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Comparison of penetration resistance and vertical capacity of short piles installed by Standard Press-in in loose sand

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ABSTRACT: In the Press-in Method, a vertical jacking force required for installing a pile is automatically measured for every single pile. This information is expected to be utilized to confirm the vertical performance of the pile. In this paper, results of a series of large-scale model tests are reported to compare the resistance during installation and during static load test. The test pile was a closed or open-ended tubular pile with an outer diameter of 318.5 mm, embedded in a loose sand to around 6 m. Phenomena of pile set-up or set-down were observed in some cases. Through literature review and detailed analyses of the experimental data, the cause of the set-down was concluded to be mainly due to the pile installation being terminated at a depth where the soil strength decreased with depth and to a lesser extent because negative pore water pressure was generated during installation.

1 INTRODUCTION

In the Press-in Method without any installation assistance such as water jetting or augering (Standard Press-in), a pile is installed by a static jacking force, and the piling data including the information on the jacking force can be monitored and recorded for every single pile. It has been suggested that the jacking force during installation (especially at the end of installation (EOI)) can be linked to the capacity of the pile. It is generally understood that the penetration resistance in a load test is usually larger than that during installation (Komurka *et al*., 2003; Gavin *et al*., 2015), which is known as “pile set-up”. For pressed-in piles in cohesive soils, a two-fold or three-fold increase in total resistance with time after EOI has been confirmed in the field tests (Ishihara & Haigh, 2018; Ishihara *et al*., 2020). In such cases, the penetration resistance at EOI of a pile can be taken as the lower-bound for the capacity of that pile. To put this idea into practice, it is important to know under what circumstances the opposite trend (pile set-down) is encountered.

This paper reports the results of a series of large-scale model tests using a closed or open-ended tubular pile with the outer diameter of 318.5 mm, which were pressed-in with or without surging (applying downward and upward displacement l_d and l_u repeatedly as illustrated in Figure 1) in a loose sand to around 6 m. Phenomena of pile set-up or set-down were observed in some cases, and the causes for the set-down are discussed through literature review and detailed analyses on the experimental data.

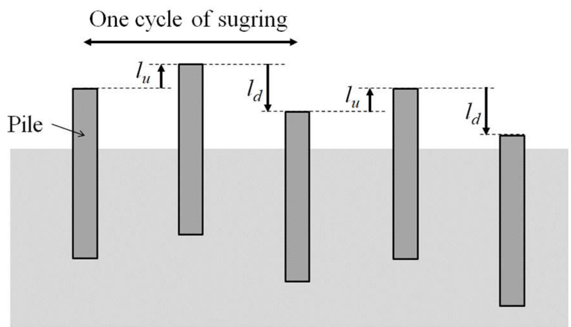


Figure 1. Process of surging

2 POSSIBLE MECHANISMS OF PILE SET-DOWN

The difference in the penetration resistance at EOI and in a load test (i.e. either pile set-up or pile set-down) is partly caused by the differences in loading conditions during installation and in the load test. For example, the loading rate (or penetration rate) during installation (typically 20 mm/s or greater) is much higher than that in the load test (0.06 mm/s or lower for piles with the outer diameter of 1000 mm). The loading direction is often two-way during installation (i.e. installation is associated with surging) while it is consistently one-way in a load test. The condition of the soil around the pile is relatively “unstable” (highly variable with time) during installation, with excess pore water pressure being generated by the pile installation, while it is relatively "stable" at the start of the load test because there is usually a 5-days (for sand) or 14-days (for clay) interval after EOI to assure a complete dissipation of excess pore water pressure (JGS, 2002) prior to load testing. In this paper, possible mechanisms for pile set-down will be summarized in terms of (1) penetration rate, (2) surging and (3) others.

2.1 *Effect of penetration rate*

The penetration rate could be the cause of pile set-down in the following ways.

[R1] If the soil is dilatant, negative or reduced excess pore water pressure will be generated during installation, which will increase the penetration resistance (Silva & Bolton, 2005; Lauder *et al*. 2012), as illustrated in Figure 2 (after White *et al*., 2010) where v is the penetration depth, D is the pile diameter and c_v is the coefficient of consolidation. The higher penetration rate during installation than in the load tests will lead to greater negative excess pore water pressure during installation, causing the pile set-down after a period of pore pressure equalization.

[R2] The higher penetration rate might increase the penetration resistance due to the effect of viscosity. Considering the findings of Robinson & Brown (2013) in clays as shown in Figure 3 where the definition of the horizontal axis is identical with that in Figure 2, with the higher penetration rate and the greater strain level during installation than in the load test, the resistance during installation might become larger than in the load test. On the other hand, this trend will be absent in loose contractile sands (Chow *et al*., 2020).

[R3] According to Tatsuoka *et al*. (2008) and Enomoto *et al*. (2009), unbound granular materials show four different responses due to the viscosity effect when sheared, as shown in Figure 4, depending on their particle shape, particle grading, particle size, particle crushability and so on. Until the local peak stress is reached, the effective stress increases when the strain rate is increased via four typical responses. After the local peak stress is experienced, soils behave differently. It might follow that, for the base resistance which may be associated with peak stresses, larger resistance would always be experienced at higher installation rates, while the shaft resistance may be increased or decreased according to the types of responses of the soil as it is associated with residual stresses.

[R4] Watanabe & Kusakabe (2013) conducted triaxial compression tests on dense Toyoura sand and confirmed that the internal friction angle and deformation modulus become greater when the strain rate is increased. This will lead to a larger resistance (both for base and shaft) during installation.

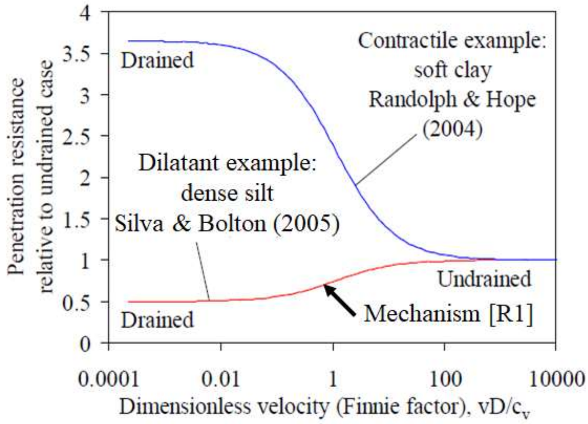


Figure 2. Rate effect on penetration resistance (after White *et al*., 2010)

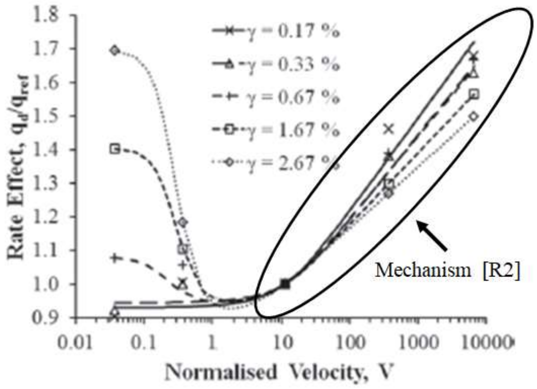


Figure 3. Effect of strain level on the rate effect (after Robinson & Brown, 2013)

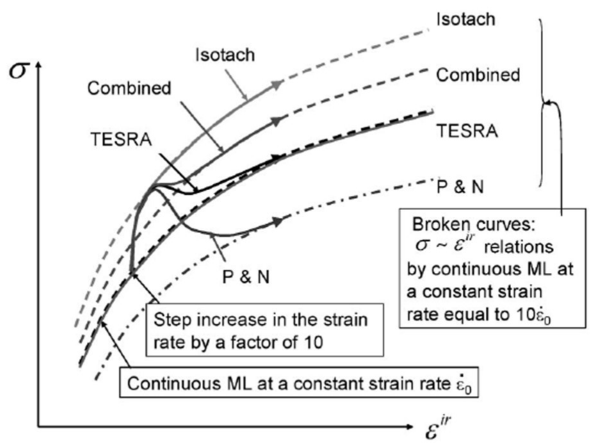


Figure 4. Four types of responses in terms of viscosity effect (Tatsuoka *et al*., 2008)

2.2 *Effect of surging*

The motion of surging (repeated penetration and extraction during installation) could be the cause of pile set-down in the following ways.

[S1] It is hypothesized that lifting the pile may cause a vacuum to form (or very negative pressures) that tries to draw water into any void formed. This will increase the effective stress on the next downward stroke and increase the penetration resistance.

[S2] Jeffery *et al*. (2016) conducted CPT around jacked piles and confirmed that the cone resistance near these piles becomes greater than that of the virgin ground if it consists of loose sand, and this increase is attributed to the increase in the relative density. White & Bolton (2002) confirmed that the surging motion promotes the densification of sands at the pile-soil interface. This densification could increase the dilation and thus the potential of the rate effect associated with the pore pressure effect (i.e. [R1]) during installation, while reducing the confinement stress from the far field (and the horizontal stress on the pile shaft) when loaded slowly, which is known as "friction fatigue".

2.3 *Other effects*

Other than the penetration rate and surging, the following ([O1], [O2], [O3]) would be possible influences on the mechanisms of pile set-down. In addition, mechanisms [O4], [O5] and [O6], which lead to the absence in set-up of shaft resistance, can be the indirect causes of set-down in total resistance when the set-down trend in base resistance due to other mechanisms are significant.

[O1] The variation of the soil strength (cone resistance or SPT) with depth. If the installation is stopped where the soil strength decreases with depth, the resistance in the load test will be lower than the resistance at EOI, as the depth of pile base is greater during and after load testing.

[O2] Lower displacements during the load test (to define the pile capacity) than during installation. This factor could be more influential if the stiffness of the soil around the pile is lower. Also, considering that a greater displacement is required to mobilize full resistance on pile base than on pile shaft, this factor will be more influential if the pile is shorter as suggested by Zhang *et al*. (2006), since the total capacity depends more on base capacity for shorter piles.

[O3] Particle crushing and volume contraction. Leung *et al*. (1996) confirmed that the pile displacement under sustained loading (i.e. creep) is caused by particle crushing and the subsequent volume contraction of the soil around the pile base. Mitchell (2004) argued that the "locked-in" stress around the pile base as a result of the pile installation will act as the sustained load and lead to crushing, re-orientation and slip of particles, which causes a reduction in confining stress and base capacity.

[O4] Lack of increase in dilatancy with time after EOI. Based on the results of the measurement of horizontal stresses on the pile shaft after EOI and during load test, Axelsson (2000) proposed that the set-up in shaft resistance is explained by an increase in dilatancy of soils on the pile shaft with time after EOI, which would be caused by the rearrangement and the subsequent intrusion of soil particles into the rough surface of the pile ("constrained dilatancy"). Bowman & Soga (2005) conducted creep tests and confirmed that the increase in the volumetric strain (i.e. dilation) was apparent in dense sand but was absent in loose sand.

[O5] Lack of increase in horizontal stresses on the pile shaft. It is believed that a creation of arching (increased hoop stress) in the soil around the pile shaft during installation and the dispersion of the arching with time leading to the increase in horizontal stresses on the pile shaft after EOI (Chow *et al*., 1998). However, this mechanism was not clearly observed in the experiments of Axelsson (2000).

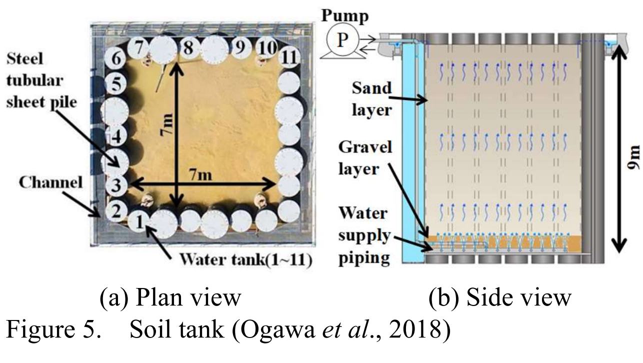
[O6] Lack of increase in plug strength (or inner shaft capacity). Randolph *et al*. (1991) showed that the plug strength of an open-ended pile in drained conditions is much higher than that in undrained conditions. This will lead to pile set-up if the pore water pressure inside the pile is sufficiently increased during installation. Alternatively, if the pore pressure increase is small, the set-up trend may be absent, and set-down trend could be observed especially when base or shaft resistance shows set-down behavior due to other mechanisms.

3 EXPERIMENTAL METHODS

3.1 *Apparatus*

Experiments were conducted in a large soil tank 7 m square and 9 m deep as shown in Figure 5. The soil tank is equipped with a water supply system at its base, to make the whole model ground liquefiable (boiling state) to allow preparation of a consistent soil bed. Silica sand NSK-40 (with its effective grain size D_{50} being 0.23 mm) was used as the material for the model ground (soil test bed). The relative density (D_r) of the model ground was roughly 50% on average (estimated from the weight of the sand put into the soil tank and the depth of the completed model ground), although the site density tests conducted at shallow depths (surface and 1 m below the surface) showed that D_r was as small as 4 %. Details of this apparatus and the model ground can be seen in more detail in Ogawa *et al*. (2018).

A closed-ended pile or an open-ended pile with an outside diameter (D_o) of 318.5 mm was used as a test pile. The closed-ended pile was equipped with a load



(a) Plan view (b) Side view

Figure 5. Soil tank (Ogawa *et al*., 2018)

cell and strain gauges at its base to measure base resistance and base torque separately, and with earth pressure transducers and pore water pressure transducers on its surface to measure the total horizontal stresses and pore water pressures at 0.25 m, 1.5 m and 3.0 m above the pile base ($\sigma_{hp\text{-}1}$, $\sigma_{hp\text{-}2}$, $\sigma_{hp\text{-}3}$, $u_{p\text{-}1}$, $u_{p\text{-}2}$ and $u_{p\text{-}3}$). The wall thickness of the open-ended pile was 10.3 mm.

Horizontal stresses in the model ground were measured by earth pressure transducers at the positions indicated in Figure 6. Horizontal stresses in radial direction (σ_{hr}) were measured at three levels at two positions, whereas those in circumferential direction ($\sigma_{h\theta}$) were measured at one position.

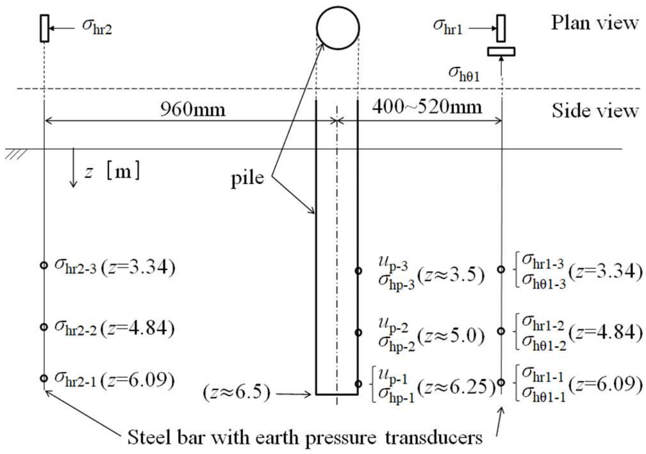


Figure 6. Positions of earth pressure measurement

3.2 *Procedures*

The tests were conducted based on the following procedures. (1) The model ground was prepared by injecting water from the bottom of the soil tank inducing a "boiling" state throughout the test bed, stopping the water injection and vacuum pumping the water from the bottom of the stand pipes (installed at four corners in the tank) to create the water level required, and waiting for several hours to attain static pore water pressure distribution in the model ground. (2) The test pile was installed at the center of the model ground by Standard Press-in by a press-in machine with or without surging as described in Table 1. In press-in without surging, the pile was unloaded at every 850mm penetration depth as the length of the cylinder of the piling machine was 850mm. To secure the reaction force, the press-in machine grasped the reaction beam which was fixed to the soil tank. (3) The test pile was left for 1 or 7 days. (4) The test pile was vertically load tested, based on JGS standard (JGS, 2002). (5) The test pile was extracted monotonically using the press-in machine.

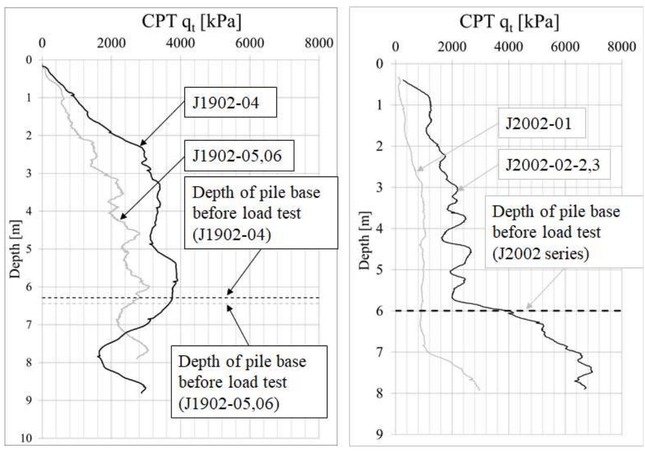
During installation, head load (Q) was obtained by removing the weight of the chuck from the load applied by the press-in machine measured by pressure sensors (Q'). The penetration depth (z) was measured using a wire-type stroke sensor, except for J1902-04 where a stroke sensor in the press-in machine was used. During static load tests, head load was measured by a load cell placed on the pile head. The pile displacement was measured by two displacement transducers placed on the pile head.

3.3 *Test cases*

Test cases are summarized in Table 1, where v_d and v_u are the downward and upward velocity of the pile and t_{LT} is the time after the end of installation to the start of load test. The ground surface level and the water level were measured from the reference level in the soil tank (near the ground surface). Initially, three tests (J1902 test series) were conducted using the closed-ended pile. As discussed later in Section 4, pile set-down was observed in these tests, which was thought to be mainly due to the decreasing trend of the soil strength around the depth of the pile base as shown in Figure 7a. To cope with this issue, the model ground was carefully mixed by installing an H-shaped pile with water jetting hose and nozzle with the flowrate of around 300 liters per minute at nine different positions, in addition to injecting water from the bottom of the soil tank. Then additional tests (J2002 test series) were conducted in an improved soil condition with a shorter embedment depth as shown in Figure 7b, using an open-ended pile. In J2002-03, the installation was conducted with the upper limit of jacking force (Q'_{UL}), in which the pile was

Table1 Test cases

Test No.	End condition	Ground Surface level [m]	Water level [m]	l_d [mm]	l_u [mm]	v_d [mm/s]	v_u [mm/s]	t_{LT}
J1902-04	Closed	-0.01	-7.5	400	200	45	48	7 days
J1902-05	Closed	-0.10	-1.0	400	200	45	48	7 days
J1902-06	Closed	-0.11	-1.1	850	0	45	-	7 days
J2002-01	Open	-0.20	-0.5	850	0	51	46	24 hrs
J2002-02-2	Open	-0.21	-7.9	850	0	51	46	24 hrs
J2002-03	Open	-0.25	-8.4	100	50	51	46	23.5 hrs



(a) J1902 test series (b) J2002 test series

Figure 7. Ground condition and depth of pile base

extracted by a certain amount (l_u) every time Q' reached Q'_{UL}.

It should be noted that CPT (Cone Penetration Test) in Figure 7 was not conducted in each test shown in Table 1, but conducted in a fresh ground with high water level once and in a fresh ground with low water level once, in each test series, at the center of the model ground.

4 EXPERIMENTAL RESULTS AND DISCUSSIONS

4.1 *Resistances recorded in J1902 test series*

Figure 8 shows the measured head load (Q), base resistance (Q_b) and shaft resistance (Q_s) obtained by $Q - Q_b$ in each test, where bold lines represent the values recorded in load tests. In J1902-05 and J1902-06 (with higher water levels), values of Q, Q_b and Q_s were lower than those in J1902-04 (with low water level). Comparing J1902-05 and J1902-06, both of Q_b and Q_s were reduced by surging during installation, and this reduction was more prominent in Q_s than in Q_b. Similar trends of reduction in Q_b and Q_s were found in the subsequent load test in these two tests.

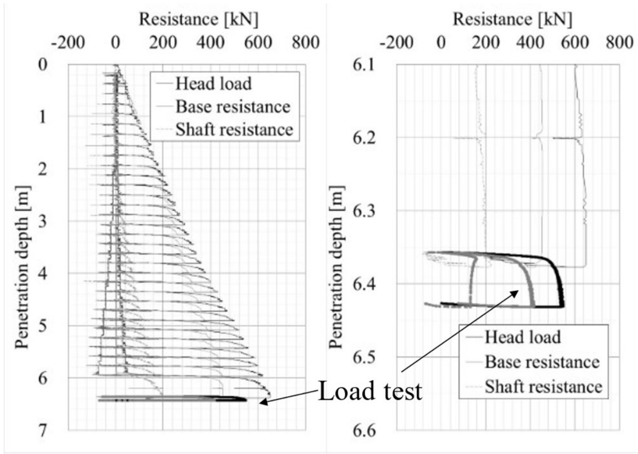
In all the three tests, Q_b during load test was lower than that at EOI. On the other hand, such a reduction in Q_s was only found in J1902-04 (with low water level), and in the other two tests there was little reduction (and little increase) in Q_s values. As a result of these trends in Q_b and Q_s, Q was lower in the load test than at EOI in all of the three tests.

4.2 *Narrowing down the mechanisms for the pile set-down observed in J1902 test series*

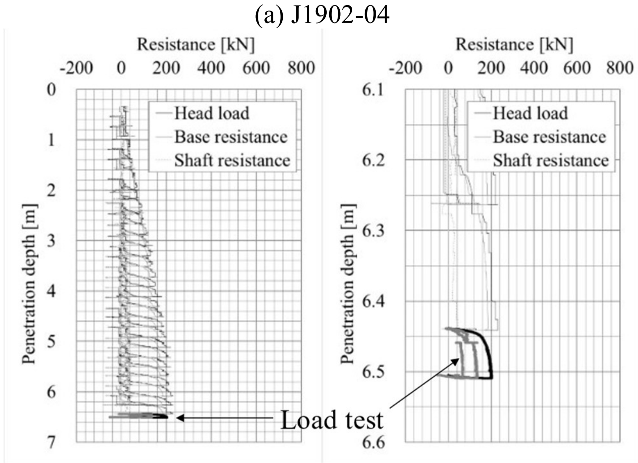
4.2.1 *Penetration rate: mechanism [R1]*

Figure 9 shows the pore water pressure measured on the pile shaft (u_p) during installation in J1902-06 (closed-ended, without surging), together with the theoretical static pore water pressure. Reduced pressures were recorded in u_{p-2} and u_{p-3} by up to 5 kPa, which will lead to an increase in Q_s at EOI by up to around 10 kN. After EOI, values of u_{p-2} and u_{p-3} recovered to their static values immediately (in less than 10 seconds) as shown in Figure 10, where zero in the horizontal axis was taken as the point of time when Q became zero and z became constant (i.e. when the rebound finished).

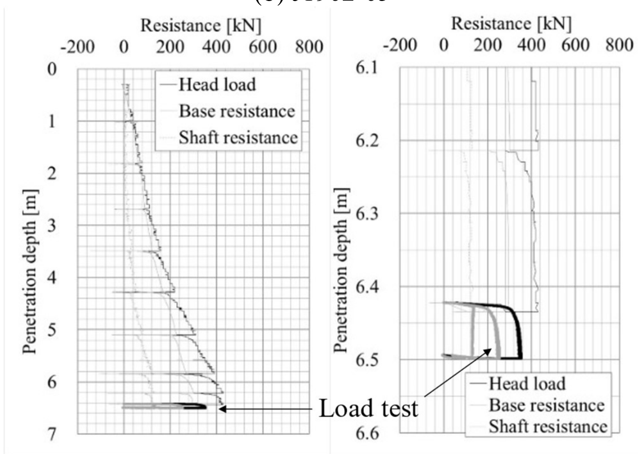
Figure 11 shows the axial strain – volumetric strain relationships obtained in triaxial tests (CD) on saturated NSK-40 sand, with the relative density (D_r) at 4% which was confirmed as similar to 1 m BGL in the model ground. The cylindrical sample had a diameter and height of 50 mm and 10 mm respectively. The strain rate was 0.5 %/min. There was no tendency of positive dilatancy under the confinement stresses of 50, 100 and 200 kPa. These stresses are lower than



(a) J1902-04



(b) J1902-05



(c) J1902-06

Figure 8. Resistances during installation and load test (bold lines) in J1902 test series

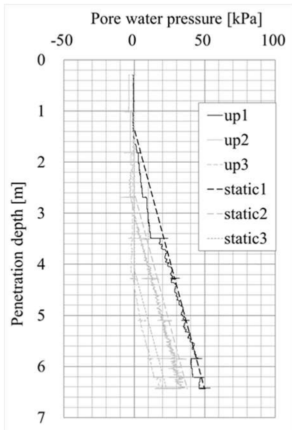


Figure 9. Pore water pressure on pile shaft during installation in J1902-06

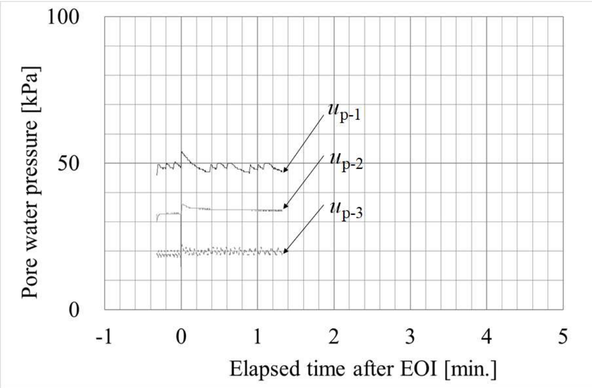


Figure 10. Variation of pore water pressure with time at EOI in J1902-06

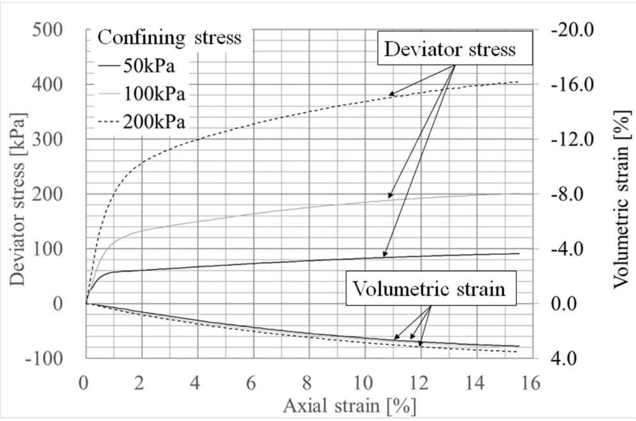


Figure 11. Results of triaxial test on NSK-40 sand

the base resistance recorded during press-in, and it is not clear whether this soil shows positive dilatancy under the higher confining stresses during installation. However, judging from the experimental data, the reduction in u_p values (which might have been caused by dilation) was recorded at levels away from the pile base while the set-down was observed not in Q_s but only in Q_b. This suggests that mechanism [R1] is not the reason for the set-down observed in J1902-06, although it has to be noted that this discussion is limited to pore pressure measurement on the pile shaft as there was no pore pressure information below the pile base.

4.2.2 *Penetration rate: mechanism [R2]*

It was confirmed by a consolidation test on NSK-40 that the coefficient of consolidation (c_v) at the average consolidation pressure of around 3.5 MPa, which roughly corresponds to the unit base resistance during installation, is around 3.8x10^3 cm^2/day. On the other hand, the penetration rate (v_d) in J1902-06 varied from around 40 to 50 mm/s as a whole. It follows that the normalized velocity V (= $v_d D_o / c_v$) in this experiment was approximately 3300 in installation. Considering the loading record in the load test in Figure 12, V was about 0.3 in the load test. According to Figure 3 which applies to clays, these V values mean that the installation was fully undrained while the load test was partially drained, and the unit base resistance (q_b) in the load test will be smaller than that during

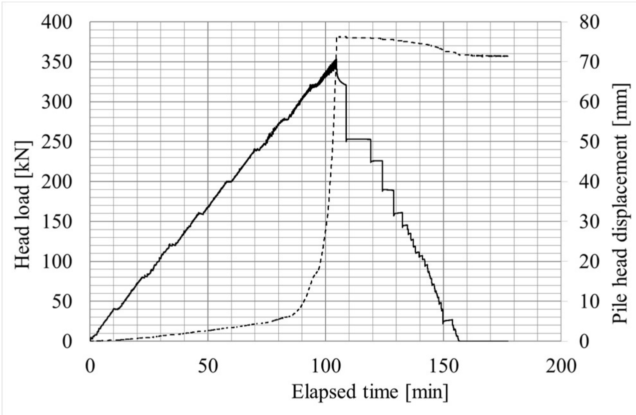


Figure 12. Loading record in J1902-06 load test

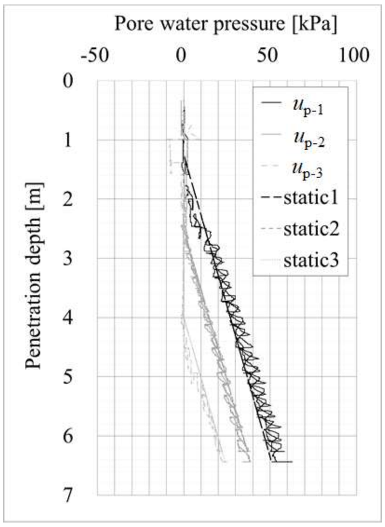


Figure 13. Pore water pressure on pile shaft during installation in J1902-05

installation. However, this effect of viscosity at high penetration rates could be assumed to be absent in J1902 test series, where the model ground consisted of loose permeable sands. In this context, the mechanism [R2] may not be the main factor for the set-down observed in J1902-06.

4.2.3 *Surging: mechanism [S1]*

Figure 13 shows the pore water pressure measured on the pile shaft (u_p) during installation in J1902-05 (with high water level). The excess pore water pressure was negative during the upward motion, while it was positive during the downward motion. However, these values were small regardless of their sign conventions, resulting in little variation after EOI as shown in Figure 14. Therefore, the mechanism [S1] is not thought to be the main factor for the set-down observed in J1902-05.

On the other hand, in J1902-04 (closed-ended, with low water level), a greater extent of negative excess pore pressure was recorded near the pile base (u_{p-1} and u_{p-2}) than in J1902-05 (closed-ended, with high water level), as can be confirmed in Figure 15. This suggests that the suction generated by the creation of

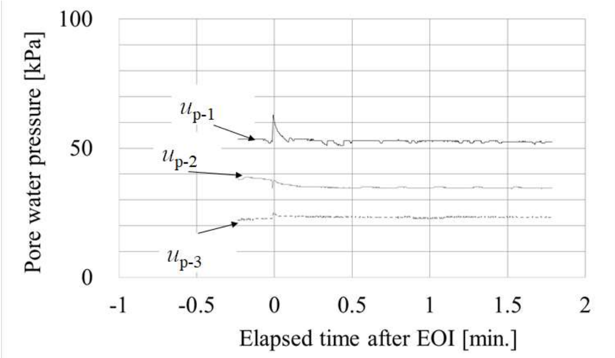


Figure 14. Variation of pore water pressure with time at EOI in J1902-05

a void beneath the pile base when the pile was moved upwards continued to exist during installation because of the lack of u_p building up in the subsequent downward motion of the pile. As well as this, the greater suction (because of the lower water level) may have led to a reduced volume of soil collapsing into the void when the pile was moved upwards and thus increased the extent of the additional suction induced by the pile's upward motion. As shown in Figure 16, values of u_{p-1} and u_{p-2} increased during the period from EOI to the start of load test by 5～15 kPa. This will cause 10～30 kN decrease in Q_s, which partly explains the reduction in Q_s in Figure 8. However, u_{p-3} decreased from a positive value by around 5kPa after EOI. It would be concluded that the mechanism [S1] could be a minor cause for the observed set-down in Q_s in J1902-04.

4.2.4 *Surging: mechanism [S2]*

Looking at Figure 8, Q_s during installation were lower in J1902-05 (closed-ended, with surging) than that in J1902-06 (closed-ended, without surging). This suggests either that there was little promotion of densification (and dilatancy) of soils around the pile shaft due to surging, or that the densification was promoted but the reduction in the confining stress (and resultant friction fatigue) was more influential than the increase in dilation even during installation. Therefore, [S2] is not thought to be the main mechanism for the set-down observed in J1902-05.

4.2.5 *Others: mechanism [O1]*

Figure 17 is the comparison of q_b and CPT corrected cone resistance (q_t). In J1902-04 (closed-ended, with low water level), the depth of EOI was where q_t tends to decrease with depth. This suggests that the mechanism [O1] is the cause of the set-down observed in J1902-04. On the other hand, in J1902-05 and J1902-06 (closed-ended, with high water level), at the depth of EOI, q_t tends to increase with depth. However, it is generally understood that Q_b reflects the strength of the soils not only at the depth of the pile base but also those around the pile base. If q_t is averaged by the Dutch method (Lehane *et al.*, 2005) to consider the strength of the soils around the pile base, the averaged

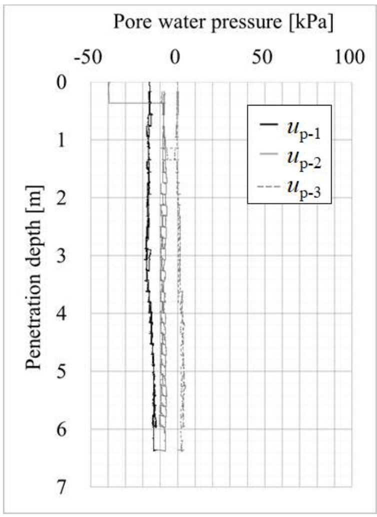


Figure 15. Pore water pressure on pile shaft during installation in J1902-04

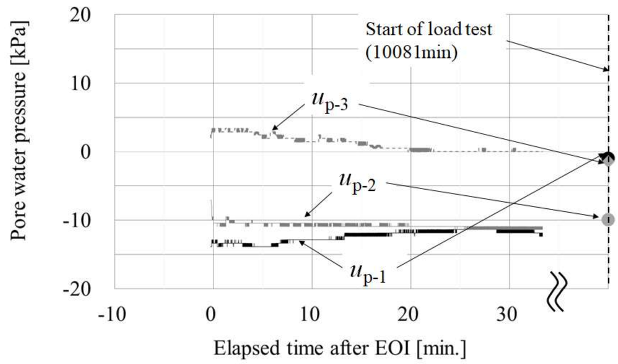


Figure 16. Variation of pore water pressure with time after EOI in J1902-04

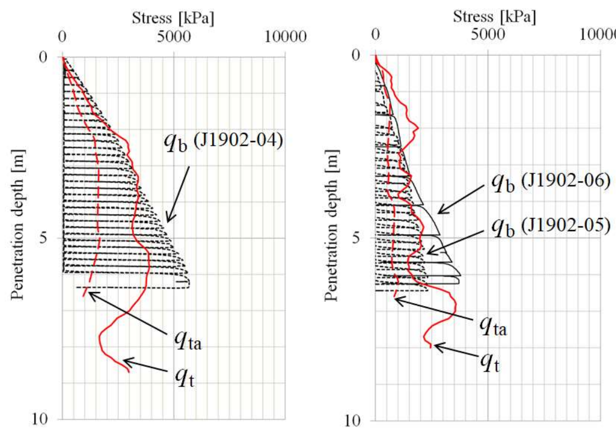


Figure 17. Comparison of q_b and CPT q_t in J1902 test series

q_t (denoted by q_{ta}) tends to decrease slightly with depth, reflecting the decrease in q_t values at greater depth. In light of this, [O1] could be the mechanism for the set-down in J1902-05 and J1902-06 as well.

4.2.6 *Others: mechanism [O2]*

Looking at Figure 18, Q_b almost reached its residual values during load test, at the pile head displacement of 30 mm which is lower than the displacement defining the capacity. This suggests that the mechanism

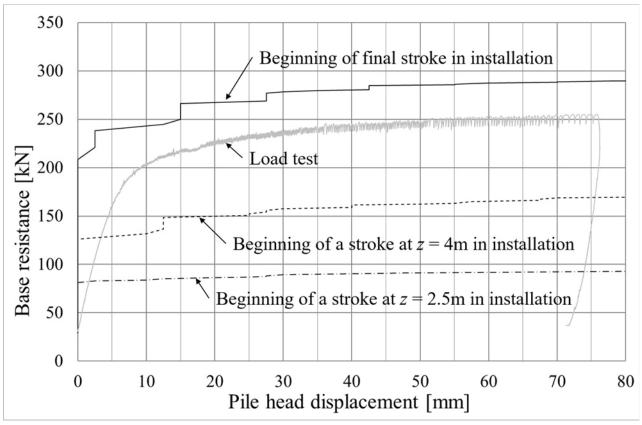


Figure 18. Comparison of base resistance at the beginning of each stroke in installation and in load test in J1902-06

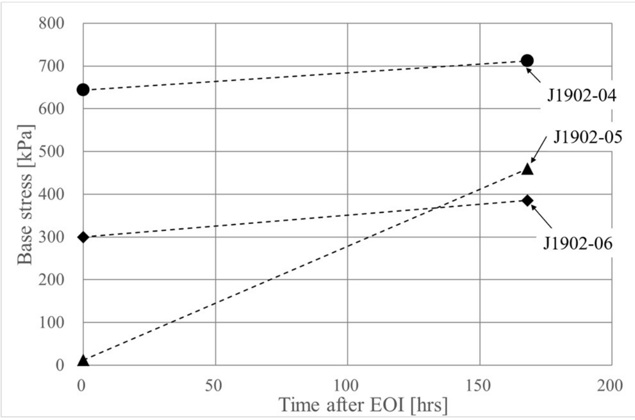


Figure 19. Base stress after EOI and before load test

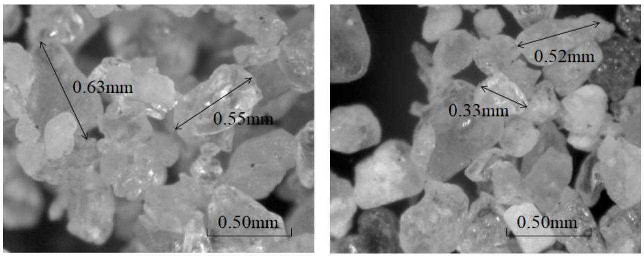
[O2] would not be the reason for the observed set-down in the three tests.

4.2.7 *Others: mechanism [O3]*

Figure 19 shows the base stress just after EOI (i.e. after rebound) and just before the start of load test in each test. Stresses were almost constant (with a slight increase) after EOI except for J1902-05, with the maximum stress level being around 700 kPa. This does not seem to be high enough to induce particle breakage, which was confirmed by taking images of NSK-40 sand particles (Figure 20) before and after applying a constant stress of 735 kPa for 24 hours in a simple testing apparatus shown in Figure 21. Mechanism [O3] does not appear to be the reason for the observed set-down in the three tests.

4.2.8 *Others: mechanisms [O4] and [O5]*

Axelsson (2000) argued that the increase in horizontal stresses measured on the pile shaft (σ_{hp}) in load tests becomes definitive due to the "confined dilatancy". Figure 22 shows the comparison of σ_{hp} during the initial loading in the final stroke in installation and in the load test in J1902-06. In the beginning of the load test, σ_{hp} values decreased with displacement, and were lower than those at the beginning of the final stroke of installation at the displacement when the peak shaft resistance was recorded. In other words, the trend of confined dilatancy was not seen at the



(a) Before test (b) After test

Figure 20. Sand particles before and after being subjected to a constant stress

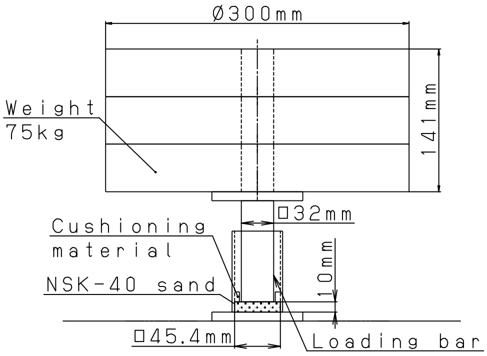


Figure 21. Apparatus to apply a constant stress to sand particles

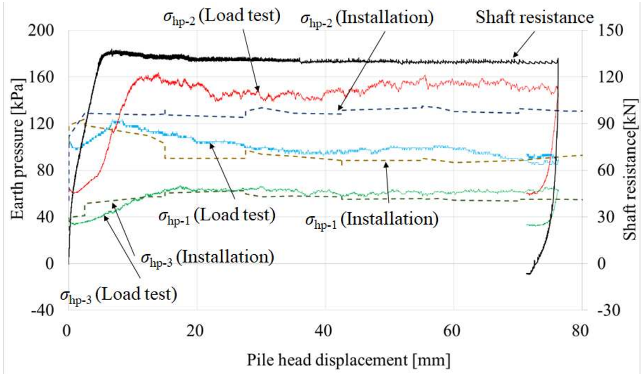


Figure 22. Earth pressure on pile shaft in the final stroke in installation and in load test (J1902-06)

displacements lower than those at the peak shaft resistance. This suggests the possibility of the mechanism [O4] in J1902-06.

Figure 23 shows the variation of σ_{hp}, σ_{hr} and $\sigma_{h\theta}$ with time after EOI in J1902-06. It was found that $\sigma_{h\theta}$ slightly increased shortly (in less than 30 seconds) after EOI and remained constant until the start of the load test, and the opposite trend was found for σ_{hr}. σ_{hp} showed a similar trend as σ_{hr}, decreasing shortly after EOI and remain almost constant until the start of the load test. These trends suggest that mechanism [O5] could contribute to the observed set-down in J1902-06.

4.3 *Additional experiments and discussions*

As explained in Section 3, additional tests (J2002 test series) were conducted with an improved soil condition using an open-ended pile. Figure 24 shows the variation of resistances in these tests. Values of Q at

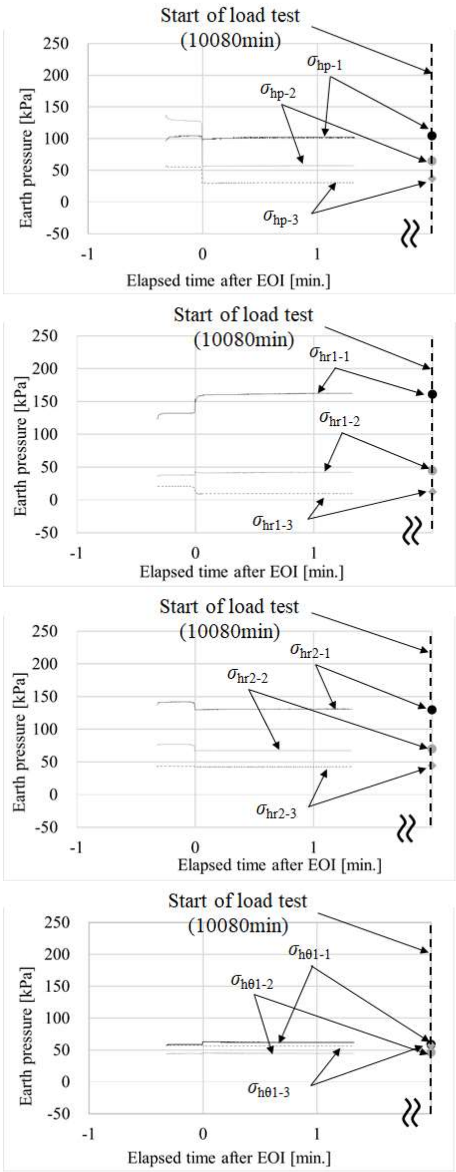
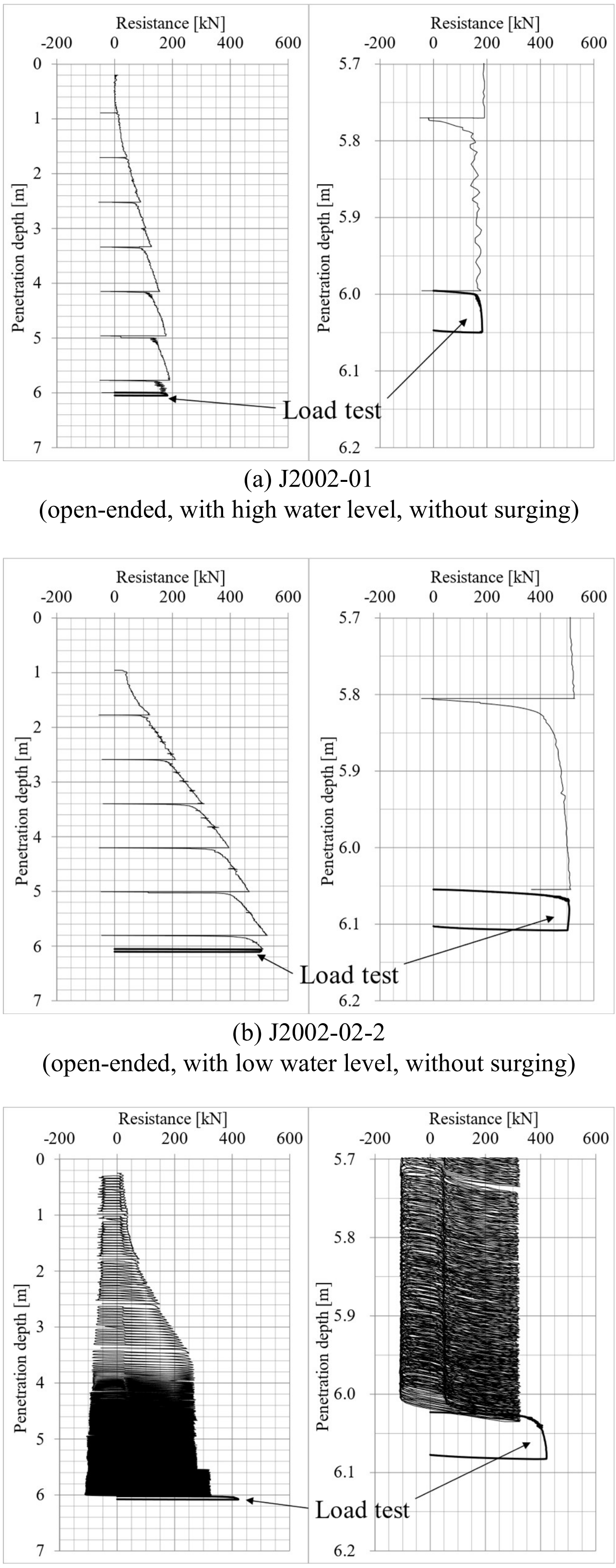


Figure 23. Variation of σ_{hp}, σ_{hr}, and $\sigma_{h\theta}$ after EOI in J1902-06

larger displacements in the load test were comparable to or slightly greater than that at EOI. This increase was more apparent in J2002-01 (open-ended, with high water level) than in J2002-02-2 (open-ended, with low water level). In J2002-03 (open-ended, with low water level, with upper limit of jacking force (Q'_{UL})), this increase was more significant. This result seems to be consistent with the centrifuge test results by Burali d'Arezzo *et al*. (2013) where a large number of cyclic motions of the pile induced a significant increase in the base resistance when the pile is installed further, which would have been caused by the compaction of the soil beneath the pile base. It is often the case that the press-in piling is conducted with Q'_{UL}, and this seems to lead to greater pile set-up than what would be experienced in a pile installed without Q'_{UL}.

From the experimental evidence that the set-up trend was found consistently in three tests in J2020 test series, and that the differences of test conditions in J2020 and J1902 test series were the ground



(a) J2002-01
(open-ended, with high water level, without surging)

(b) J2002-02-2
(open-ended, with low water level, without surging)

(c) J2002-03
(open-ended, with low water level,
with load-controlled surging)

Figure 24. Resistances in installation and in load test (bold lines) in J2002 test series

condition and the pile base condition (open or closed), it is suggested that the observed set-down in J1902 test series was caused mainly by mechanism [O1], and to a lesser extent by [O4], [O5] and [S1] as discussed in Section 4.2, and additionally by [O6].

5 CONCLUSIONS

Large scale model tests were conducted to compare the penetration resistance between installation and load tests in loose permeable sands. A trend of pile set-down was observed in some tests using closed-ended piles. Conversely, in additional model tests conducted in an improved model ground using an open-ended pile, a trend of pile set-up (or no trend of pile set-down) was observed.

Through literature review of the mechanisms of pile set-down and detailed analyses on the experimental data, the set-down trends observed in tests using a closed-ended pile were thought to be caused by (1) mechanism [O1], the decreasing trend of soil strength with depth at around the pile base, leading to the set-down in base resistance (Q_b); (2) to a lesser extent mechanisms [O4], [O5] and [O6], the lack of factors that lead to set-up in shaft resistance (Q_s) or in plug strength; and (3) similarly mechanism [S1], negative pore water pressure during installation, induced by surging in unsaturated soils, leading to minor set-down in Q_s.

Further model tests in an improved ground using a closed-ended pile, as well as triaxial tests with higher strain rates, will be effective in assessing these observations and suggested mechanisms. In addition, investigating the pore water pressure behavior beneath the pile base will allow a more reliable discussion on the causes of set-up or set-down of base resistance.

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