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Experimental Study on the Effectiveness of Prefabricated Vertical Drains under Cyclic Loading

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Synopsis: This study is an attempt to investigate the effectiveness of a prefabricated vertical drain (PVD) installed in remoulded soft clay, where a large-scale cyclic triaxial process simulation rig was employed to simulate cyclic loading representing a track environment. The experimental data show that PVD successfully prevents excessive accumulation of pore water pressure during cyclic loading, and their dissipation continues to occur after the load is removed. The findings of this study have direct significance to rail track environments, e.g. in coastal Australia, where PVD installation beneath rail tracks constructed on low-lying estuarine soils has been recently adopted.

Keywords: ballast, deformations, prefabricated vertical drains, rail track.

1. Introduction

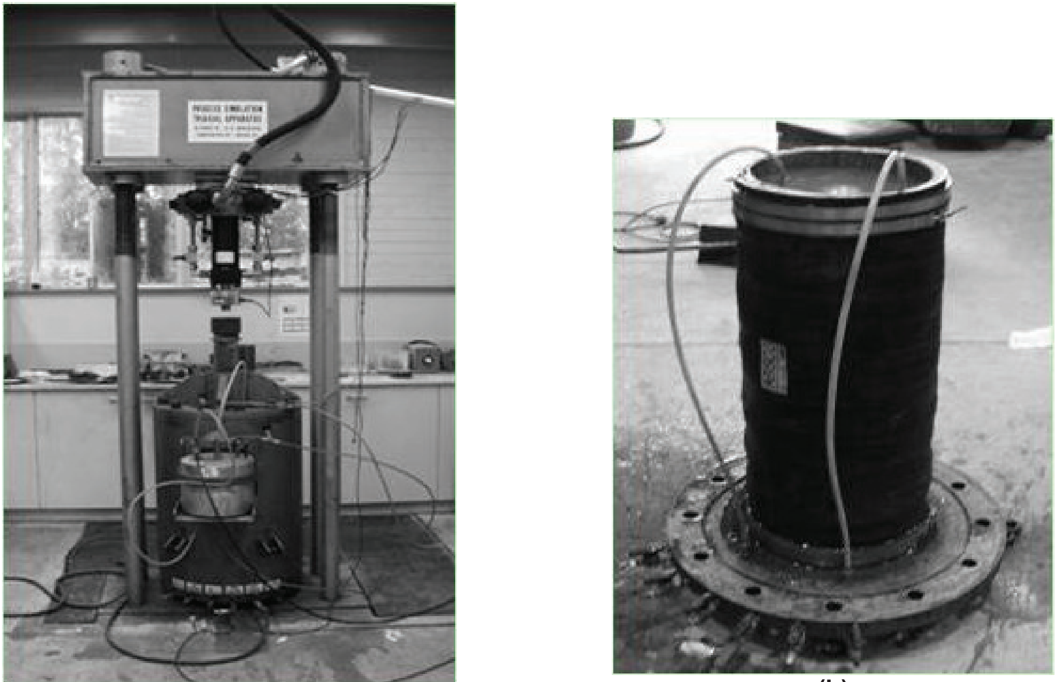
Rapid urbanisation world-wide and associated infrastructure developments have compelled geotechnical engineers to use the poorest of soft soils, as most of the rail and road networks and ports have to be constructed on coastal and marshy areas of very soft clays. In many coastal areas of Australia, soft soils and estuarine deposits can often exist to depths exceeding 30m and are characterised by low permeability, high compressibility, high lateral yield upon loading and low shear strength and bearing capacity. In the absence of appropriate ground improvement, the soft deposits with high volumes of plastic clays can sustain high excess pore water pressures during both static and cyclic (repeated) loading [4, 6]. In poorly drained situations, the increase in excess pore pressures generated by cyclic loads (e.g. high speed rail and aircrafts) will decrease the effective load bearing capacity of the soil foundation. Under such circumstances, slurrying or pumping of clay beneath rail tracks may initiate catastrophic undrained failure [3, 5], causing unacceptable settlements and damage to overlying infrastructure.

Installation of prefabricated vertical drains (PVDs) is a common and effective technique to facilitate the ground improvement process by shortens the drainage paths. In poorly drained situations, the PVDs are also effective in preventing the build-up of cyclic-induced pore water pressure to critical level. These PVDs continue to dissipate excess pore water pressure even after the cyclic loading stops.

Several previous studies have been devoted to radial drainage under seismic and/or dynamic loads in stabilising liquefiable sand deposits. Nevertheless, the performance of PVDs in low permeable soft soils under cyclic loads has not been studied comprehensively. In this study, the effectiveness of PVDs under cyclic loading (e.g. high speed train and aircrafts) was investigated by large scale cyclic triaxial consolidation tests. A preliminary finite element analysis was employed to examine the performance of track stabilised by vertical drains in Sandgate (NSW). The monitored settlement and lateral displacement results were also presented and discussed.

2. Sample Preparation and Methodology

In order to investigate the effectiveness of PVDs under cyclic loading (e.g. high speed train and aircrafts), large scale consolidation tests were conducted on kaolin sample with or without PVDs. The testing system is presented in Figure 1(a). The equipment is capable of accommodating specimens of 300-mm in diameter and 600-mm in height as shown in Figure 1(b). Different frequencies and cyclic stress ratios (CSR) were applied to simulate the variation in train speed and train loading.



(a) (b)

Figure 1. (a) Large-scale triaxial apparatus, (b) soil specimen

Due to the difficulty of obtaining large undisturbed soil samples (at 300-mm diameter), the reconstituted Sandgate clay was used in this study. The Sandgate clay has a liquid limit, w_L of 66% and a plastic limit, w_P of 28%. Standard oedometer test was conducted on undisturbed specimen to determine the consolidation parameters of the soil. The results are presented in Table 1.

Table 1. Consolidation parameters of undisturbed Sandgate clay

Consolidation Parameters	value
Compression Index (C_c)	0.84
Swelling Index (C_r)	0.14
Permeability Change Index (C_k)	0.90

The reconstituted Sandgate clay was prepared as follow:

a. Sandgate clay was wet-screened through a # 40 sieve (0.425 mm opening size) to remove larger particles including shells and gravels.
b. Clay was then remixed with water (using rotary mechanical mixer) until the water content was consistent throughout the clay sample.
c. The rubber membrane was clamped to the base of the triaxial equipment and a filter was placed at the base to prevent obstruction of the drainage line with pore pressure transducers.
d. Subsequently, the remoulded saturated slurry was placed and lightly compacted in four layers (15 cm each) inside the membrane (unit weight @ 15.45 kN/m^3, representing field density) using a mild vibration.
e. During the placement of slurry, six saturated pore pressure transducers were positioned at the locations illustrated in Figure 2.
f. A vertical band drain was inserted vertically using a steel mandrel at the centre of the clay sample
g. A geotextile filter was placed at the top of the sample after the vertical drain insertion to protect the drainage holes of the top cap from clogging.

After specimen setup, a cell pressure of 120 kPa and a back pressure of 99 kPa were applied. To ensure that the clay was fully saturated, the Skempton *B* parameter was calculated by back pressure technique (>0.99). The sample was then consolidated under K_0 condition (K_0=0.6) determined from past laboratory tests, made available by the track owner to the Authors. The sample was subjected to a vertical overburden pressure of 35 kPa and a horizontal pressure of 21 kPa to simulate the stress condition at the mid depth of PVDs (4m depth). The consolidation process was carried out in three incremental stages to avoid any risk of shear failure.

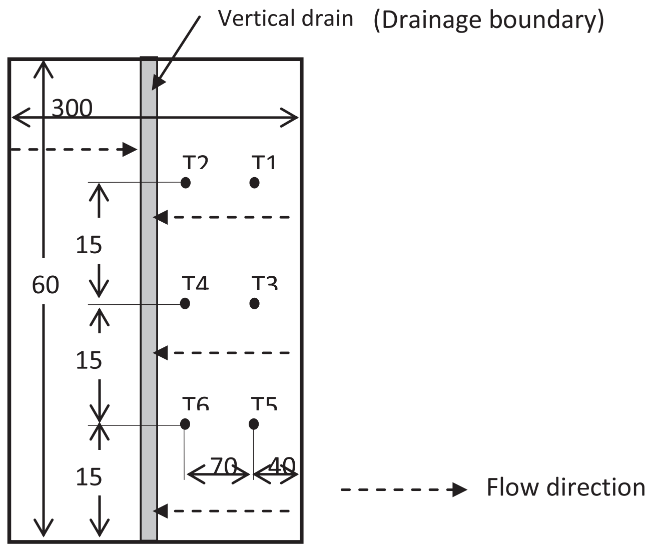


Figure 2. Location of pore pressure transducers inside the soil specimen

After K_o consolidation, the test was conducted at a frequency of 5 Hz, typically simulating the load distribution to the subgrade at a train speed of less than 40 km/hr. The cyclic load was applied with a maximum load intensity of 56 kPa to produce the same average stress at the 3m depth for a typical 25 tones/axle train load. The total number of load cycles applied in each test was 1100 cycles to simulate the passage of the train (i.e. 5Hz at 3.7 mins). The cyclic loading was then terminated. The built up excess pore water pressures due to cyclic load and the dissipation of excess pore pressures after removal of the cyclic load were recorded during the test.

3. Test Results and Discussion

Three series of tests were carried out: (a) cyclic with PVD, (b) cyclic consolidated undrained (cyclic CK_oU) without PVD and (c) cyclic unconsolidated undrained (cyclic UU) without PVD. A series of conventional monotonic triaxial tests was conducted to obtain the maximum deviator stress at failure (q_f) during static loading. Then a cyclic stress ratio (CSR) of 0.65 was chosen, where CSR is s the ratio of the cyclic deviator stress q_{cyc} to the static deviator stress at failure q_f [2, 9]. Larew and Leonards [7] discussed the critical cyclic stress ratio as the level of cyclic deviator stress above which a sample would fail after a certain number of loading cycles. Failure signifies a condition of rapidly accumulating permanent deformations, and this can be represented in a semi-logarithmic plot at the point where the deformation curve slope starts to change rapidly. Various researchers show that this critical cyclic stress ratio varies between 0.6 and 0.7 [1, 8, 9].

3.1 Excess Pore Pressure

Excess pore ware pressure ratio (u^*) is denoted as the excess pore water pressure normalized to the initial effective mean pressure [8, 9]. Figures 3 presents the excess pore pressures ratio (u^*) versus the number of loading cycles (N) under the condition with PVD. The response of the six pore pressure transducers presents the effect of the drainage path length on the accumulation of the excess water pore pressures. Measured excess pore pressures and the corresponding excess pore water pressure ratio (u^*) versus the number of loading cycles (N) under the three separate series of tests are illustrated in 3(a). Without PVD, the excess pore pressure accumulated rapidly ($u^* \approx 0.9$), and undrained failure can be observed. On the other hand, with PVD, the excess pore pressure raised to a constant value ($u^* < 0.4$) after 500 loading cycles. The excess pore pressures measured at T3 and T4 were the minimum (i.e. shortest drainage path length), while the maximum values were observed at T1 and T6 (Figure 4). The data verify the performance of PVD in reducing the rapidly induced excess pore water pressures under cyclic loads, thereby mitigating potential undrained failure. Figure 4(b) presents the development of volumetric strains (compression) with the number of cycles associated with the dissipation of the excess pore pressures with PVD. The volumetric strains approach a constant level at 1.7% in compliance with the relatively constant u^*. For the tests without PVD, the measured volumetric strains were almost zero (Figure 4b) as the cyclic load applications were carried out under totally undrained conditions.

3.2 Settlement

The measured axial strains (ε_a) are shown in Figure 5(a). Without PVD, large strains developed, and the failure occurred after about 200 cycles, and after about 100 cycles for the cyclic CK_oU test and cyclic UU test, respectively. Failure is detected when ε_a-log*N* curves start to concave downwards rapidly. With PVD, axial strains increased gradually to a constant level and no failure was evident even after 3000 loading cycles as illustrated in Figures 4(a) and (b).

Figure 5 shows the relationship between the development of pore pressure ratio (u^*) and the calculated values of the accumulated shear strains (ε_s) for tests with and without PVD. The equations used for the shear strain calculations are shown below.

$$\varepsilon_s = \frac{2}{3}(\varepsilon_a - \varepsilon_r) \quad (1)$$

$$\varepsilon_w = \varepsilon_a + 2\varepsilon_p \quad (2)$$

As the shear strain exceeds a critical level of 1.5-2%, the rate of increase in the excess pore pressure ratio becomes substantial. As expected, at the same shear strain, the cyclic excess pore water pressures without PVD are always greater than those with PVD. It can be seen from Figure 5 that there is an exponential relationship between the developed pore pressures and the accumulated permanent shear strains.

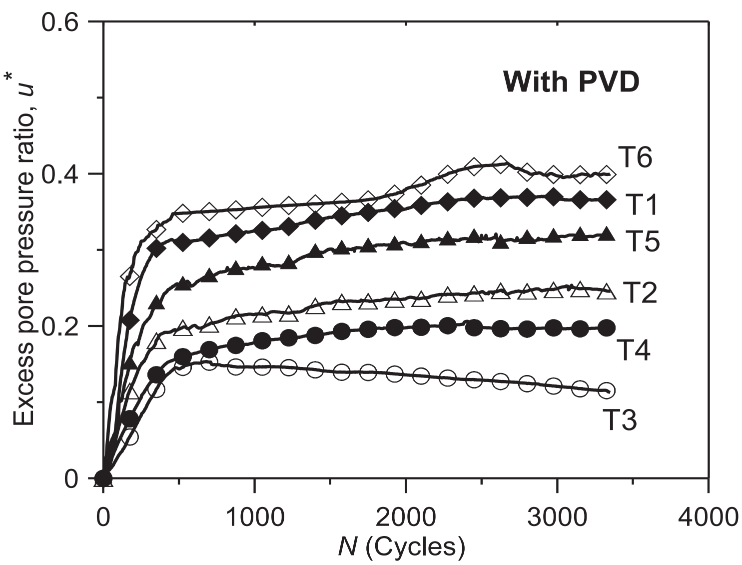


Figure 3. Excess pore pressures generated inside the soil sample at different locations from the PVD with the application of cyclic loads [4]

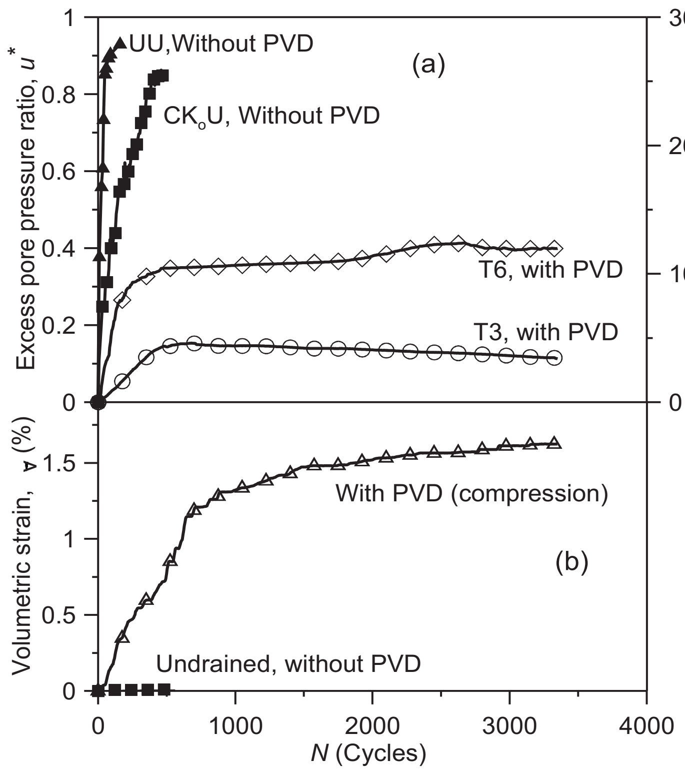


Figure 4. (a) Excess pore pressures generated with and without PVD under cyclic loads, (b) Volumetric compressive strains developed under cyclic loads with PVD [4]

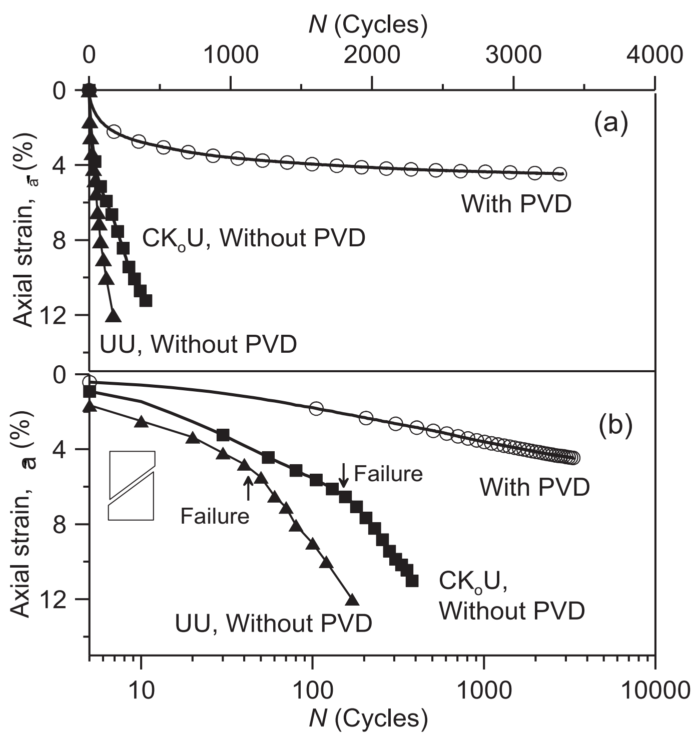


Figure 5. Axial strains during cyclic loading with and without PVD versus number of loading cycles (*N*): (a) Arithmetic scale, (b) Semi-logarithmic scale [4]

4. Conclusions

A series of large-scale cyclic triaxial tests was carried out on soft kaolinite clay with and without PVD. The effect of radial drainage under the repeated loads was investigated. The findings of the study suggest:

a) During the application of cyclic loads, the PVD decreases the buildup of excess pore water pressure, and also accelerates its dissipation during the rest period.
b) The cyclic-induced excess pore pressures tend to increase substantially as the shear strain exceeds 1.5-2%.
c) Soft clays provided with radial drainage (PVD) can be subjected to cyclic stress levels higher than the critical cyclic stress ratio without causing undrained failure.

5. Acknowledgements

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