore pressure transducers were inserted manually at various locations, as shown in Figure 3. Preloading and vacuum pressure was applied according to the appropriate test series, as summarized in Table 2. It is noted that in order to measure the initial distribution of vacuum along the drain, pore pressure transducers T1, T2 and T3 for test series SP and SV were initially installed at the boundary, and then moved 70mm away from it.

Undisturbed vertical and horizontal specimens were collected from different locations at known radii within the cell. A procedure for collecting samples is described by Indraratna and Redana (1998). Oedometer tests were then conducted to establish the extent of smear zone. In each test series the water content was measured at 0.24m, 0.48m and 0.72m from the bottom of the drain to compare with the initial condition.

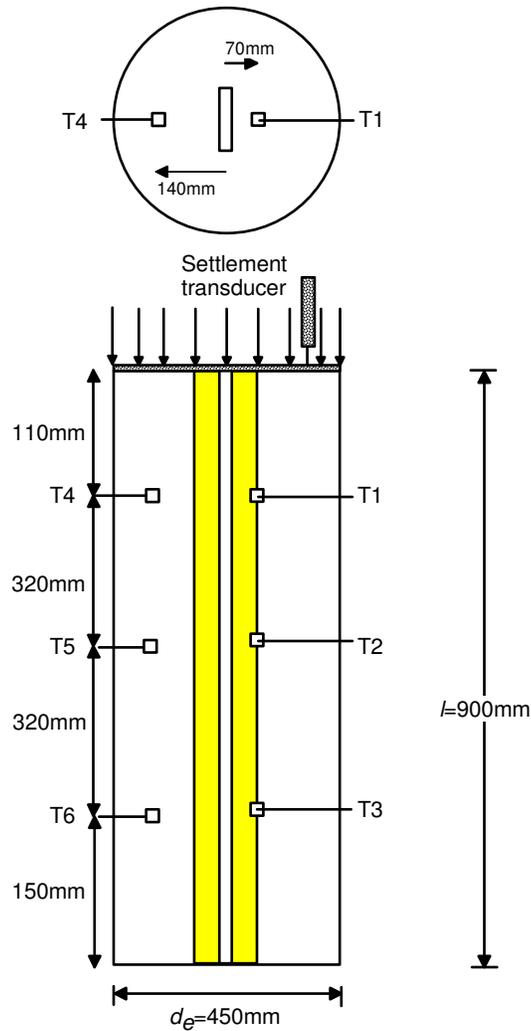


Figure 3. Schematic section of the large-scale, radial drainage consolidometer showing the central drain associated smear zone, and typical locations of pore pressure transducers.

4. SOLUTION FOR RADIAL CONSOLIDATION INCORPORATING SURCHARGE AND VACUUM PRELOADING

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 (Rujikiatkamjorn 2005):

$$R_u = (1 + p_0(1 + k_1) / 2\Delta p) \exp(-8T_h^* / \mu) - p_0(1 + k_1) / 2\Delta p \quad [1]$$

In the above expression,

$$T_h^* = P_{av} T_h \quad [2]$$

$$P_{av} = 0.5 \left[1 + \left(1 + \Delta p / \sigma'_i + p_0 (1 + k_1) / 2 \sigma'_i \right)^{1 - C_c / C_k} \right] \quad [3]$$

$$T_h = c_h t / d_e^2 \quad [4]$$

Where, $n = d_s / d_w$, $s = d_s / d_w$, 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 undisturbed zone (m/s), and 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, and μ = a group of parameters representing the geometry of the vertical drain system and smear effect. Hansbo (1981) assumed the smear zone to have a reduced horizontal permeability that is constant throughout this zone. The μ parameter can be given by:

$$\mu = \ln n / s + k_h / k'_h \ln s - 0.75 \quad [5]$$

The average degree of consolidation based on excess pore pressure can be obtained as follows:

$$U_p = 1 - R_u \quad [6]$$

5. TEST RESULTS AND ANALYSIS

5.1 Vacuum pressure distribution along the drain interface

In order to measure the distribution of vacuum pressure at the boundary of the drain, pore pressure transducers T1, T2 and T3 were placed close to the boundary of the vertical drain during the initial test series SP and SV1. Figure 4 shows the pore pressure measurement along the drain length. It is observed that the vacuum pressure propagates immediately and also decreases down the length of the drain. The rate of development of vacuum pressure within the drain may depend on the length and type of PVD (core and filter properties). The measured pore pressure distribution will be used as a boundary condition in the proposed analytical solutions.

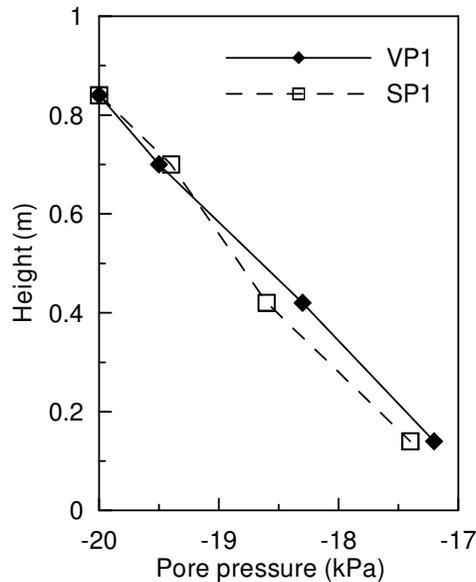


Figure 4. Distributions of measured negative pore water pressure along drain boundary in laboratory testing.

5.2 Evaluation of the extent of the smear zone

Indraratna and Redana (1998) suggested that a dimensionless ratio k_h/k_v is a proper method for determining the smear zone because it minimizes the error occurring when determining the coefficient of permeability in any direction. Figure 5 illustrates the ratio k_h/k_v along the radii of the unit cell and smear zone boundary determined at the preconsolidation pressure of 40 kPa by square root fitting method. The average radius of the smear zone based on an equivalent area was 100 mm or about 3 times the value of r_w , which agrees with the observation of Indraratna and Redana (1998) for the same soil. The ratio of average permeability in undisturbed zone to smear zone (k_r/k_s) of 1.3 can be obtained from both tests.

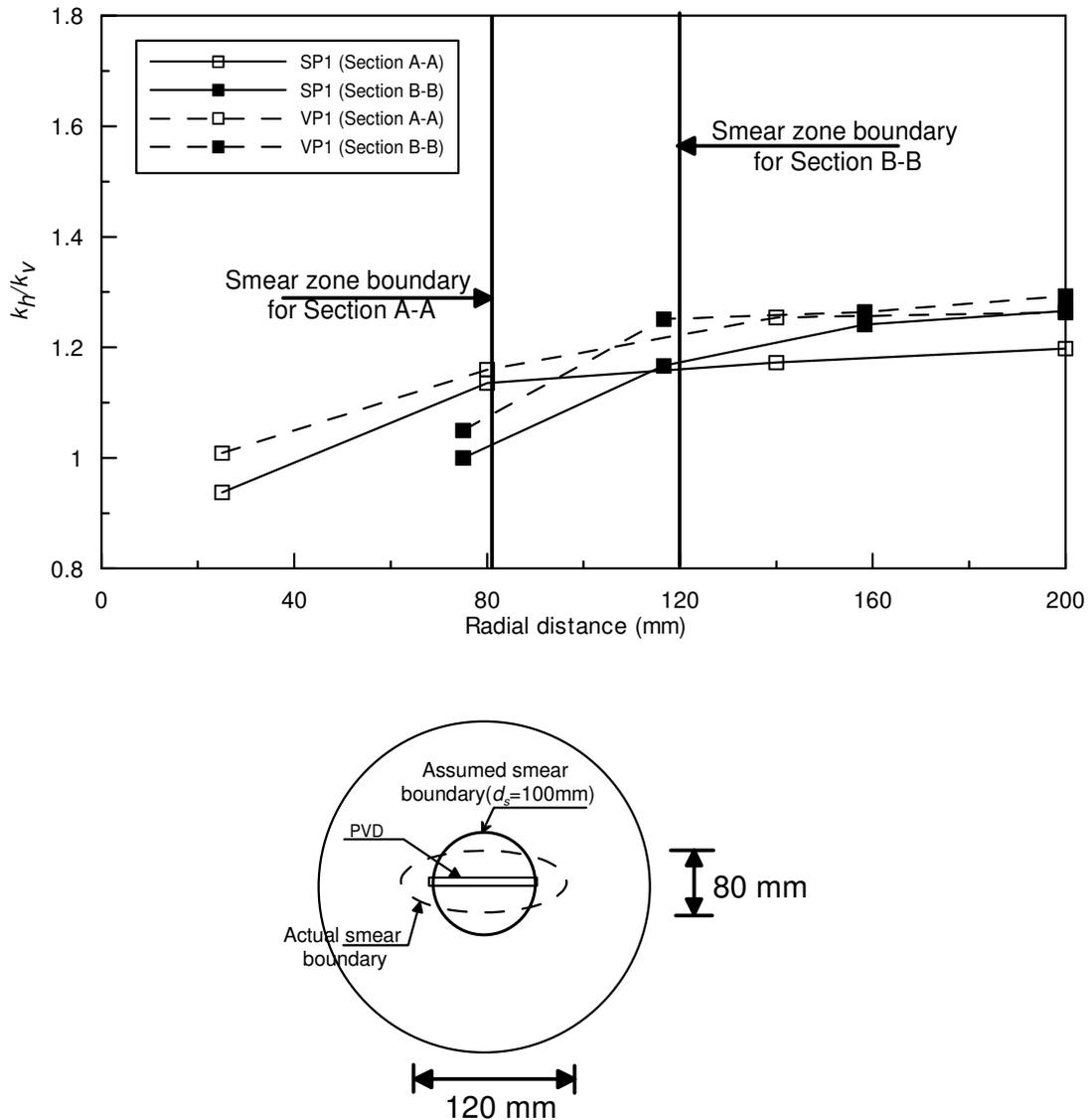


Figure 5. Smear zone extent (a) ratio of k_h/k_v along the radial distance from central drain and (b) smear zone boundary.

5.3 Excess pore pressure dissipations

Figures 6-8 represent a comparison of excess pore pressures between the results calculated and measured. The excess pore pressures were determined by excluding the hydrostatic pore water pressure from the pore pressure at the

measurement locations, whereas the calculated excess pore pressures were modelled by Equation (1). It can be seen that these excess pore pressures agree with the proposed analytical results. The excess pore pressures close to the impermeable boundary (e.g. T1, T2 and T3) were dissipated at a slower rate than those close to the drain (e.g. T4, T5 and T6). At the initial state (time less than 1 day), the excess pore pressures at the vicinity of the drain (T1, T2 and T3) dropped significantly due to the permeable boundary created by PVD. As expected, the tests simulating the surcharge loading (SP and SV) experienced a much higher excess pore pressure compared to those with only a vacuum (VP). In the case of a vacuum surcharge, the final excess pore pressure approached the applied vacuum pressure at the drain boundary. The dissipation rate of excess pore water pressure depends on the magnitude of applied vacuum pressure. It is clear that the PVD allows for negative pore pressure generated along the boundary of the drain. With a vacuum applied the maximum excess pore pressure can be reduced, minimizing the risk of shear failure.

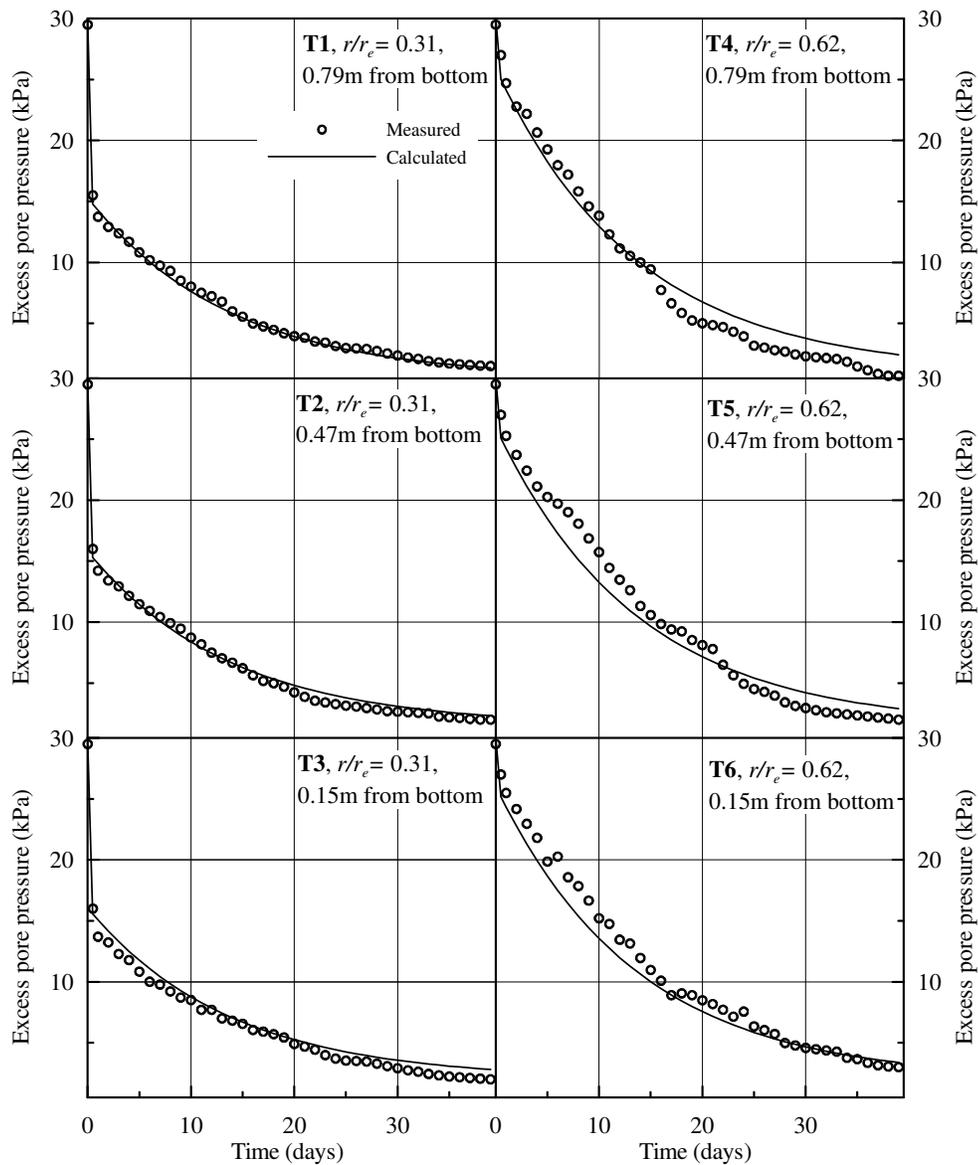


Figure 6. Comparison between the measured and calculated excess pore pressure dissipation for SP1.

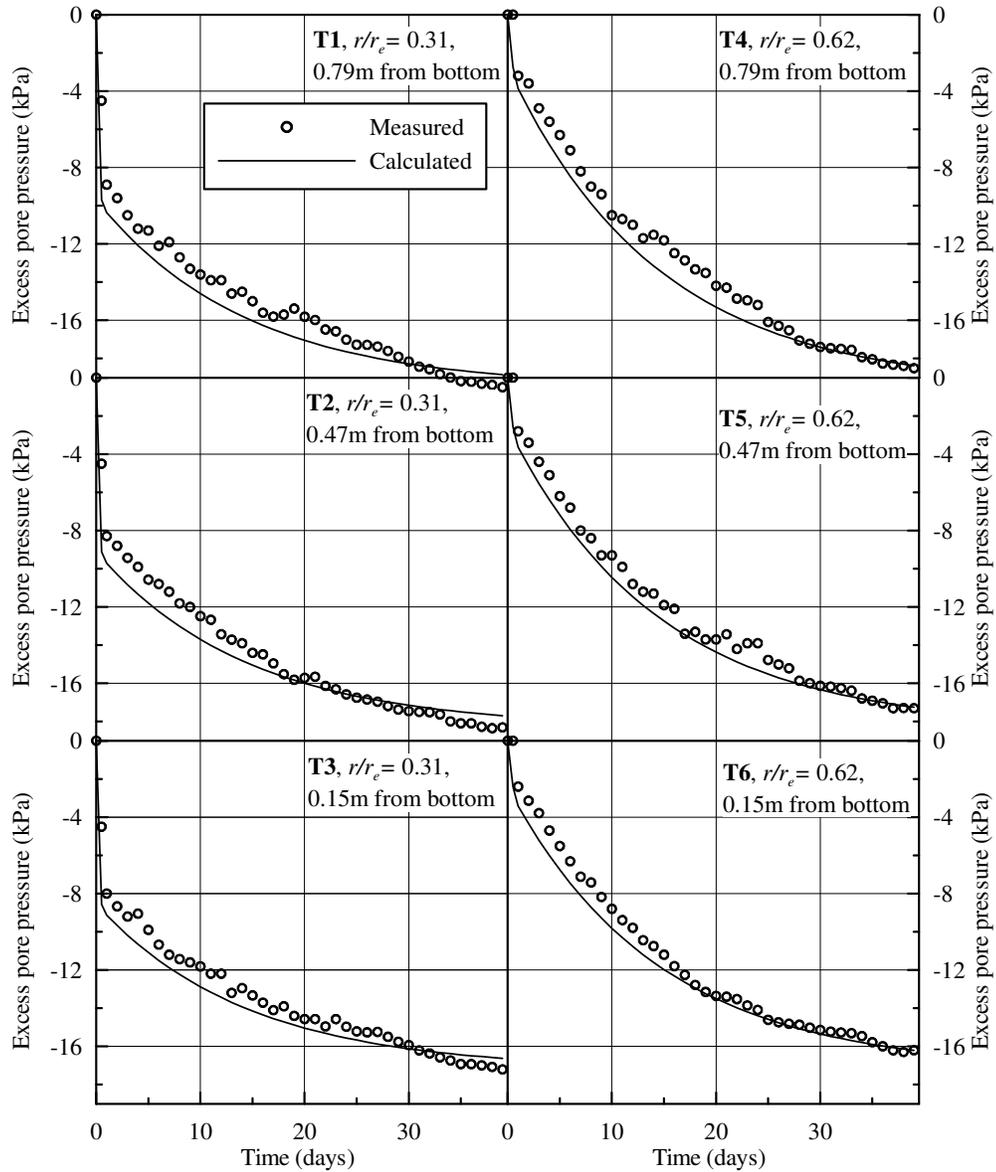


Figure 7. Comparison between the measured and calculated excess pore pressure dissipation for VP.

5.4 Consolidation settlement

The surface settlement-time curves associated with vacuum and surcharge load are shown in Figure 9. The final degree of consolidation for each test is approximately 95%. Clearly, the application of vacuum pressure increases the lateral pore pressure gradient, thus promoting radial flow. This accelerated consolidation increases the rate of settlement as well as the ultimate settlement, which is analogous to increasing the applied surcharge load. However, it is also found that the measured settlement is not directly proportional to the amount of applied load, due to non-linear soil stiffness. The solution incorporates the compression indices and the variation of lateral permeability of the soils, whereas the conventional solution (Hansbo, 1981) considers constant compressibility and constant horizontal permeability. The settlement predictions from the conventional solution underestimate the laboratory results, whereas the predictions using the proposed solutions agree with the laboratory data because the ratio of C_v/C_k is 0.64 (less than 1). This verifies that apart from the permeability and drain configuration, consolidation depends on the variation of soil compressibility and permeability.

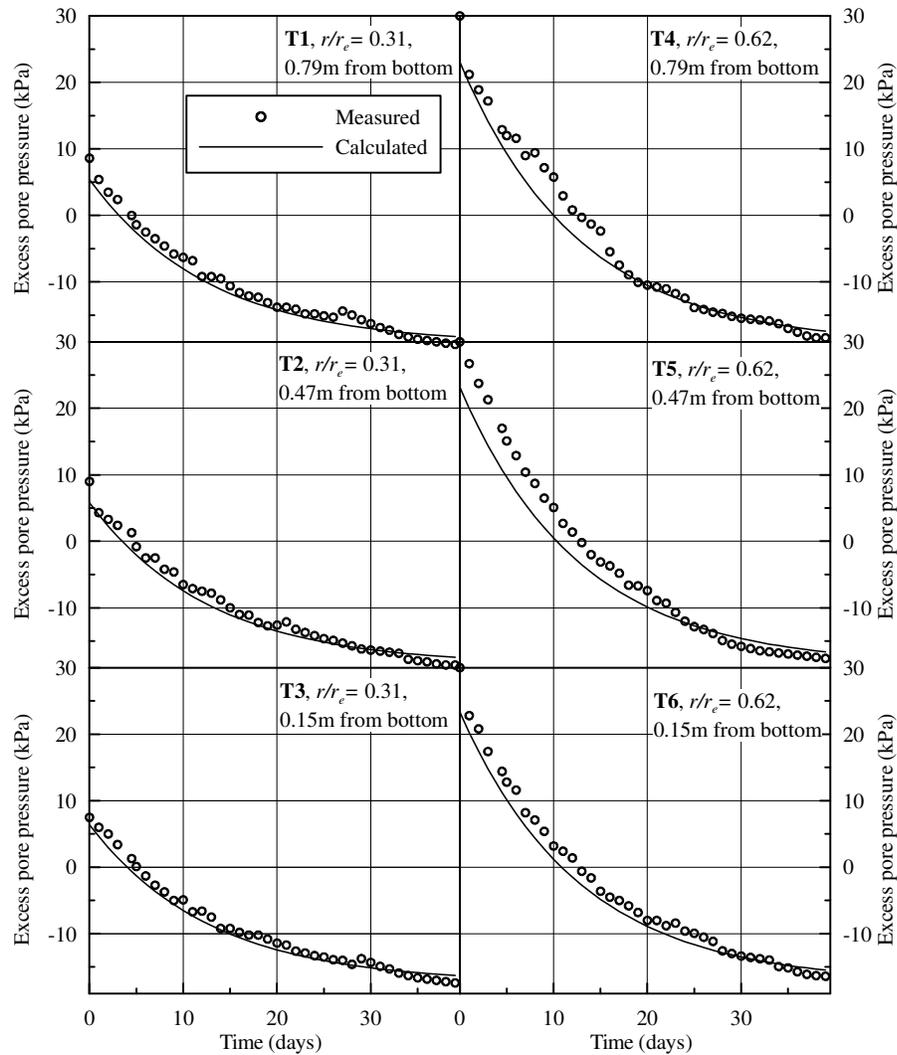


Figure 8. Comparison between the measured and calculated excess pore pressure dissipation for SV1.

6. CONCLUSIONS

The results clearly indicate that a system of prefabricated vertical drains (PVD) combined with vacuum preloading is an effective method for accelerating consolidation. An increase in rate of settlement and pore pressure is expected whenever additional load is applied. The assumption of a linearly varying vacuum pressure along the length of the drain can be observed using the pore pressure transducers at the drain boundary located at different depth. In the field also, the applied vacuum pressure at the top of the drain may not always propagate towards the bottom.

It was found that the average radius of the smear zone was approximately 100 mm, or about 3 times the value of r_w . The average lateral soil permeability in the undisturbed zone is 1.3 times that in the smear zone. The excess pore pressure close to the drain decreased significantly when a vacuum pressure was applied. The excess pore pressures at the end of primary consolidation approach the value of applied vacuum pressure at the boundary.

The amount of settlement observed in the tests depends on the magnitude of applied surcharge and vacuum. However, it is not linearly proportional to the magnitude of applied surcharge and vacuum due to the non-linear soil stiffness. The settlements and excess pore water pressures were analysed using the proposed analytical solution and compared with the laboratory data. The proposed solution incorporating the compressibility indices (C_c and C_r), and the variation of horizontal permeability coefficient (k_h) gives a greater accuracy of the predictions than the previous solutions proposed by Hansbo (1981).

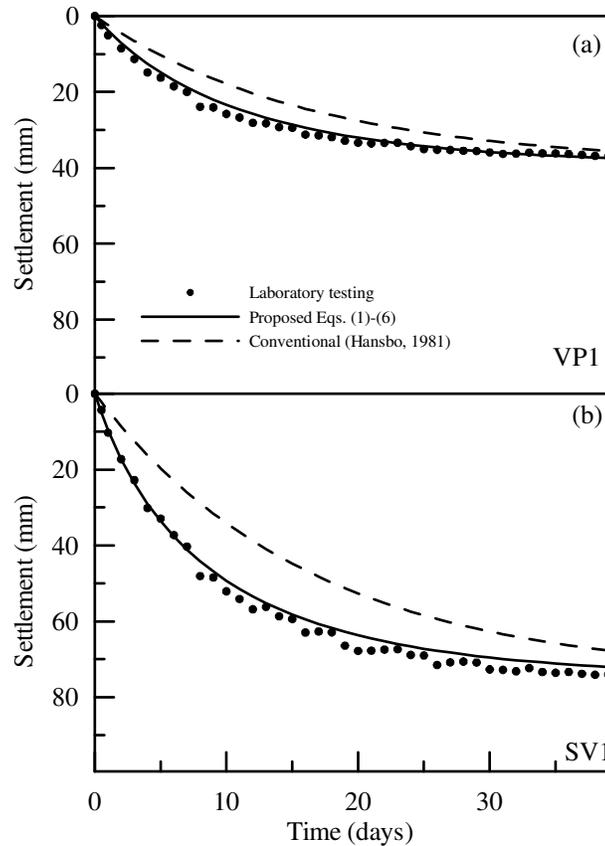


Figure 9. Settlement-time curves.

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