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PAPER TITLE: **Laboratory investigation of pore pressure dissipation in clay around permeable piles**

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Abstract

Excess pore water pressures induced by pile driving could have detrimental effect in the project, especially on the construction sequence and ground settlement. Measures that can effectively accelerate dissipation of pore pressures are therefore valuable. One way to reduce the pore pressure is the use of permeable piles. This paper presents a series of model-scale laboratory tests conducted on piles with drainage holes around the pile circumference. It is found that permeable piles could accelerate soil consolidation significantly, and the beneficial effect are more apparent for group pile tests. An approximate influencing zone of permeable piles was derived in this study based on the experimental observations.

Keywords: consolidation, model tests, pore water pressure, permeable piles, clays

Introduction

It is a common practice these days to use driven piles especially when an incompetent layer of soft clayey soil is encountered on site. However, the pile driving process induces displacement (including radial deformation, ground heave and settlement) and generates pore water pressure around the pile that can be detrimental to the project. For example, construction sequence needs to be controlled carefully to minimize the impact of pile installation on existing piles; otherwise, undesired deflection, bending, fracturing and floating may occur. Ground deformation can cause adverse influence on adjacent infrastructure, such as crack initiation, excessive subsidence and differential settlement of buildings, and failure of roads and underground facilities (e.g., pipelines and tunnels). The slow dissipation of pore water pressure and long consolidation time after pile installation introduces difficulty in estimating the correct axial capacity of pile due to the set-up effects.

A full dissipation of pore water pressure in sensitive clay could be reached after several months of pile installation (Bozozuk et al. 1978; Roy et al. 1981). As the consolidation proceeds, driven piles can gain capacity with time, which is called the set-up effects. The increase of pile resistance with time does not only occur in clays, but in sands too (Chow et al. 1997; Jardine et al. 2006). They reported a series of field tests at Dunkirk, northern France, where the strength of pipe piles embedded in dense sand increased by 85% after five years. The enhanced radial stress was believed to be induced by creep and aging in sand, which restricted the arching effect and resulted in stronger dilation (Chow et al. 1997). The time effects have also been observed for axial capacity of displacement piles, where a database of 80 pile load tests indicated that both sands and clays experienced the set-up effects (Long et al. 1999). The recent work of Lim and Lehane (2014) illustrated a new theory that the set-up effects could be considered as a strength recovery from soil disturbance induced by pile installation, rather than a capacity gain with time. Although the mechanism of improved pile performance with time is still controversial, it is a common perspective that reducing the time for consolidation in the surrounding soil is beneficial to improve pile's resistance at early stages of construction.

Measures that can provide drainage paths are therefore sought to accelerate pore pressure dissipation. Hird and Moseley (2000) conducted a set of experimental work on vertical drains in compressible soils. The influence of smeared soil disturbance was evaluated, which was equivalent to an established vertical drainage path within the soil. Analytical solutions have also been derived for vertical drains (Tang and Onitsuka 2000; Zhu and Yin 2004; Basu et al. 2006; Walker and Indraratna 2006). Chu et al. (2004) summarized some design and practical considerations for vertical drains from field monitoring data for a land reclamation project, and the impact of surcharge on shortening consolidation was especially elaborated. The benefits of vertical drains have been identified for embankment on soft clay deposit both experimentally (Pothiraksanon et al. 2010) and numerically (Shen et al. 2005). However, Hird and Srisakthivel (2005) pointed out that hydraulic fracturing could occur around vertical cavities at relative small pressure heads, where the drains were degraded by partially softened fill material and the permeability was lowered. Similarly, Weber et al. (2010) reported the degradation of consolidation for stone columns.

Alternatively, researchers have been attempted to address the problems with driven piles by making the piles permeable, so that vertical drainage paths could be created with minimal disturbance to the surrounding soil. For example, pervious concrete has been used to cast the piles (Suleiman et al. 2014; Ni et al. 2016). However, the strength of pervious concrete could limit the application of this new type of piles (Obla 2010). Ni et al. (2017b) have suggested the solution of permeable pile as given in Fig. 1(a) to increase the rate of consolidation, reducing the pore water pressure and settlement. The authors demonstrated the efficacy of permeable pile using numerical techniques, and concluded that drilling drainage holes around the pile circumference within the bottom 50% of the pile length could improve the degree of consolidation by over 40%. It should be emphasized that plugs must be employed to block drainage holes during pile driving to minimum the potential of clogging. A mechanical pulling system can then be used to withdraw plugs (Mei et al. 2011b); or alternatively, plugs can be manufactured by dissolvable materials, which allows opening of drainage holes slowly with time (Mei et al. 2011a). Clogging is often a critical problem for soil consolidation using the technique of vertical drains (Chu et al. 2004). A filtering system is therefore required at drainage openings for permeable piles to avoid the occurrence of clogging. However, no experimental work on permeable piles has been reported.

This paper presents the results of experimental investigation on permeable piles. A comprehensive model-scale laboratory test program on permeable piles embedded in clay is introduced. Drainage holes were drilled at four quadrants around the circumference of permeable pile to provide a drainage path at the soil-pile interface (see Fig. 1(a)). Three soil containers with dimensions of 2 m × 2 m × 1.5 m (i.e., width × length × depth) were fabricated. The results of three series of tests on different drainage conditions, for which the pile type, the pile number, the time to open drainage holes and the discharge condition were varied, are presented. The consolidation behaviour of the surrounding soil in terms of pore water pressure variations was measured by an array of piezometers. In the first series, the efficacy of drainage conditions for a single pile was investigated, where a normal pile was compared to two permeable piles with different opening time of drainage holes. The second series was conducted to evaluate whether the discharge of drained water using the siphon principle could improve the performance of the pile. Finally, pile groups were tested in the third series to evaluate the influence of permeable piles on pore pressure dissipation.

Experimental procedures

Experimental setup

Three soil containers as shown in Figs. 2 and 3(a) have been constructed in the geotechnical laboratory of the Institute of Geotechnical Engineering at Nanjing Tech University. A schematic representation of the soil box is given in Fig. 2, where it has inside dimensions of 2 m wide, 2 m long and 1.5 m high. Three identical containers were fabricated, so that direct comparison tests could be conducted. Four 40 mm thick steel plates were welded together on a base unit of 40 mm thick. Additional steel stiffeners were added to the frames to further increase the rigidity of the sidewalls. The arrangement of steel plates and stiffeners could effectively reduce the sidewall deflection to 1 mm even at a surcharge load of 1 MPa (Brachman et al. 2000). Therefore, the maximum strain within the soil was limited to 0.1% (i.e., the maximum lateral deflection could be 2 mm (two sidewalls) over a span of 2 m).

In this study, plastic model piles were fabricated with an outer diameter (d) of 60 mm. For single pile tests (see Fig. 2(a)), the distance between pile and model boundary was 1000 mm, which corresponded to 16.7d. For pile group tests (see Fig. 2(b)), the distance between side pile and model boundary was 880 mm, which was about 13.3d. The model size of 2 m was designed to ensure that lateral boundaries exerted a negligible influence on experimental measurements. The distance from hard sidewalls in both series of tests was larger than the threshold of 10d (Gui et al. 1998). Similarly, Ng et al. (2015) used a distance of 7d and found that the boundary effects could be neglected. If more piles were considered to form a pile group more than a 2 × 2 pile group, the side piles could be very close to the sidewalls acting as sacrificing piles (Ng et al. 2015). Alternatively, a friction treatment system, such as glass insert panels (Saiyar et al. 2016), a thin film of silicone grease (Lam et al. 2009), and double layers of polyethylene sheets with silicone-based bearing grease lubrication (Ni et al. 2017c), should be applied at sidewalls to simulate the soil of infinite width, which can reduce the boundary effects (Gui et al. 1998).

The soil containers were not only designed for pile tests, but could also be used for other applications. For example, if the performance of small-diameter buried pipes was to be tested, enough width of soil box should be desired to accommodate the pipe's upward and downward motion without undue interference from the lateral boundaries (Ni et al. 2017c). All considerations on the dimensions of soil box for different potential usage were taken into account.

Test piles and backfill soils

The purpose of this investigation is to evaluate the efficacy of the proposed concept of permeable pile in accelerating pore water pressure dissipation in the surrounding soil. Note that the opening of small permeable holes around the pile circumference may slightly degrade the axial strength of the pile, but a permeable pile can still have a strength that corresponds to at least 80% of the value for a normal pile based on the uniaxial compression tests of Ni et al. (2017a). From four point flexural tests, Ni et al. (2017a) also found that permeable piles can have a slightly higher flexural capacity compared to normal piles, since loads could be more uniformly distributed along the pile length. It should be emphasized that openings of drainage holes could potentially result in the deterioration of the pile in the long term, where the risk of using permeable piles in corrosive soils could be increased. However, the current investigation is only focused on the ability of permeable piles to accelerate soil consolidation. The structural performance of permeable piles under complex loading conditions, such as combined axial and lateral loading, cyclic loading or dynamic loading, and the long term performance associated with durability issues are out of the scope of this study.

Plastic pipes of 900 mm long and 60 mm in diameter were used in this study to reproduce the behaviour of piles. Plastic pipes were selected based on the easiness in the fabrication perspective. Permeable piles were manufactured by drilling drainage holes at four quadrants around the pile circumference (see Fig. 3(b)). The diameter of these holes was 6 mm (i.e., 0.1*d*) and the spacing was 36 mm (i.e., 0.6*d*). A conical pile tip joint was butt fusion welded with the pile to model a closed-ended pile, such that soil plugging from the end of a pipe pile was avoided. A pipe sleeve (of slightly smaller diameter) was inserted into the permeable pile to block the drainage path at the soil-pile interface during the pile driving process. The inserted pipe can be removed to control the opening time of drainage holes. The technique of using inserted pipe was to model a mechanical pulling system (Mei et al. 2011b), which can be withdrawn after the completion of pile installation to avoid clogging during pile driving. As shown in Fig. 3(b), geotextiles were used to wrap around the permeable pile to simulate a filtering system to minimize the potential of drainage blockage during the tests. All these measures were to avoid clogging of drainage holes by soil particles.

The clay used in this study was taken from a site of Oufei reclamation project in Longwan District, Wenzhou, China. The particle distribution of the material was determined using a laser diffractometer (Malvern Mastersizer 2000). The material was classified as poorly graded (i.e., more than 50% particles featured a diameter of less than 10 μm), which could result in clogging of drainage holes and vertical drains easily (Wang et al. 2014). Physical properties of the clay were measured as summarized in Table 1. All parameters show reasonable consistency with those reported by Wang et al. (2014). Clay was mixed with water uniformly, and the mixture was then filled in the soil box. After completion of consolidation, instrumentation and pile installation, the water content and undrained shear strength were measured and will be presented subsequently.

Model preparation

The vacuum method has been proven as an effective approach to remove trapped air bubbles to facilitate consolidation of very soft, highly compressive clays over extensive geographic areas for reclamation project (Shang et al. 1998; Wang et al. 2016). Special concerns must be considered when using the technique for model-scale laboratory tests. Porous plastic drains need to be inserted in the model, which provides drainage paths during the vacuum preloading phase. However, pushing plastic drains into the soil directly could potentially cause rupture or undesired deformation of the material. Hence, a squared steel frame was manufactured, on which porous plastic drains were adhered. A spacing of 800 mm was adopted between mandrels to provide evenly distributed drainage paths. The mandrels and drains were then driven into the soil together. Surplus water was kept to reduce the trapped air bubbles within the model as little as possible (Hird and Moseley 2000). A composite system of geotextiles and double polyethylene sheets was used to cover the soil surface to eliminate water evaporation. All vertical drains were connected to a vacuum pump through T-joints and drainage pipes. After assembly, a vacuum pressure was applied to the model overnight (Hird and Moseley 2000). The remaining trapped air bubbles were reported to have negligible influence on the test results after two-hours vacuum for a clay with more than 100% moisture content (Lam et al. 2009).

De-aired distilled water was used in the vacuum preloading process to minimize trapped air bubbles in the soil, which was important to get accurate measurements for piezometers (Strout and Tjelta 2005). After vacuum pressure soaking, the same water was filled in the model. An air bladder (Brachman et al. 2000) was used to spread a surcharge pressure of 50 kPa uniformly across the model. Chu et al. (2004) have reported that a surcharge load could help to shorten the time for consolidation. The full saturation was reached after 2 weeks, once there was no change in the height of the soil (i.e., interpreted using a tape measure attached on the sidewall). The soil was stabilized at a final height of 120 mm, from an initial level of 140 mm before consolidation. Note that the representative stresses induced by pile driving and the impact of pile driving cannot be replicated by this experiment.

Instrumentation

Different techniques have been successfully used to measure the changes in pore water pressure, such as miniature piezocones (Hird et al. 2003), tensiometer (Take and Bolton 2003), pore pressure piezoresistive transducers (Beddoe and Take 2016), short term estimates using CPT or piezoprobe and long term measurement using piezometers (Strout and Tjelta 2005). As mentioned by Take and Bolton (2003), the accuracy of measurements is very much influenced by the states of pre-existing and entered air bubbles. The full saturation in model preparation was to ensure that there is no time-delay in the response of piezometers (Strout and Tjelta 2005), since piezometers were to provide measurements over many days during the tests. The YH-30301A piezometer used in this study has a diameter of 25 mm and a tip length of 170 mm. It can measure gauge pressure up to 100 kPa with a resolution of ±0.05 kPa under a temperature of -20°C~80°C.

The influence of installation methods of piezometers was investigated separately. It showed that different size of piezometers and their orientation could alter the results, which is consistent with the observations of Kutter et al. (1990) that the direction of installation mandrels and rotation of pore pressure transducers influence readings significantly. In addition, the soil reinforcement from mandrels may change the measurements as well. The choice of the installation method of piezometers adopted in this paper was to minimize the impact of instrumentation.

Fig. 2 shows only the centre locations of piezometers. In one test, nine piezometers were installed across the height and width of the soil box. To be specific, three rows of transducers were buried at a depth of 200 mm (i.e., 3.3*d*), 450 mm (i.e., 7.5*d*) and 700 mm (i.e., 11.7*d*) from the ground surface. In the horizontal direction, three columns of sensors were embedded at a distance of 60 mm (i.e., 1*d*), 180 mm (i.e., 3*d*) and 300 mm (i.e., 5*d*) from the pile for the single pile tests, and of -120 mm (i.e., -2*d*, at the centre of pile group), 0 mm (i.e., 0*d*, at the edge of pile group) and 120 mm (i.e., 2*d*, outside the pile group) from the side pile for the pile group tests, respectively. Three columns of piezometers were attached to a 1 m long steel mandrel at specific locations, which was to guide accurate installation of these sensors. The driving of three mandrels was conducted after model preparation to minimize interference with soil movement during consolidation (Beddoe and Take 2016).

After all piezometers had been driven into the model, the surcharge pressure of 50 kPa was resumed until a new equilibrium was reached. This was because the installation of instrumentation could introduce disturbance to the soil (Hird and Moseley 2000). Pore pressure readings were taken every 2 hours. Once the readings became steady, the surcharge loading was removed. The pore pressures were recorded for use in further monitoring of their variations during and after pile driving. The measurements of soil properties after model preparation and instrumentation are tabulated in Table 2 and were obtained from a separate trial test. Note that the observed values fell within the measured ranges of Wang et al. (2014). If the rotation of shear vane is too fast, viscous effects can increase the measured undrained shear strength; likewise, if the rotation is too slow, consolidation occurs and the response is not undrained. Hence, the result is strongly dependent on rate of rotation and also presence of any surface water that can flow into the hole along the shaft, leading to reduced values (Mayne 1985a, b; Peuchen and Mayne 2007). The rotation of shear vane was carefully controlled to measure the representative undrained shear strength values. For the pile tests, the water content and undrained shear strength were not measured again to avoid disturbance to the soil.

Testing program

Similarly, a steel mandrel was fabricated to guide vertical installation of driven piles at a rate of 5 cm/s into the bored hole. Dyson and Randolph (2001) investigated the influence of pile installation method on the degree of densification in the surrounding soil, and found that jacking the pile could result in an intermediate degree of local densification compared to a lower bound of preinstalling and an upper bound of driving. The pressed-in method employed in this investigation may underestimate the degree of densification compared to pile driving, thereby the generation of excess pore pressure. In addition, plastic pipes were used to reproduce the behaviour of permeable piles, which could be significantly influenced by the strain-rate dependent effect of the material if piles were driven into the ground. Therefore, a constant pressing velocity was chosen to (a) evaluate the ability of permeable piles to accelerate soil consolidation under moderate local hydraulic gradients at the soil-pile interface, and (b) to minimize the impact of cyclic loading on material's response. Further studies should be performed to assess the influence of pile installation method.

The depth of pile penetration was controlled to be 700 mm. Figs. 3(c) and 3(d) show the photos captured after completion of pile driving for the single pile and pile group tests, respectively. Since piles were pushed into the saturated soft clay within a short period of time, the surrounding soil experienced undrained shearing. However, the pipe sleeve was inserted to block drainage holes during pile driving, so that the initial pore pressures for normal and permeable piles could be comparable (i.e., consolidation was not allowed during pile driving for permeable piles as well). The initial values of pore pressures were measured immediately after pile installation. In the first 3 hours, readings were taken every 20 minutes. It switched to 2 hours per measurement and 4 hours per measurement after 3 hours and 24 hours respectively until a full dissipation was obtained.

Three series of tests were conducted, where the drainage conditions were varied. The testing procedures for all seven tests with different test type, pile model, opening time of drainage holes and discharge condition are summarized in Table 3. In test series 1, Tests 1, 2 and 3 were comparable, where the effectiveness of drainage holes in enhancing pore pressure dissipation for a single pile was investigated. After pile driving, pore pressure did not reach the peak immediately, whereas there was a time-delay in the response of pore pressure generation, especially at far distance from the pile. Therefore, the opening time of drainage holes was varied in Tests 2 and 3. In test series 2 (Tests 4 and 5), the influence of discharge condition was studied. The drained water collected in the permeable pile could flow upward above the water surface in an inverted ‘U’ shape tube, with no pump, and finally discharge at a lower level in a burette, based on the siphon principle, as shown in Fig. 1(b). In Test 4, every 5 hours, a displacer type level indicator (operating on Archimedes' principle) was used to measure the height of drained water. In Test 5, the drained water was discharged every 5 hours to the burette to read the volume of discharged water. In test series 3 (Tests 6 and 7), the effects of permeable piles on pore water dissipation for a 2×2 pile group (i.e., pile spacing of 240 mm or 4*d*) were evaluated.

It should be pointed out that the current model scale tests were conducted under the 1 g environment. The stress field within the soil reproduced during the tests were not fully representative of the prototype-scale behaviour. A scaling law can be employed to interpret the measured response, but cautions must be taken. Therefore, this investigation reports all measurements without applying a scale factor to avoid misleading. The purpose of this study is to evaluate the potential of using permeable piles to accelerate the dissipation of excess pore water pressure in the surrounding soil. Further investigations can be carried out to assess the behaviour of permeable piles under high local hydraulic gradients at the soil-pile interface in the prototype scale.

Experimental results

Test series 1 – opening time of drainage holes for single pile tests

Upon the completion of pile driving, the initial pore pressures were measured as presented in Table 4. It can be seen that all piezometers provided very consistent results in different tests. The maximum difference in pressure readings was 0.2 kPa, which was only slightly larger than the accuracy of piezometers of ±0.1 kPa. This error level was reasonable due to the variation in three tests. For example, if the accurate measurement at piezometer P3 was 4.1 kPa, the reading could vary from 4.0 kPa to 4.2 kPa given the resolution of the instrument. The pore pressure recorded at P3 in Test 1 lied within 5.0% and 2.5% of values measured from Tests 2 and 3, respectively. This demonstrated the repeatability of the tests.

In the following, variations of pore pressures were adjusted using the average initial values measured in three tests for better illustrative comparisons. Fig. 4 depicts the dissipation of excess pore water pressure with time for normal pile in Test 1 and permeable piles in Tests 2 and 3. In each subfigure, the measurements obtained from piezometers at a specific distance from the pile were reported to study the impact of depth. The same ordinate scale was used to illustrate the difference in magnitude of pore pressure as a function of distance. Similarly, the same abscissa scale was adopted to show the required time for full dissipation of excess pore pressure at different distance from the pile.

In Fig. 4(a), there was a slight increase in pore pressures at a distance of $r = 1d$ once the tests were initiated. An obvious time-delay in the response of pore pressure generation was observed at far distance (i.e., $3d$ and $5d$) from the pile. Pore pressures peaked after approximately 1.5 hours since the start of the tests, Figs. 4(b) and 4(b). Pile driving influenced the soil within a shorter distance immediately, but the response was delayed for the far-field soil. The increase of pore pressures from the initial values to the peak was calculated in Table 5. Tests 1 and 2 presented very consistent calculations, since the opening time of drainage holes was set after 1.5 hours of pile installation in Test 2. During this period, the peak pore pressures had already occurred. Before removing the pipe sleeve, a permeable pile in Test 2 was actually equivalent to a normal pile in Test 1. It is interesting that a negligible increase of pore pressure was observed in Test 3. For the time-delay effect, the drainage condition in Test 3 allowed the dissipation of pore pressure to occur immediately, which restrained the generation of pore pressure with time.

Control of the opening time of drainage holes could reduce the increase of pore pressure to facilitate consolidation as much as possible.

Both Tests 2 and 3 demonstrated a beneficial influence of permeable piles on accelerating consolidation. The measured pore pressures from the permeable pile tests were always lower than those obtained from the normal pile test, Fig. 4. The immediate drainage condition in Test 3 presented much better performance in reducing pore pressures (e.g. after 10.5 hours of pile installation, the pore pressures at P3 in Tests 2 and 3 were 14.6% and 24.1% lower than that measured in Test 1, respectively). The faster dissipation of pore pressures, especially at early stages, could potentially reduce the adverse impact of pile driving on existing piles and adjacent infrastructure, and accelerate the construction process. Therefore, drainage holes should be opened immediately after completion of pile installation for permeable piles.

At a given distance from the pile (e.g., $r = 3d$ at Fig. 4(b)), pore pressures were increased with depth, as well as the time-delay effect. Larger values were obtained at a shorter distance (comparing P3, P6 and P9 in Fig. 4). Roy et al. (1981) suggested that an influencing zone of pore pressure generation could be as far as $r = 3d$. This investigation shows that pore pressures were not zero even at a distance of $5d$, so a large range of $15d$ (Hwang et al. 2001; Ni et al. 2017b) seems to be a more reasonable estimation. The maximum pore pressures were obtained at piezometer P3, as it was placed at a greater depth and a shorter distance. In contrary, piezometer P7 measured the minimum values due to its shallower depth and farther distance. The rate of dissipation depends very much on the peak values of pore pressure. At $r = 5d$, the time required for full dissipation was much shorter, even through the larger distance from the pile may be considered as a longer drainage path.

Comparing to a normal pile, a considerable reduction of pore pressures can be seen for a permeable pile in piezometers P1, P2, and P3 at $r = 1d$ (see Fig. 4(a)), P5 and P6 at $r = 3d$ (see Fig. 4(b)) and P9 at $r = 5d$ (see Fig. 4(c)), respectively. Drainage holes actually helped dissipation of pore pressures in other piezometers, but the beneficial effect was not apparent. An approximate influencing zone of a permeable pile is therefore sketched in Fig. 5, which is an increasing function of depth. The profile of influencing zone is consistent with the trend of pore pressure generation due to pile driving, which also increases with depth and peaks at the pile tip (Ni et al. 2017b). At the pile tip of 11.7d, a permeable pile can help to accelerate pore pressure dissipation at a distance of at least 5d. This is of particular importance since pore pressure decreases with distance exponentially (a logarithmic function of distance), although it could influence a range within 15d (Hwang et al. 2001; Ni et al. 2017b).

Test series 2 – discharge condition for single pile tests

As the immediate drainage condition after pile driving provided a much more efficient solution to accelerate dissipation of pore pressure in test series 1, the following test series adopted the strategy of opening drainage holes once the permeable pile had been installed. Test series 2 was conducted to evaluate the performance of permeable piles under different discharge conditions. Measurements of water drainage were taken every 5 hours.

The volume of drained water in Test 4 can be compared to that discharged in the burette in Test 5 directly as illustrated in the bar chart in Fig. 6. In the first 5 hours of the tests, the two permeable piles in Tests 4 and 5 had exactly the same drainage condition. The same amount of water was therefore collected in the two tests. In Test 5, discharge was carried out at 5 hours mark and was allowed at an interval of 5 hours using the siphon principle, so that the drainage condition was different from that in Test 4 since then. It can be seen that the volume of drained water within 5 hours reduced with time in both tests. This conforms to the Terzaghi consolidation theory that soil deforms with a reduction of pore pressure dissipation over time. Throughout the tests, the drained water for the permeable pile with discharge condition was always more than that obtained from the test under no discharge condition. This demonstrated that the collected drained water within the permeable pile (i.e., Test 4) could limit the dissipation of pore pressures. After 55 hours, the volume of drained water in Test 5 was 6 mL, and the value of 7 mL was obtained in Test 4 after 60 hours. There was negligible drained water after the corresponding time in the two tests, which could be regarded as the total time required for consolidation. Therefore, the discharge condition could accelerate dissipation of pore pressures, but the influence was not apparent.

Fig. 7 presents the variations of total drained water with time. More water was drained when discharge was allowed, especially at later stages of the tests. At early stages, the difference in total drained water from the two tests was not significant. A measure of discharge capacity could be computed using the initial linear part of the curves as 1.1×10^{-5} m^3/s and 1.2×10^{-5} m^3/s in Tests 4 and 5, respectively. Basically, the influence of discharge condition on drainage capacity was negligible. It is interesting that the estimated discharge capacity for permeable piles was slightly less than the value of 2.5×10^{-5} m^3/s for conventional prefabricated vertical drains (PVDs) and the value of 5.0×10^{-5} m^3/s for integrated PVDs (Wang et al. 2016). It should be noted that the discharge capacity estimated for PVDs was based on field measurements under high local hydraulic gradients at the soil-pile interface. The model scale tests in this investigation cannot reproduce the much higher hydraulic gradients well, but the discharge capacity for permeable pile is still acceptable for use as a drainage path to accelerate soil consolidation. The drainage improvement by discharge was calculated as illustrated using the secondary ordinate. It is clear that the drainage condition was improved slightly in the beginning of the tests, and the improvement was raised steadily up to a plateau of approximately 6%. The difference was attributed to the water height within the permeable pile. At the end of consolidation, the total drained water was 1224 mL and 1290 mL in Tests 4 and 5, respectively. It raises a question whether the discharge condition is necessary in practice or not to make an impact of 6% improvement in the total volume of drained water, given the complexity involved in the test.

The measured pore pressures using piezometers are plotted against time in Fig. 8, where the responses of permeable piles with and without discharge are compared. It can be seen from the figures that there was not much significant difference between two curves measured by piezometers P1, P4 and P7 at shallow depth. At mid-depth, piezometers P2, P5 and P8 provided slightly different measurements from the two tests, and the difference became more obvious at smaller distance from the pile (i.e., the impact of discharge condition: P2 at $r = 1d >$ P5 at $r = 3d >$ P8 at $r = 5d$). The contribution of discharge condition was most significant at greater depth, where pore pressures at piezometers P3, P6 and P9 dissipated faster in Test 5 than in Test 4. Similarly, the location of piezometers was important, as the difference at larger distance from the pile was vague (i.e., P3 at $r = 1d >$ P6 at $r = 3d >$ P9 at $r = 5d$).

The Terzaghi consolidation theory considers that the reduction in soil volume occurs due to water drainage from the soil. Therefore, excess pore pressures dissipate over time until a balance between effective stress and external pressure is reached. However, no attention has been given to where the drained water goes and whether the drained water affects further consolidation. For permeable piles, the drained water is collected within the pile. The accumulated water increases the pressure head, where the associated impact on pore pressure dissipation needs to be assessed.

The water height was back calculated from the volume of drained and discharged water in Tests 4 and 5, respectively. Fig. 9 shows the water level within the permeable piles. Piezometers P3, P6 and P9 were buried at the pile tip, and the distance from transducers P2, P5, and P8 and from sensors P1, P4 and P7 to the pile tip was 250 mm and 500 mm respectively, Fig. 2. It is interesting that the discharge condition in Test 5 helped to accelerate pore pressure dissipation at P1 after about 25 hours since the start of the test (see Fig. 8(a)). At 25 hours mark, the water level reached 484 mm in Test 4 (see Fig. 9), which was very close to the location of P1 at 500 mm above the pile tip. Similarly, pore pressures measured at P2 and P5 started to show increased dissipation due to water discharge in Test 5 after approximately 10 hours of pile driving (see Figs. 8(a) and 8(b)). At 10 hours mark, the water level in Test 4 was at 237 mm (see Fig. 9), which was slightly lower than the height of P2 and P5 of 250 mm from the pile tip. At the pile tip, the enhanced dissipation of pore pressures in piezometers P3, P6 and P9 was already observed in Test 5 after around 5 hours (see Figs. 8(a), 8(b), and 8(c), at which the water height of 125 mm (see Fig. 9) was above these transducers.

Therefore, it is reasonable to suppose that the pressure in the permeable pile without discharge increased due to the accumulated drained water, which reduced the pressure head for pore water flowing through drainage holes and slowed down the dissipation of pore pressures in the surrounding soil. This assumption can be well justified by the difference between Tests 4 and 5 (without and with discharge), which could be roughly correlated with the drained water level within the permeable piles. Again, piezometers P4, P7 and P8 presented negligible difference due to the discharge condition, Fig. 9, as they were located outside the approximate influencing zone of a permeable pile as given in Fig. 5. However, the beneficial effect of allowing discharge was not apparent. Given the complexity of operation, discharge is not recommended in practice.

Test series 3 – pile group tests

In the pile group tests, four piles were installed in sequence as shown in Fig. 2(b). The pipe sleeves were removed together to open drainage holes after pile installation. This was to guarantee the same initial conditions of pore pressure generation. The construction sequence influences the distribution of pore pressures, where the mechanism is complex. In this paper, the efficacy of permeable pile group in accelerating dissipation of pore pressures is to be evaluated. Therefore, the variations of pore pressures during pile driving will only be presented briefly.

Fig. 10 displays the measured pore pressures during the installation process and 3 hours after completion of pile driving. In general, pore pressures increased with the number of installed piles. When the first pile was placed, it corresponded to a single pile test, where larger pore pressures were generated at a shorter distance from the pile (e.g., P1 > P4 > P7, P2 > P5 > P8, P3 > P6 > P9). As the depth increased from 3.3*d*, 7.5*d* to 11.7*d*, pore pressures were raised significantly (e.g., P3 > P2 > P1, P6 > P5 > P4 and P9 > P8 > P7). Upon the installation of the second pile, pore pressures became greater, but the magnitude of increase was less than that observed for the first pile. After the third pile, pore pressures at the edge of pile group increased drastically (see P4, P5 and P6 in Fig. 10(b)). This was because the distance from piezometers P4, P5 and P6 to the third pile was smallest. The fourth pile induced a reduction of pore pressures at greater depth at the centre (i.e., P2 and P3) and the edge (i.e., P5 and P6) of pile group, respectively, whereas, other piezometers measured increased pore pressures. The two tests (Tests 6 and 7) provided comparable measurements of pore pressures that differed by less than 0.2 kPa (within the accuracy range of ±0.1 kPa). Both normal and permeable pile groups presented the pressure reduction at the end of installation, which demonstrated that the tests were repeatable.

It is reasonable that the difference between the two tests became larger after 3 hours, Fig. 10, since the drainage condition was altered for the permeable pile group. Due the time-delay effect, pore pressures peaked at a later stage, which was approximately 3 hours for both tests. The magnitude of increase in pore pressures after the installation of pile groups is quantitatively tabulated in Table 6. Dissipation occurred directly after pile driving at some piezometers with no increase of pore pressures (P2 and P5 in Test 6, and P1, P2, P5 and P7 in Test 7). It is clear that the increase of pore pressures in Test 7 was much less than that in Test 6. Allowing drainage in permeable piles reduced the pore pressure generation effectively.

The comparisons of pore pressure dissipation obtained from normal (Test 6) and permeable (Test 7) pile group tests are presented in Fig. 11. The curves of pore pressures versus time for permeable piles were below those for normal piles, which confirmed the effectiveness of permeable piles in providing drainage paths to accelerate consolidation of the surrounding soil. After peak pore pressures, the slope of these dissipation curves was very steep initially, reflecting a rapid consolidation rate, which followed by a gentle slope at later stages. The beneficial effect of permeable piles was distinct at early stages, since there was a dramatic gap between the two curves measured in Tests 6 and 7. However, the gap reduced with time. The drainage condition of permeable piles did not change the total time required for full dissipation of excess pore pressures too much in the end of the tests. The lower the peak pore pressures, the faster the occurrence of full dissipation.

Similar to single pile tests, the best improvement of pore pressure dissipation by permeable piles was obtained at greatest depth (i.e., P3, P6 and P9), and the benefits reduced as the depth became shallower (e.g., P3 > P2 > P1, P6 > P5 > P4 and P9 > P8 > P7), as illustrated in Fig. 11. This was partially because larger pore pressures were generated at greater depth, where the changes induced by the drainage condition were more obvious. It is interesting that the improvement by permeable piles in pile group tests was more significant than that in single pile tests due to the combined effect of multiple piles (e.g. after 23 hours of pile driving, a reduction of pore pressure of approximately 31.3% was obtained for the pile group test in Test 7 compared to that measured in Test 6. In single pile tests, the reduction was below 25%). Fig. 11 also shows that permeable piles helped to accelerate dissipation of pore pressures much more at the centre of pile group (i.e., P1, P2, and P3) compared to other locations. Four piles could make equal contribution, as piezometers at the centre of pile group had the same distance from piles (same drainage paths). On the other hand, two piles contributed more to reduce pore pressures at the edge of pile group, as well as outside the pile group. At the farthest distance (i.e., P7, P8 and P9), permeable piles exerted the least influence on pore pressure dissipation due to the longest drainage path.

Conclusions

Displacement piles cause soil disturbance, where both undesired deformations (ground heave, settlement and radial displacement) and excess pore water pressures may result in damage of existing piles and adjacent infrastructure. Measures that can facilitate dissipation of pore pressures to achieve a rapid consolidation of the surrounding soil are of interest to practising engineers. A conceptual model of permeable pile has been tested experimentally using three model-scale soil containers in this paper. Model piles with an outer diameter of 60 mm were used to reproduce permeable piles by drilling drainage holes around the pile circumference. The fabricated piles were buried in saturated clay, where the pile type, the pile number, the opening time of drainage holes, and the discharge condition were varied. The benefits of permeable piles were evaluated by comparing the pore pressure variations measured using an array of piezometers for both single pile and pile group tests. Further studies should be performed to assess the structural performance of permeable piles under complex loading conditions, as well as the long term performance before the technique becomes an established practical approach.

After pile driving, there was a time-delay in the response, where pore pressures were generated slowly, especially at far distance from the pile. In single pile tests, a permeable pile could effectively accelerate dissipation of pore pressures. This was partially because, upon the completion of pile installation, pore pressures increased for a normal pile, during which drainage was allowed in a permeable pile that restrained the generation of pore pressures. The lower the peak pore pressures, the faster the dissipation. It was proven that the immediate drainage condition after pile driving was preferable.

The discharge condition was also evaluated, where the drained water was measured in the permeable pile or in the burette (discharged from the pile). The drained water level in the permeable pile could be roughly correlated with the pressure head to restrain pore water flowing though the drainage holes. The higher the water level was, the slower pore pressures dissipated. The full dissipation was achieved faster for the permeable pile with discharge, but the beneficial effect was not apparent. Considering the complexity involved in the discharge operation, it is not suggested to discharge the drained water from the permeable pile using the siphon principle.

In pile group tests, the influence of permeable piles on accelerating dissipation of pore pressures was much clearer than that in single pile tests. This was due to the combined contribution from all piles. It is not surprising that the best improvement by permeable piles was obtained at the centre of pile group, since the distance to all piles were small. This is consistent with the results of single pile tests that the best dissipation of pore pressures was observed at the least distance from the pile. An approximate influencing zone of permeable pile was evaluated in both single pile and pile group tests, which was an increasing function of depth. At the pile tip, a permeable pile could help to accelerate pore pressure dissipation at a considerable distance of as far as 5 times the pile diameter.

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Figure Captions

Fig. 1. Schematics of (a) a permeable pile; (b) the discharge of drained water using the siphon principle

Fig. 2. Physical model of piles embedded in clayey soil: (a) single pile test; (b) Pile group test

Fig. 3. Annotated photo of test setup: (a) soil box; (b) normal and permeable piles; (c) single pile test; (d) pile group test

Fig. 4. Dissipation of excess pore water pressure for normal (Test 1) and permeable (Tests 2 and 3) piles at a distance of: (a) $r = 1d$; (b) $r = 3d$; (c) $r = 5d$

Fig. 5. The approximate influencing zone of a permeable pile

Fig. 6. Volume of drained water for permeable piles with and without discharge

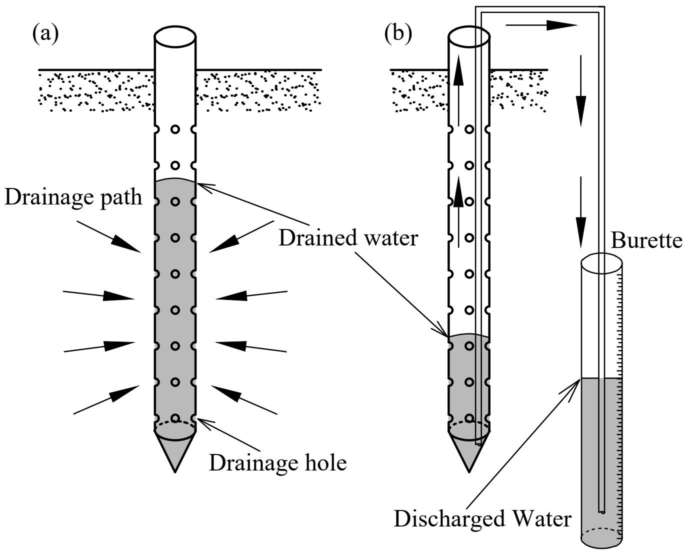
Fig. 7. Drainage improvement by allowing discharge for permeable piles

Fig. 8. Dissipation of excessive pore water pressure for permeable piles with and without discharge at a distance of (a) $r = 1d$; (b) $r = 3d$; (c) $r = 5d$

Fig. 9. Water level within the permeable piles with and without discharge

Fig. 10. Generation of excess pore water pressure during pile driving: (a) at the centre of pile group; (b) at the edge of pile group; (c) outside the pile group

Fig. 11. Dissipation of excess pore water pressure for normal (Test 6) and permeable (Test 7) pile groups: (a) at the centre of pile group; (b) at the edge of pile group; (c) outside the pile group



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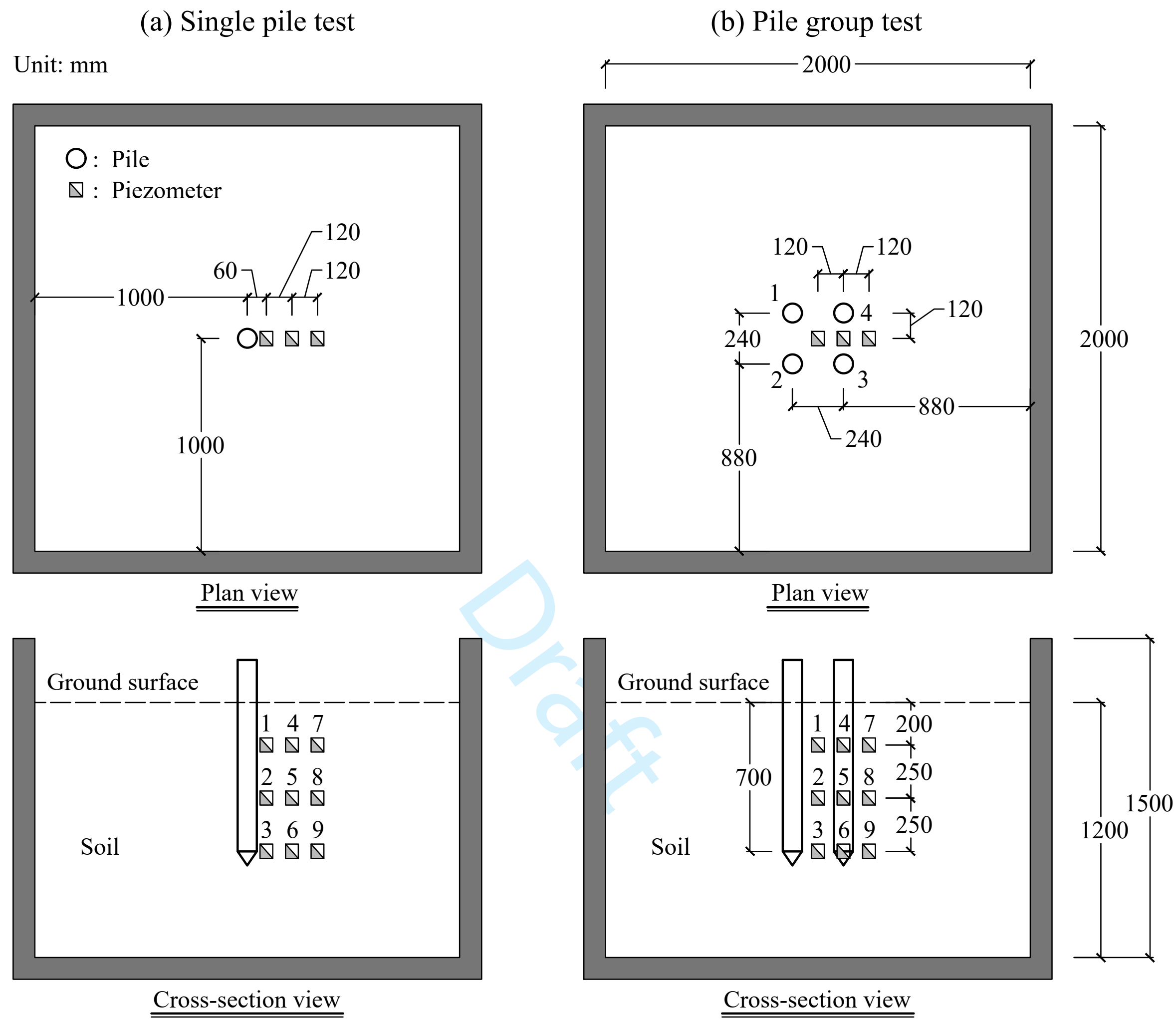


Fig. 2. Physical model of piles embedded in clayey soil: (a) single pile test; (b) Pile group test

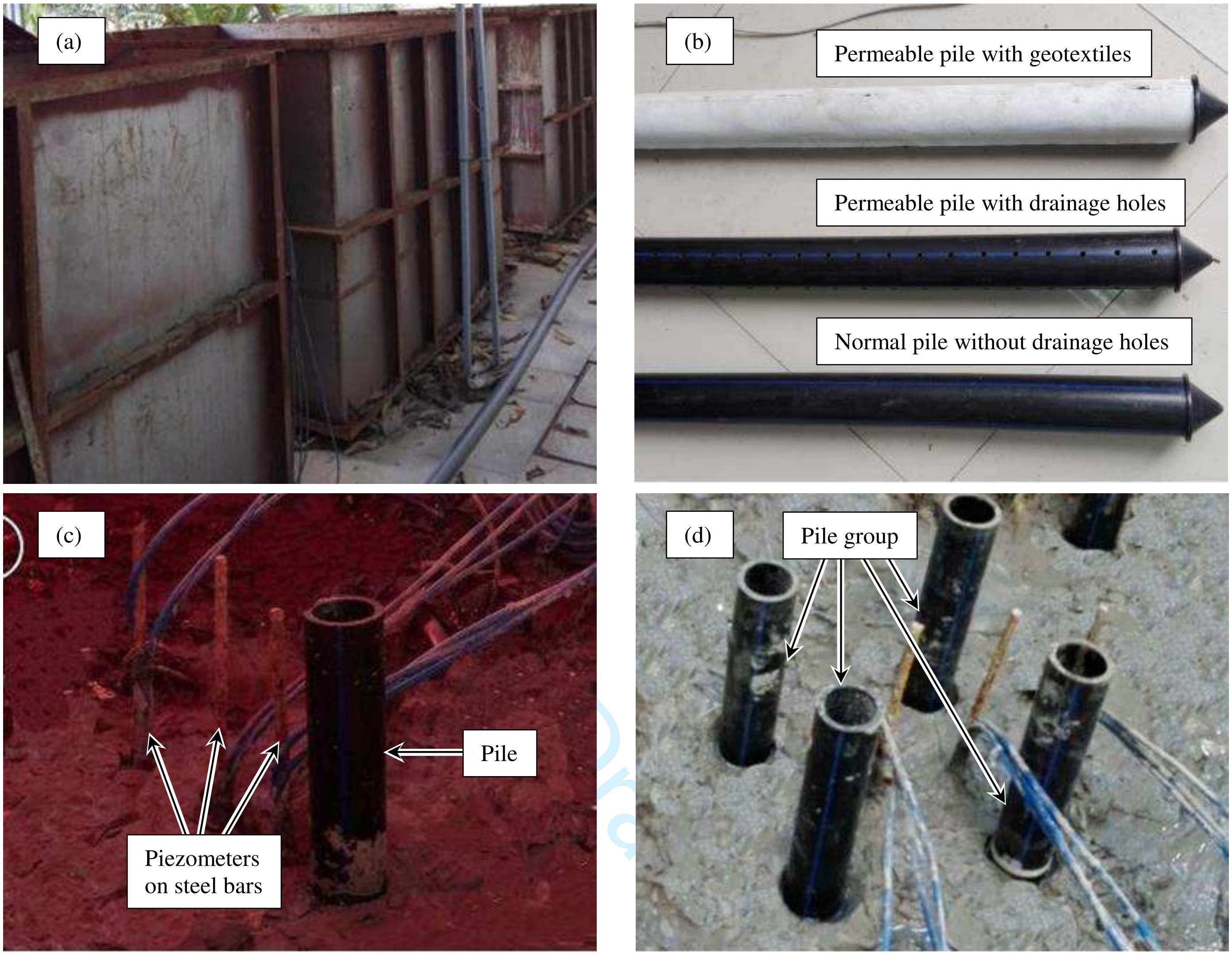


Fig. 3. Annotated photo of test setup: (a) soil box; (b) normal and permeable piles; (c) single pile test; (d) pile group test

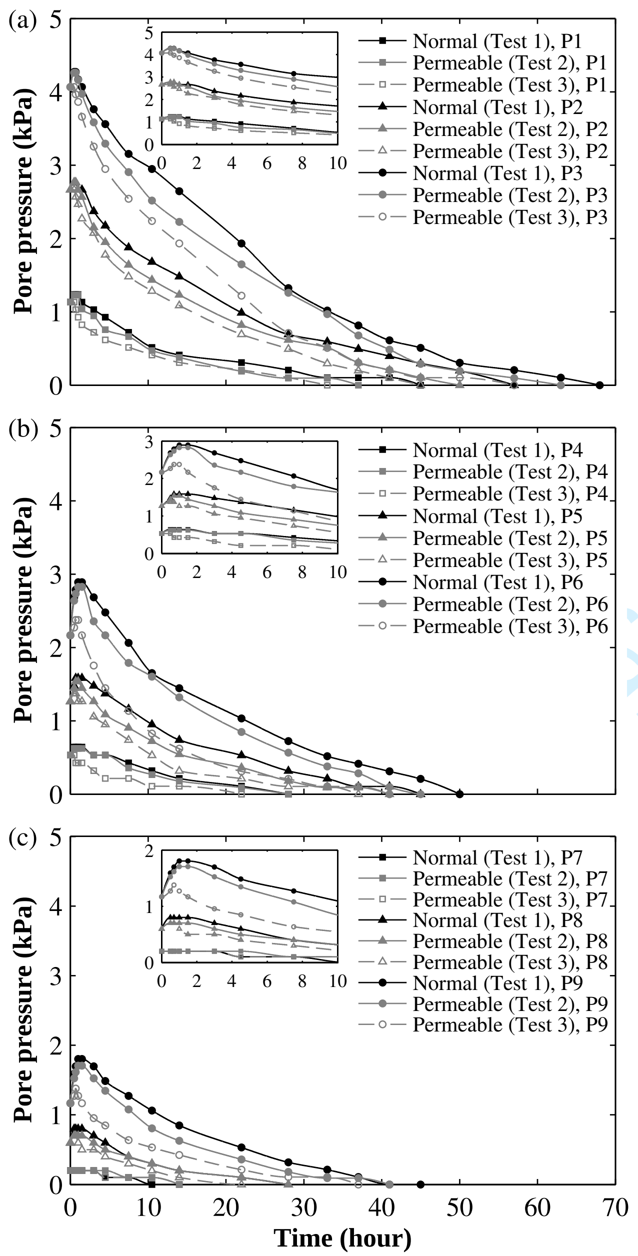
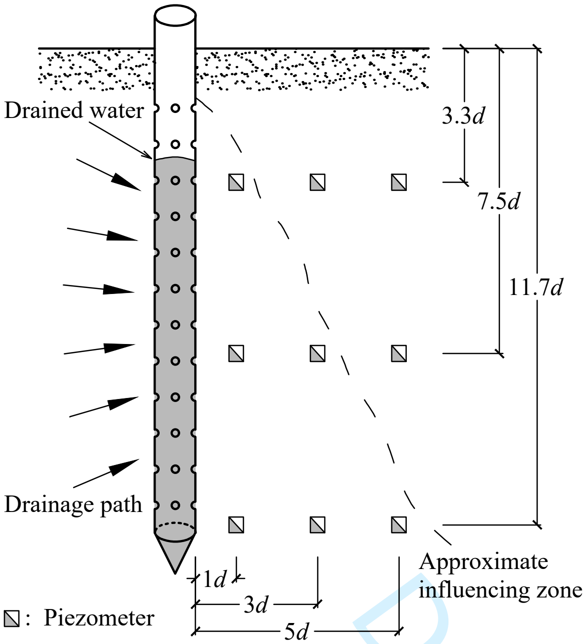


Fig. 4. Dissipation of excess pore water pressure for normal (Test 1) and permeable (Tests 2 and 3) piles at a distance of: (a) $r = 1d$; (b) $r = 3d$; (c) $r = 5d$



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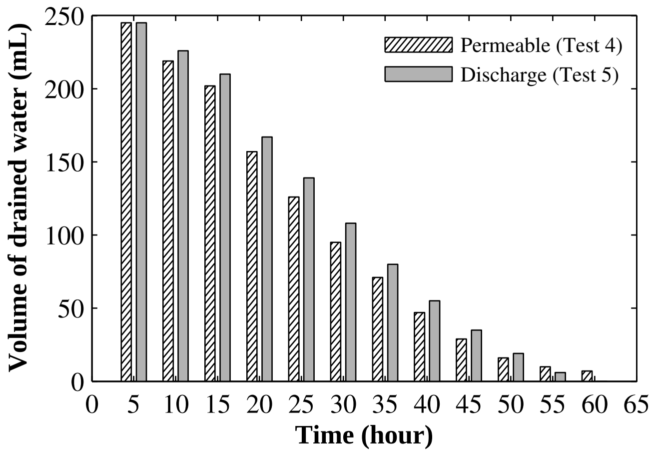

Fig. 6. Volume of drained water for permeable piles with and without discharge

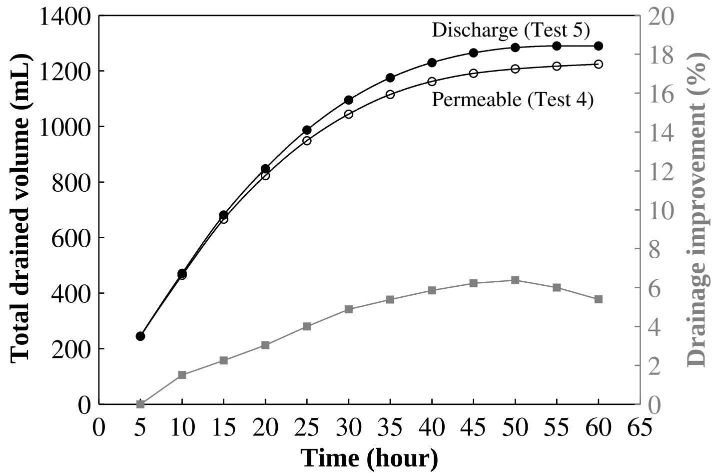

Fig. 7. Drainage improvement by allowing discharge for permeable piles

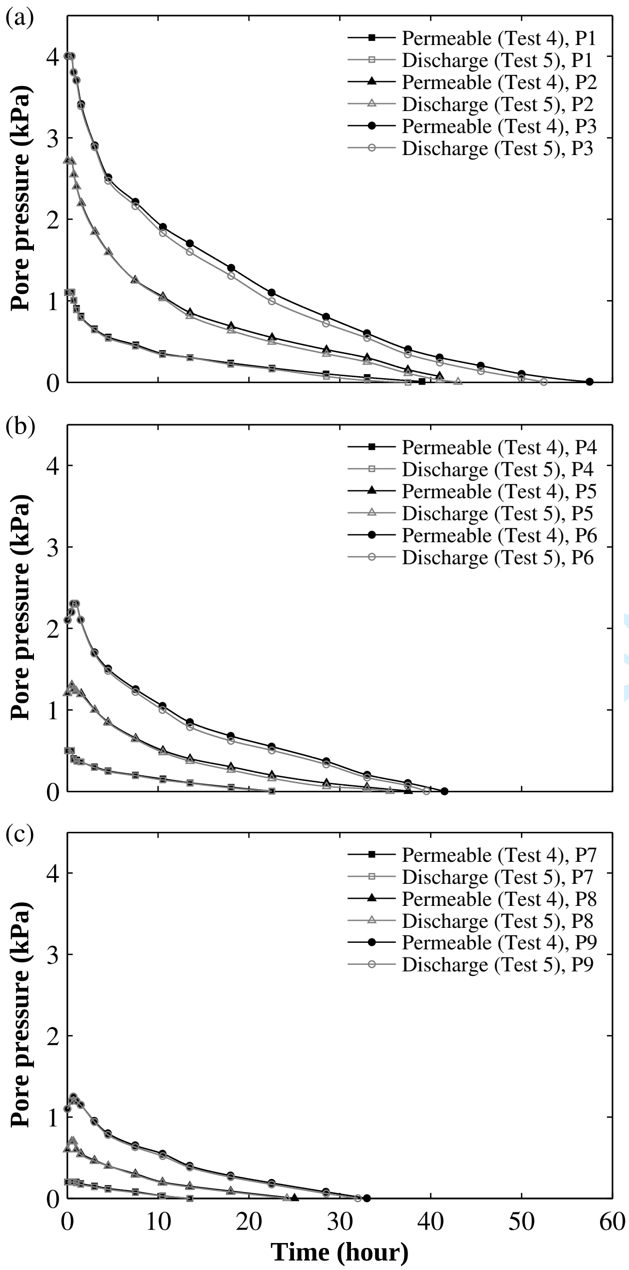

Fig. 8. Dissipation of excessive pore water pressure for permeable piles with and without discharge at a distance of (a) $r = 1d$; (b) $r = 3d$; (c) $r = 5d$

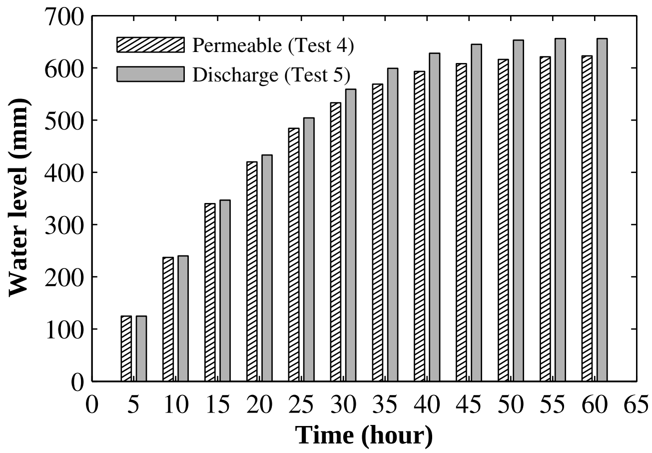

Fig. 9. Water level within the permeable piles with and without discharge

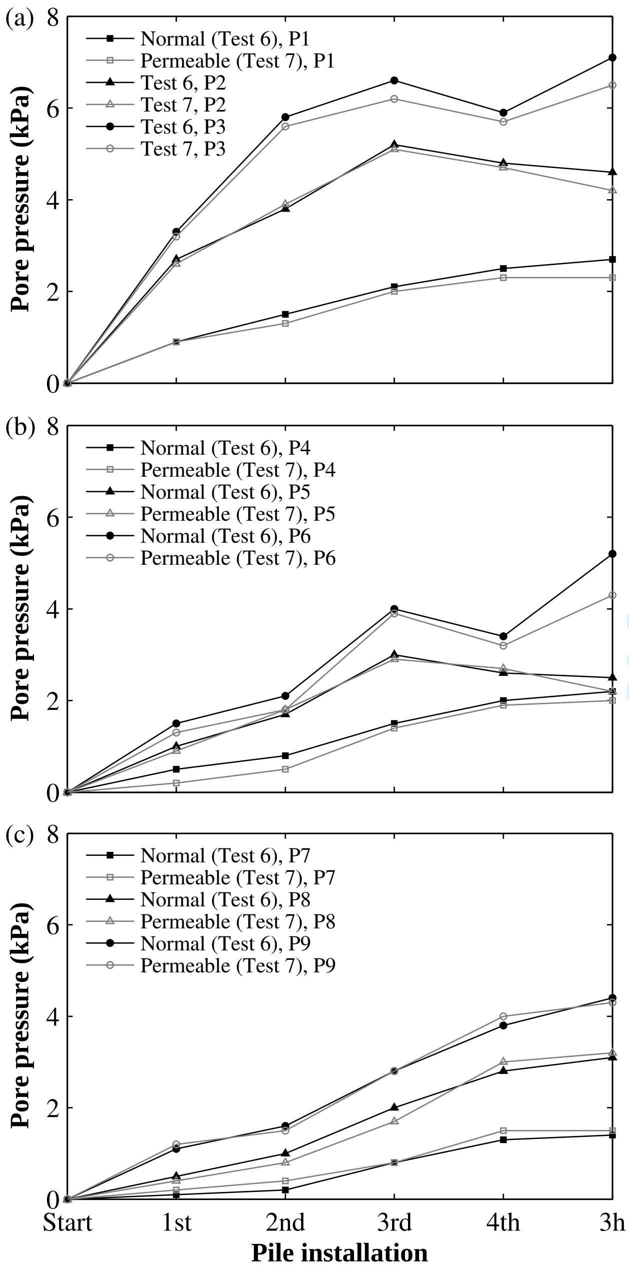

Fig. 10. Generation of excess pore water pressure during pile driving: (a) at the centre of pile group; (b) at the edge of pile group; (c) outside the pile group

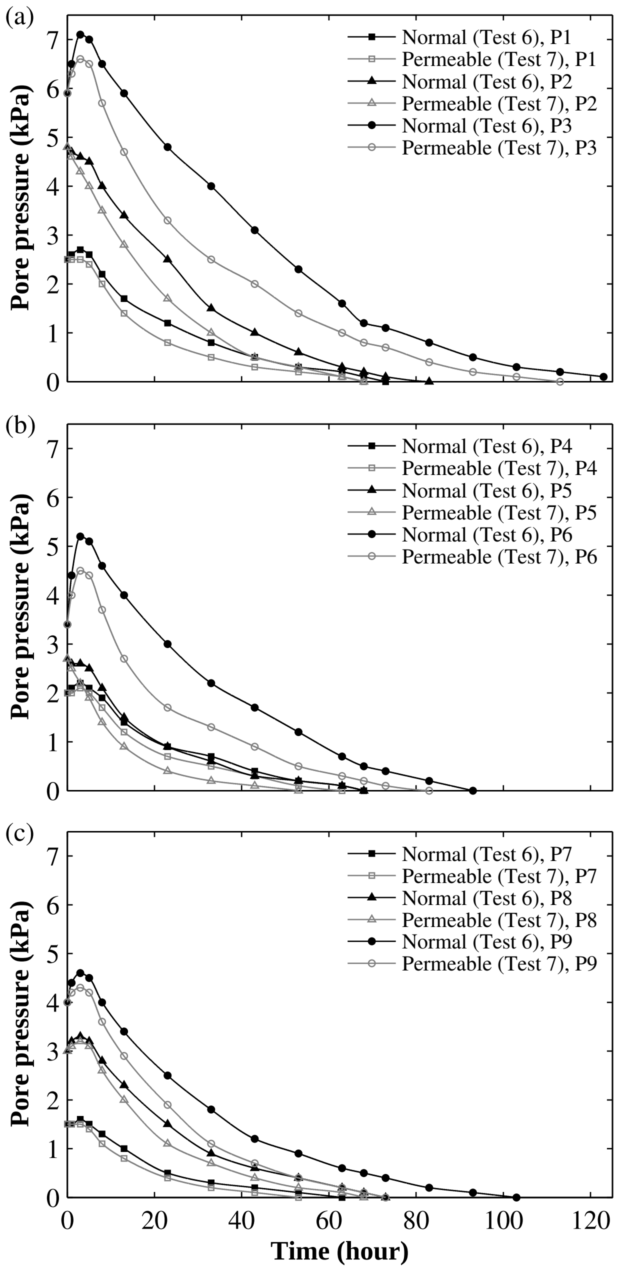

Fig. 11. Dissipation of excess pore water pressure for normal (Test 6) and permeable (Test 7) pile groups: (a) at the centre of pile group; (b) at the edge of pile group; (c) outside the pile group

Table 1. Physical properties of the clay

Properties	Water content (%)	Unit weight (kN/m^3)	Specific gravity	Void ratio	Degree of saturation (%)	Plastic limit (%)	Liquid limit (%)
Measured	96.9	14.42	2.71	2.73	99.5	35	73
Wang et al. (2014)	109.4	14.43	2.73	2.73	99.6	27	56

Table 2. Measured soil properties after model preparation and instrumentation

Properties	Depth (mm)	Measuring points								Average	Wang et al. (2014)
		1	2	3	4	5	6	7	8		
Water content (%)	0	46.0	46.4	44.3	44.7	46.4	44.7			45.4	45.6-53.1
	200	51.9	51.8	50.8	53.6	50.2	51.3			51.6	
	400	52.4	53.9	53.0	45.8	51.2	52.0			51.4	
Undrain shear strength (kPa)	0	17.7	16.5	17.6	18	17.9	17.5	17	14.7	17.1	9.2-18.9
	200	15.1	14.2	12.8	12.3	16.5	13.7	13.4	12.9	13.9	
	400	11.5	11.1	10.6	9.4	14.3	11	11.6	11.7	11.4	
	600	10	9.2	8.2	8.6	12.8	10.6	9	8.6	9.6	

Table 3. Test program

Series	Test	Test type	Pile model	Opening time of drainage holes	Discharge condition
1	1	Single pile	Normal	N/A	N/A
	2	Single pile	Permeable	1.5 hours after pile driving	No
	3	Single pile	Permeable	Immediate after pile driving	No
2	4	Single pile	Permeable	Immediate after pile driving	No
	5	Single pile	Permeable	Immediate after pile driving	Yes
3	6	Pile group	Normal	N/A	N/A
	7	Pile group	Permeable	Immediate after pile driving	No

Table 4. The initial values of pore water pressures after pile driving in test series 1

Piezometer	P1	P2	P3	P4	P5	P6	P7	P8	P9
Test 1	1.1	2.7	4.0	0.5	1.2	2.1	0.2	0.6	1.1
Test 2	1.2	2.6	4.2	0.6	1.4	2.3	0.2	0.6	1.3
Test 3	1.2	2.7	4.1	0.6	1.3	2.2	0.3	0.7	1.3

Table 5. The increase of pore water pressures after pile driving in test series 1

Piezometer	P1	P2	P3	P4	P5	P6	P7	P8	P9
Test 1	0.1	0.1	0.2	0.1	0.3	0.7	0	0.2	0.6
Test 2	0.1	0.1	0.2	0.1	0.3	0.7	0	0.1	0.6
Test 3	0	0	0	0	0.1	0.2	0	0.1	0.2

Table 6. The increase of pore water pressures after pile driving in test series 3

Piezometer	P1	P2	P3	P4	P5	P6	P7	P8	P9
Test 6	0.2	0	1.3	0.2	0	1.8	0.1	0.3	0.6
Test 7	0	0	0.8	0.1	0	1.1	0	0.2	0.3