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Physical modeling and analytical derivation of soil consolidation around permeable pipe pile

Modélisation physique et dérivation analytique de la consolidation du sol autour du pieu perméable

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ABSTRACT: During pile driving, excess pore water pressures can be generated in the surrounding soil. A pile can gain capacity with time (i.e., the set-up effect) due to soil consolidation. However, the consolidation time is often very long, which hinders the use of full pile bearing capacity at the early construction stage, leading to conservative design. A novel concept of permeable pipe pile is proposed to provide drainage channels by opening drainage holes around the pile circumference, through which pore pressures can be dissipated. A model-scale testing program is conducted under the controlled laboratory condition, in which the responses of a normal pile and two permeable piles are measured to demonstrate the efficacy of introducing drainage channels along the pile length. The use of permeable pile is found to be capable of accelerating the consolidation process obviously. An analytical solution is then derived to calculate the degree of consolidation around permeable pile. Essentially, a mixed drainage boundary condition is established, where all drainage holes are fully pervious and the pile surface is completely impervious. The consolidation rate is found to be sensitive to the strip number and the width of drainage zone. For a fixed total area of drainage channels, a larger strip number and a smaller width of drainage zone should be implemented to increase the bearing capacity of permeable pile more effectively.

RÉSUMÉ : Pendant le battage des pieux, des pressions interstitielles excessives peuvent être générées dans le sol environnant. Un pieu peut gagner en capacité avec le temps (c'est-à-dire l'effet de mise en place) en raison de la consolidation du sol. Cependant, le temps de consolidation est souvent très long, ce qui empêche l'utilisation de la pleine capacité portante des pieux au début de la construction, ce qui conduit à une conception conservatrice. Un nouveau concept de pieu perméable est proposé pour fournir des canaux de drainage en ouvrant des trous de drainage autour de la circonférence du pieu, à travers lesquels les pressions interstitielles peuvent être dissipées. Un programme d'essais à l'échelle du modèle est mené dans des conditions de laboratoire contrôlées, dans lequel les réponses d'un pieu normal et de deux pieux perméables sont mesurées pour démontrer l'efficacité de l'introduction de canaux de drainage le long de la longueur du pieu. On trouve que l'utilisation de pieux perméables est évidemment capable d'accélérer le processus de consolidation. Une solution analytique est ensuite dérivée pour calculer le degré de consolidation autour du pieu perméable. Essentiellement, une condition aux limites de drainage mixte est établie, où tous les trous de drainage sont entièrement perméables et la surface du pieu est complètement imperméable. Le taux de consolidation est sensible au nombre de bandes et à la largeur de la zone de drainage. Pour une superficie totale fixe de canaux de drainage, un plus grand nombre de bandes et une plus petite largeur de zone de drainage devraient être mis en œuvre pour augmenter la capacité portante du pieu perméable plus efficacement.

KEYWORDS: permeable pile; consolidation; drainage hole; model test; analytical solution.

1 INTRODUCTION.

Displacement pile is often used to improve the bearing capacity of soft ground composed of clayey soils. Different installation methods are available for driven piles, including impact driving, vibro-driving, pressing, and combination of pressing and torque. Regardless of the installation approach, significant excess pore water pressures can be generated in the surrounding soil around a driven pile, leading to difficulty or problem in sequential construction of pile group (e.g., radial deformation, ground heave, settlement, and bending, fracturing, and floating of preinstalled pile). Dissipation of pore pressures usually takes a very long time (up to several months) (Bozozuk et al. 1978; Roy et al. 1981), since the hydraulic conductivity of clay is extremely low, which can influence the bearing capacity of driven pile (i.e., the set-up effect) before the full consolidation state in the surrounding soil is reached (Jardine et al. 2006; Lim and Lehane 2014). Driven pile can, therefore, gain capacity with time.

Different measures have been proposed to accelerate the consolidation process by enabling more drainage channels in the ground. This can help to utilize the full pile resistance at the early stage of construction. The most common practice is to include vertical drains between driven piles. Most researchers worked on the assessment of consolidation behavior for soft ground treated with prefabricated drains alone, in which the smear effect and the well resistance effect were taken into account in different manner (Chu et al. 2004; Walker and Indraratna 2006; Pothiraksanon et al. 2010; Chen et al. 2020b). However, the maximum excess pore water pressure occurs at the pile circumference, and it reduces with distance from the pile (O'Neill 2001). The use of vertical drains can never introduce drainage channels at the position where pore pressure peaks. Hence, engineers came out various techniques to introduce drainage channels at the buried structure itself by either setting grooves to install vertical drains around the circumference (Wang et al. 2020) or changing the permeability of the structure (Weber et al. 2010; Suleiman et al. 2014; Ni et al. 2016). However, vertical drain has limitation of clogging as time goes by, which can hinder the technique of setting grooves (Wang et al. 2020). Stone column (Weber et al. 2010) or pervious concrete pile (Suleiman et al. 2014; Ni et al. 2016) often has limited axial capacity, which can be a good solution for embankment loading, but not for high surcharge loading under high-rise buildings. Alternatively, a solution of so-called permeable pipe pile was proposed as schematically illustrated in Figure 1, where strips of drainage holes are drilled around the pile circumference. A close-ended pipe pile can then form a cavity, and pore water can be drained through drainage holes to enter the pile cavity, accelerating the consolidation process. Previous studies were mainly focused on the feasibility of the technique through physical and numerical modeling (Ni et al. 2017a, b, 2018), but research with emphasis on hydraulic and strength design of permeable pile is rather limited.

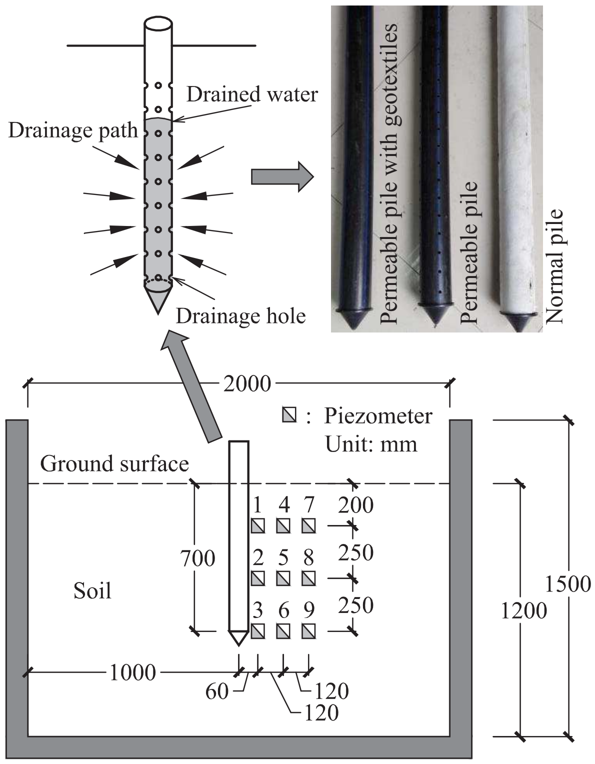

Figure 1. Illustration of test setup and permeable pile.

In this investigation, 1-g model-scale laboratory tests on the consolidation behavior of normal and permeable pipe piles are reviewed, and the beneficial effect of drainage openings on the dissipation of excess pore water pressure is illustrated. An analytical solution is proposed to characterize the consolidation process in the surrounding soil around permeable pile, where the soil-pile interface is simulated as a mixed drainage boundary with partial pervious interface and partial impervious interface. The experimental measurements of pore pressures are then used to assess the effectiveness of the proposed analytical method. Finally, a short parametric study is conducted to provide design implications for permeable pile.

2 PHYSICAL MODELING

2.1 *Experimental setup*

Model-scale laboratory tests are carried out on normal and permeable pipe piles under the 1g condition. The soil container has dimensions of 2 m × 2 m × 1.5 m (width × length × height) as presented in Figure 1. Steel stiffeners are welded around the circumference of the soil container to increase the rigidity of sidewalls. In this way, the lateral deflection of sidewall can be limited within 0.1% of the span, when the soil is subjected to a high level of surcharge loading (Brachman et al. 2000). Plastic pipes are manufactured to have a length of 900 mm and a diameter (d) of 60 mm to reproduce the behavior of pipe pile. The distance from the sidewall to the pile exceeds 16 times the pile diameter (16d). In addition, friction treatment is also implemented at sidewalls using lubricated polyethylene sheets. This can effectively minimize the boundary effect.

The use of plastic pipe is to consider the ease of fabrication of drainage holes. At four quadrants around the pile circumference, drainage holes are opened to have a diameter of 6 mm (0.1d). Along the pile, the spacing between drainage holes is controlled at 36 mm (0.6d). A close-ended pile tip is butt fused, which can help to form the pile cavity, as well as to avoid the occurrence of soil plugging. A pipe with a slightly smaller diameter is inserted into the permeable pipe pile. Removal of inside pipe helps to control the opening time of drainage holes. Around the outside surface of permeable pile, geotextiles are wrapped to form a filter to avoid the occurrence of clogging of drainage holes. The pressing-in method is adopted to install the pile, which could underestimate the degree of densification compared to pile driving, as well as the magnitude of excess pore water pressure.

Poorly graded clay with a plastic limit of 35%, a liquid limit of 73%, and a natural water content of 96.9% is used to fill the soil container. Plastic drains are attached on mandrels, which can then be pushed into the soil. Water is added above the ground surface to reduce the amount of trapped air bubbles. Geotextiles and double polyethylene sheets are employed to form the cover. Vacuum pressure is applied through drainage pipes for 2 weeks to facilitate the consolidation of soil. The ground surface is stabilized at a height of 120 mm in the end of consolidation. The average undrained shear strength is measured to vary from 9.6 kPa to 17.1 kPa, being consistent with the observed values of Wang et al. (2014).

A full saturation state is important, such that the phenomenon of time delay in the response of piezometers can be minimized (Strout and Tjelta 2005). In this work, the diameter of piezometer is 25 mm, and the tip length is 170 mm. The piezometer has a capacity of 100 kPa, and an accuracy of 0.05 kPa. Mandrels are also used to install piezometers at the specific spatial locations. Installation of these piezometers is done after the full consolidation is reached, which can minimize interference with soil movement during consolidation (Beddoe and Take 2016).

A total of three tests are conducted, including a normal pile test (test 1), and two permeable pipe pile tests. Permeable pile tests differ only in the opening time of drainage holes, where drainage holes are opened after 1.5 h of pile driving in test 2, and immediate opening after pile driving is allowed in test 3. More details about the model-scale tests can be found in Ni et al. (2018).

2.2 *Experimental results*

Variations of excess pore water pressure with time measured from piezometers for the three tests are plotted in Figure 2. In different test, one can see the phenomenon of time delay in the response of pore pressure generation. The increase of pore pressure after pile driving becomes more apparent for permeable pile than that for normal pile. Essentially, pore pressure peaks after approximately 1.5 h of pile driving. It should be noted that the phenomenon of time delay is not caused by the degree of saturation for piezometer, but is induced by the low value of hydraulic conductivity of clay, delaying the generation of pore pressure.

One can see that permeable pile can effectively reduce the generated pore pressure, or accelerate the soil consolidation process. The measured pore pressure in test 2 or 3 is always lower than that in test 1. In test 3, the drainage condition is allowed immediately after pile driving, and as such the smallest pore pressure is obtained. After 10.5 h of pile driving, the reduction of pore pressure in tests 2 and 3 is about 14.6% and 21.4% lower than the value in test 1, respectively. This demonstrates the beneficial effect of drainage holes, especially at the early stage of consolidation. It is suggested to allow the opening of drainage holes immediately after pile driving. The full pile resistance can hence be utilized as early as possible, and the adverse effect of pile driving can be minimized.

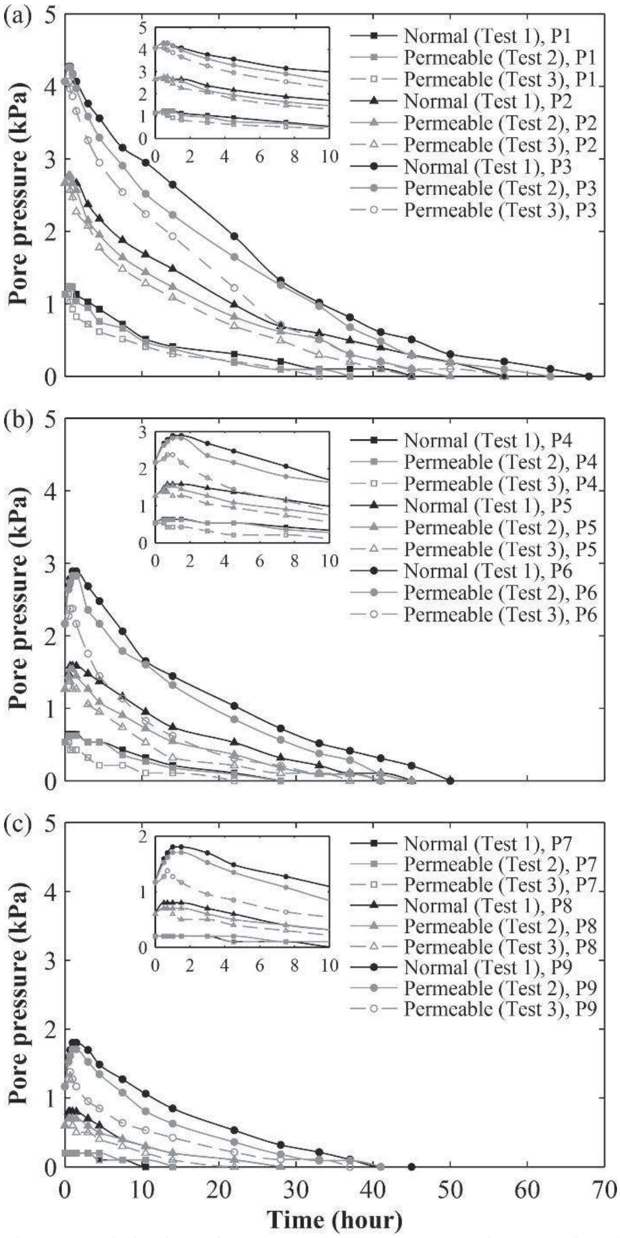

Figure 2. Dissipation of excess pore water pressure for normal and permeable piles at a distance of (a) $r = 1d$, (b) $r = 3d$ and (c) $r = 5d$.

3 ANALYTICAL SOLUTION

3.1 *Model derivation*

Figure 3 shows the schematics of the consolidation model for soils around a permeable pile. The radius and the length of the pile are denoted as r_0 and L, respectively. Upon pile driving, excess pore water pressure peaks at the pile circumference, and it reduces with distance from the pile. The distribution pattern of pore pressure is characterized as $u_0(r, z)$, where a plastic zone with a radius of r_p is formed, and it diminishes at a distance of r_e. The distance from the pile is defined as r.

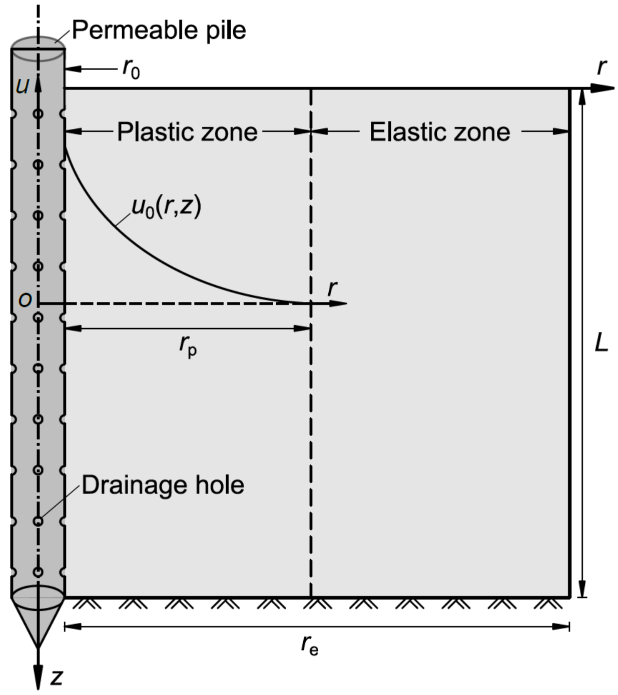

Figure 3. Sketch of the consolidation model for soils around a permeable pile.

A normalization scheme is implemented to derive the consolidation model using dimensionless variables:

$$\kappa = k_h / k_v,\ \eta = L/r_e,\ n_e = r_e/r_0,\ \rho = r/r_0,\ Z = z/L \tag{1a}$$

$$u_D = u/u_0(r_0, L),\ u_{D0}(\rho, Z) = u_0(r_0\rho, LZ)/u_0(r_0, L) \tag{1b}$$

$$T_h = k_h t / (\gamma_w m_v r_e^2) \tag{1c}$$

where m_v is the volume compression coefficient of soil; γ_w represents the unit weight of water; k_h and k_v are the hydraulic conductivity in the horizontal and the vertical directions, respectively; z denotes the depth from the ground surface; and t is the time.

The governing equation that can describe the dissipation of pore pressure in the surrounding soil due to pile installation is given below:

$$\frac{\partial u_D}{\partial T_h} = n_e^2 \left(\frac{\partial^2 u_D}{\partial \rho^2} + \frac{1}{\rho} \frac{\partial u_D}{\partial \rho} \right) + \frac{1}{\kappa \eta^2} \frac{\partial^2 u_D}{\partial Z^2} \tag{2}$$

The initial condition that can characterize the distribution of pore pressure in the soil induced by pile driving is defined as:

$$u_{\mathrm{D}}\big|_{T_{\mathrm{h}}=0}=u_{\mathrm{D0}}\left(\rho,Z\right) \tag{3}$$

Drainage holes can be considered as completely pervious, and the pile circumference is regarded as perfectly impervious. Following the work of Gray (1945), an impeded boundary is employed to capture the drainage behavior at the permeable pile-soil interface.

$$\left(\frac{\partial u_{\mathrm{D}}}{\partial \rho}-\mu u_{\mathrm{D}}\right)\Bigg|_{\rho=1}=u_{\mathrm{D}}\big|_{\rho=n_{\mathrm{e}}}=0 \tag{4}$$

where μ is the permeable pile-soil interface parameter. When the μ value approaches infinity, the impeded boundary corresponds to a fully pervious boundary; whilst when the μ value approaches zero, it reduces to a perfectly impervious boundary.

The ground surface and the bottom boundary are pervious and impervious, respectively, as follows:

$$u_{\mathrm{D}}\big|_{Z=0}=\frac{\partial u_{\mathrm{D}}}{\partial Z}\bigg|_{Z=1}=0 \tag{5}$$

The governing equation can be transformed into a linear ordinary differential equation after implementing finite sine and Weber transforms. The solution to the problem is then written as:

$$\tilde{u}_{\mathrm{D}}=\phi_{mn}\mathrm{e}^{-\left(n_{\mathrm{e}}^{2}\alpha_{m}^{2}+\frac{\beta_{n}^{2}}{\kappa\eta^{2}}\right)T_{\mathrm{h}}} \tag{6}$$

The general solution that can describe the consolidation behavior of soil around a permeable pipe pile is given as:

$$u_{\mathrm{D}}=2\sum_{m=1}^{\infty}\sum_{n=1}^{\infty}\frac{\phi_{mn}R_{m}\left(\rho\right)}{M_{m}}\sin\left(\beta_{n}Z\right)\mathrm{e}^{-\left(n_{\mathrm{e}}^{2}\alpha_{m}^{2}+\frac{\beta_{n}^{2}}{\kappa\eta^{2}}\right)T_{\mathrm{h}}} \tag{7}$$

The average degree of consolidation as a function of excess pore water pressure is derived as:

$$U=1-\frac{\int_{1}^{n_{\mathrm{e}}}\int_{0}^{1}\rho u_{\mathrm{D}}\left(\rho,Z,T_{\mathrm{h}}\right)\mathrm{d}Z\mathrm{d}\rho}{\int_{1}^{n_{\mathrm{e}}}\int_{0}^{1}\rho u_{\mathrm{D0}}\left(\rho,Z,T_{\mathrm{h}}\right)\mathrm{d}Z\mathrm{d}\rho} \tag{8}$$

Following the study of Sharifzadeh et al. (2013), a back-analysis procedure is used to determine the permeable pile-soil interface parameter. Essentially, different combinations of input parameters are analyzed using the proposed analytical solution, which can be compared against the numerical results. A linear function for interface parameter is then developed as:

$$\mu\big|_{r_{\mathrm{h}}/r_{0},\,\kappa=\text{constant}}=A_{0}\lambda \tag{9}$$

$$\mu=\frac{0.43+2.03\mathrm{e}^{-\frac{\kappa}{0.12}}+0.55\mathrm{e}^{-\frac{\kappa}{0.76}}+0.17\mathrm{e}^{-\frac{\kappa}{7.01}}}{\left(r_{\mathrm{h}}/r_{0}\right)^{1.28}-1.31\times10^{-3}}\cdot\lambda \tag{10}$$

where the opening ratio λ (area of opening/total surficial area) falls within 0-3%; the drainage hole-to-pipe pile radius ratio r_h/r_0 (r_h is the radius of drainage hole) varies from 2% to 4.5%; and the hydraulic conductivity ratio κ changes from 0.1 to 100. The number of drainage holes in a single layer is defined as n_{sl} (i.e., strip number), and N_h represents the total number of drainage holes. The parameters n_{sl} and N_h are hence directly related with the opening ratio λ.

More details about the derivation process and the definition of all parameters can be seen in Chen et al. (2020c).

3.2 *Comparison against experimental measurements*

To evaluate the effectiveness of the proposed analytical model, the measured pore pressures from model-scale laboratory tests are employed for comparison. An exponential function is often used to interpret the variations of pore pressure with distance from the pile circumference in the lateral direction, whereas the pore pressure can generally increase with depth following a linear pattern (Ni et al. 2018). The mathematical form of pore pressure distribution (Chen et al. 2020c) is written as follows:

$$u\big|_{t=0}=\begin{cases}a\left(z-h_{0}\right)\ln\dfrac{b}{r}, & 0\le r\le b,\ z\ge h_{0}\\ 0, & \text{others}\end{cases} \tag{11}$$

where a, b, and h_0 are three unknown parameters.

Based on a curve fitting excise against the experimental measurements from piezometers P4-P9, the unknown parameters are determined as $a = 4.29$ kPa/m, $b = 0.54$ m, and $h_0 = 0.1$ m. It should be emphasized that the measurements from piezometers P1-P3 are not adopted to conduct curve fitting, since these sensors are installed in a close proximity to the pile, at which position significant disturbance to the soil can be induced due to pile driving.

As reported by Ni et al. (2018), the consolidation coefficient of soil in different direction can be estimated as $C_h = k_h/(\gamma_w m_v) = C_v = k_v/(\gamma_w m_v) = 10^{-6}$ m^2/s. The interface parameter is back-calculated as $\mu = 0.37$. In test 2, drainage holes are opened after 1.5 h of pile driving, during which dissipation of pore pressure has already occurred. In test 3, all drainage holes are opened immediately after pile driving, which is more representative.

In Figure 4, the measured and the calculated pore pressures for piezometers P4-P9 in test 3 are compared as a function of time. The phenomenon of time delay in the response of piezometers cannot captured by the analytical approach. The general pattern of pore pressure dissipation obtained from model-scale laboratory tests can be captured well by the analytical solution. It suggests that the developed mathematical model with the proposed impeded boundary for characterizing the permeable pile-soil interface is reasonable for use to evaluate the consolidation behavior of soil around permeable pipe pile.

3.3 *Parametric analysis*

A parametric study is performed to assess how the opening pattern can influence the consolidation behavior of soil around permeable pile. The following parameters are defined: θ denotes the circular angle in a polar coordinate system; θ_s represents the opening angle (width of drainage zone); $\rho_p = r_p/r_0$, and $\rho_e = r_e/r_0$. From the consolidation model, one can see that the consolidation response in the surrounding soil around permeable pile is simply governed by the time factor T_h, the strip number n_{sl}, and the width of drainage zone θ_s.

The impact of strip number of drainage holes on the consolidation curve for piles with different n_{sl} values (the case of $n_{sl} = 0$ represents the behavior of normal pile, and the cases of n_{sl} = 1, 2, 3, and 4 characterize the response of permeable pile) is investigated as shown in Figure 5. It is clear that increasing the n_{sl} value can help to accelerate the consolidation process significantly. At a specific time factor, the average degree of consolidation for permeable pile with a high n_{sl} value is higher than that derived for normal pile or permeable pile with a low n_{sl} value. The beneficial effect of drainage holes is more apparent at the early consolidation stage, where the consolidation curves from different conditions differ greater. The difference in dissipation rate of pore pressure between normal and permeable piles reduces with time, and eventually approaches zero in the end. As a rough estimation, at an average degree of consolidation of $U = 20\%$, the use of drainage holes at four quadrants can reduce the consolidation time by 81.3%, and the reduction of consolidation time is 74.0%, 59.0%, and 33.5% for the cases of $n_{sl} = 3$, 2 and 1, respectively, compared to normal pile. At a later stage of $U = 80\%$, the reduction of consolidation time is calculated as 42.4%, 38.5%, 32.0%, and 18.8% for permeable pile with $n_{sl} = 4$, 3, 2 and 1, respectively, compared to normal pile. Therefore, to optimize the design of permeable pile, the strip number of drainage holes should be increased. However, the strength requirement of permeable pile must be checked through through-hole analysis or laboratory tests (Ni et al. 2017a).

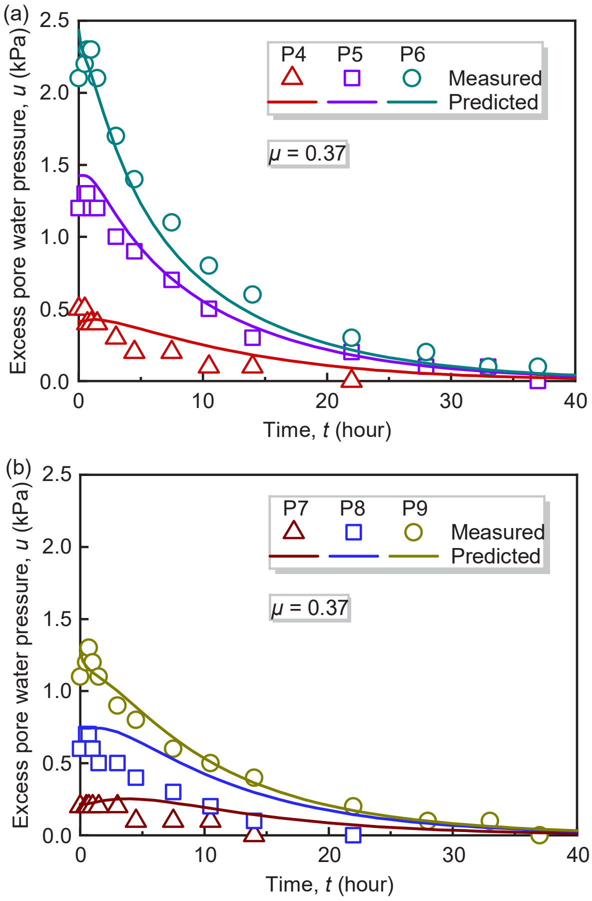

Figure 4. Comparison between measured and predicted excess pore water pressures at a distance of (a) $r = 3d$ and (b) $r = 5d$.

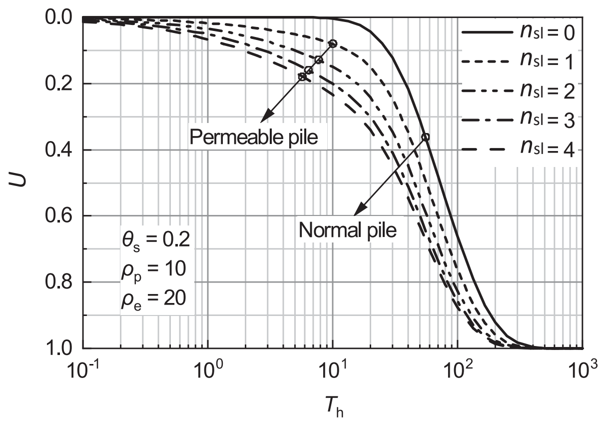

Figure 5. Average degree of consolidation for different strip number of drainage zone.

Figure 6 illustrates the calculated consolidation curves when the width of drainage zone varies from $\theta_s = 0$, 0.05, 0.1, 0.15, to 0.2. One can interpret that a permeable pile always requires less time than a normal pile to achieve a certain average degree of consolidation. The consolidation curve for permeable pile is therefore on the left side, indicating that allowing drainage holes helps to accelerate the dissipation of excess pore water pressure. The use of drainage holes facilitates the consolidation process, improving the bearing capacity of permeable pile. It is interesting that the width of drainage zone does not shift the consolidations curve too much. At an average degree of consolidation of $U = 20\%$, the case with $\theta_s = 0.2$ can help to reduce the consolidation time by 59.0% compared to normal pile, whereas the reduction of consolidation time is derived as 54.8%, 50.0%, and 43.6% for the cases of $\theta_s = 0.15$, 0.1, and 0.05, respectively. At a later stage of $U = 80\%$, the reduction of consolidation time becomes less obvious, where the cases of $\theta_s = 0.2$, 0.15, 0.1, and 0.05 reduces by 31.9%, 30.0%, 27.5%, and 24.4%, respectively.

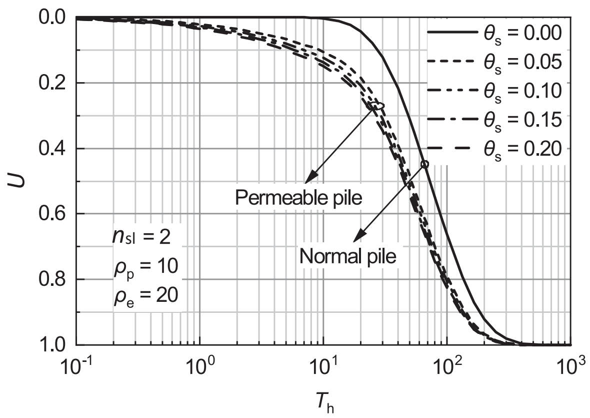

Figure 6. Average degree of consolidation for different width of drainage zone.

When the total area of drainage channels is fixed, the consolidation efficiency can be increased by permeable pile with a higher strip number of drainage holes and a smaller width of drainage zone. In other words, more drainage holes should be drilled in a single layer with a smaller diameter to provide more distributed drainage channels around the pile circumference. In this way, the drainage path can be decreased as much as possible. Similar design implications have been suggested by Chen et al. (2020a) for surcharge preloading consolidation of reclaimed land with a distributed drainage boundary.

4 CONCLUSIONS

Driven pile is increasingly used to improve the bearing capacity of soft ground, above which high surcharge load can be applied. Different approaches can be employed to install displacement piles, such as impact driving, vibro-driving, pressing, and combination of pressing and torque, but all these installation methods can inevitably cause the generation of excess pore water pressure in the surrounding soil. Dissipation of pore pressure can take a long time, since the hydraulic conductivity of clay is often very low. Hence, the full pile resistance is not mobilized at the early stage of construction, but increases with time (i.e., the set-up effect). In this study, a conceptual design of permeable pipe pile is proposed, in which distributed drainage holes are opened around the pile circumference. The pile cavity can then be used to drain pore water from the ground. Model-scale laboratory tests are conducted on a normal pile and two permeable piles to illustrate the effect of drainage holes on the dissipation of pore pressure. In addition, an analytical solution is derived to characterize the degree of consolidation around permeable pile. The main findings of this study include:

- After pile driving, the phenomenon of time delay in the response of piezometer can be seen in all three cases, regardless of the pile type and the opening time of drainage holes. This is due to the low permeability of clay materials.
- Allowing drainage opening around the pile circumference can provide good drainage channels, which increases the dissipation rate of excess pore water pressure significantly. All drainage holes should be opened immediately once the activity of pile driving is completed.
- The proposed impeded boundary can capture the drainage characters at the permeable pile-soil interface correctly, where all drainage holes are fully pervious and the pile surface is completely impervious. An expression for interface parameter is given to simplify the analysis.
- The consolidation behavior of soil around permeable pile is highly sensitive to the strip number and the width of drainage zone. When the total opening ratio keeps constant, a higher strip number and a smaller width can provide more beneficial consolidation results for permeable pile. In other words, more distributed drainage holes with a small diameter should be implemented in a single layer to create the drainage channels as much as possible.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

Beddoe, R.A., and Take, W.A. 2016. Loss of slope support due to base liquefaction: comparison of 1g and centrifuge landslide flume experiments. *Soils and Foundations* 56(2), 251-264.

Bozozuk, M., Fellenius, B.H., and Samson, L. 1978. Soil disturbance from pile driving in sensitive clay. *Canadian Geotechnical Journal* 15(3), 346-361.

Brachman, R.W.I., Moore, I.D., and Rowe, R.K. 2000. The design of a laboratory facility for evaluating the structural response of small-diameter buried pipes. *Canadian Geotechnical Journal* 37(2), 281-295.

Chen, Z., Ni, P., Chen, Y., and Mei, G. 2020a. Plane-strain consolidation theory with distributed drainage boundary. *Acta Geotechnica* 15(2), 489-508.

Chen, Z., Ni, P., Mei, G., and Chen, Y. 2020b. A semi-analytical solution for consolidation of ground with partially penetrating PVDs under the free-strain condition. *Journal of Engineering Mechanics* (accepted).

Chen, Z., Xiao, T., Feng, J., Ni, P., Chen, D., Mei, G., and Chen, Y. 2020c. Mathematical characterization of pile-soil interface boundary for consolidation analysis of soil around permeable pipe pile. *Canadian Geotechnical Journal* (accepted).

Chu, J., Bo, M., and Choa, V. 2004. Practical considerations for using vertical drains in soil improvement projects. *Geotextiles and Geomembranes* 22(1), 101-117.

Gray, H. 1945. Simultaneous consolidation of contiguous layers of unlike compressible soils. *Trans., ASCE* 110, 1327-1344.

Jardine, R., Standing, J., and Chow, F. 2006. Some observations of the effects of time on the capacity of piles driven in sand. *Géotechnique* 56(4), 227-244.

Lim, J., and Lehane, B. 2014. Characterisation of the effects of time on the shaft friction of displacement piles in sand. *Géotechnique* 64(6), 476-485.

Ni, L., Suleiman, M.T., and Raich, A. 2016. Behavior and Soil–Structure Interaction of Pervious Concrete Ground-Improvement Piles under Lateral Loading. *Journal of Geotechnical and Geoenvironmental Engineering* 142(2), 04015071.

Ni, P., Mangalathu, S., Mei, G., and Zhao, Y. 2017a. Compressive and flexural behaviour of reinforced concrete permeable piles. *Engineering Structures* 147, 316-327.

Ni, P., Mangalathu, S., Mei, G., and Zhao, Y. 2017b. Permeable piles: An alternative to improve the performance of driven piles. *Computers and Geotechnics* 84, 78-87.

Ni, P., Mangalathu, S., Mei, G., and Zhao, Y. 2018. Laboratory investigation of pore pressure dissipation in clay around permeable piles. *Canadian Geotechnical Journal* 55(9), 1257-1267.

O'Neill, M.W. 2001. Side resistance in piles and drilled shafts. *Journal of Geotechnical and Geoenvironmental Engineering* 127(1), 3-16.

Pothiraksanon, C., Bergado, D.T., and Abuel-Naga, H.M. 2010. Full-scale embankment consolidation test using prefabricated vertical thermal drains. *Soils and Foundations* 50(5), 599-608.

Roy, M., Blanchet, R., Tavenas, F., and Rochelle, P.L. 1981. Behaviour of a sensitive clay during pile driving. *Canadian Geotechnical Journal* 18(1), 67-85.

Sharifzadeh, M., Tarifard, A., and Moridi, M.A. 2013. Time-dependent behavior of tunnel lining in weak rock mass based on displacement back analysis method. *Tunnelling and Underground Space Technology* 38, 348-356.

Strout, J.M., and Tjelta, T.I. 2005. In situ pore pressures: What is their significance and how can they be reliably measured? *Marine and Petroleum Geology* 22(1), 275-285.

Suleiman, M.T., Ni, L., and Raich, A. 2014. Development of pervious concrete pile ground-improvement alternative and behavior under vertical loading. *Journal of Geotechnical and Geoenvironmental Engineering* 140(7), 04014035.

Walker, R., and Indraratna, B. 2006. Vertical drain consolidation with parabolic distribution of permeability in smear zone. *Journal of Geotechnical and Geoenvironmental Engineering* 132(7), 937-941.

Wang, J., Cai, Y., Fu, H., Lin, T., Ye, Q., Jin, Y., and Chen, X. 2014. Indoor and field experiment on vacuum preloading with new anti-clogging measures. *Chinese Journal of Rock Mechanics and Engineering* 33(6), 1257-1268.

Wang, S., Ni, P., Chen, Z., and Mei, G. 2020. Consolidation solution of soil around a permeable pipe pile. *Marine Georesources & Geotechnology* 38(9): 1097-1105.

Weber, T., Plötze, M., Laue, J., Peschke, G., and Springman, S. 2010. Smear zone identification and soil properties around stone columns constructed in-flight in centrifuge model tests. *Géotechnique* 60(3), 197-206.