Influence of Cyclic Behaviour of Vibratory Pile Driving and Surging on Pile Performance Observed in Model Load Tests in Dry and Saturated Sand Grounds

S. Moriyasu[1], M. Aizawa[2], T. Matsumoto[3], S. Kobayashi[3],and S. Shimono[3]
[1] *Steel Structures Research Lab, NIPPON STEEL & SUMITOMO METAL CORPORATION, Chiba, Japan*
[2]*Toyama Prefecture, Japan (former student of Kanazawa University)*
[3] *Graduate School of Natural Science and Technology, Kanazawa University, Ishikawa, Japan*
E-mail: moriyasu.45e.shunsuke@jp.nssmc.com[1], matsumoto@se.kanazawa-u.ac.jp[2], koba@se.kanazawa-u.ac.jp[3]

ABSTRACT: This study focuses on the effect of “cyclic” behaviour of pile installation methods on the penetration resistance and bearing capacity. A series of laboratory model test was conducted to investigate the cyclic effect by comparing three kinds of piling methods; monotonic jack-in, pseudo-dynamic push-in and pull-out, i.e. “surging” and vibratory driving in dry or saturated sand grounds. It was found that surging or vibratory pile driving decreased the pile penetration resistance due to the negative dilation caused by the cyclic shearing of the soil surrounding the pile. The static load test showed that surging and vibratory pile driving provided the same or larger pile head load than jack-in method. Furthermore, the fluctuation of the pore water pressure strongly indicated the change of dilation. It was seen that surging and vibratory pile driving prevent the positive dilation than jack-in method due to the difference of cyclic shearing and monotonic loading.
Keywords: Pile installation method, jack-in, surging, vibratory pile driving, pore water pressure.

1. INTRODUCTION

Recently, it is rather difficult to employ the impact hammer method in urban areas and residential area, because it causes noises and ground vibrations. Vibratory pile driving and rotary pile installation (jack-in) methods have been developed as the alternative methods, which can install the pile with less noises and ground vibrations. Previous studies about these methods mainly focused on the soil resistance during pile penetration. In contrast, only a few studies focused on the bearing capacity of the pile installed by these methods. If the bearing capacity is clarified, these methods will become more available.

The purpose of this study is to investigate influences of the different piling methods on the pile performance, such as penetration resistance, and bearing capacity and load-settlement relation during the pile load test. Significant factor is “cyclic” movement during pile penetration. Moriyasu et al. (2016) compared the pile behaviour between the monotonic jack-in and surging in dry model sand. Surging refers to the cyclic push-in and pull-out movement of a pile during installation. Cyclic shearing by surging caused negative dilation around the pile shaft and soil compaction below the pile tip.

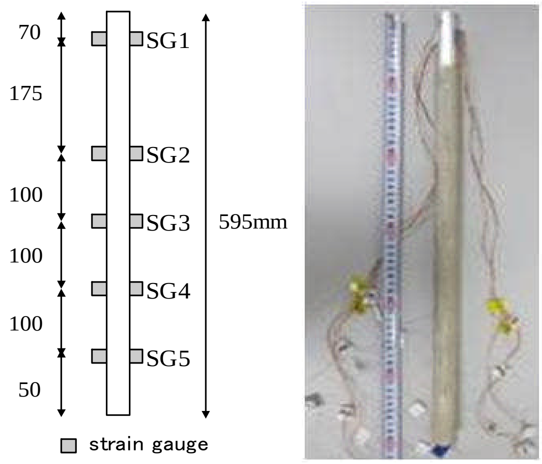
Thus, this paper focused on the influence of the dynamic surging on the pile penetration resistance and bearing capacity by comparing ~~to~~ with the influence the pseudo-dynamic surging. A series of laboratory tests were conducted in this paper. The laboratory tests included three kinds of piling method (monotonic jack-in, surging and vibratory pile driving) in two sand conditions (dry or saturated ground).

2. EXPERIMENT DESCRIPTION

2.1 Model pile

An aluminium pipe pile with an outer diameter of 32 mm, a wall thickness of 1.3 mm and a length of 595 mm was used for model piles. Regarding the pile tip condition, six cases were open-ended and one case was close-ended. In order to obtain the distribution of axial forces, strain gauges were attached to the pile shaft at 6 different levels, two strain gauges in the opposite faces at each level, as shown in Figure 1(a).

The pile surface was coated with an acrylic adhesive to protect the strain gauges, and glued with silica sand to increase the pile shaft resistance (Figure 1(b)). The sand was the same material as the model ground.



(a) location of strain gauges (b) photo of the pile
Figure 1 Model pile

2.2 Model ground

The material of the model ground was Silica sand #6. Table 1 lists the physical properties of the sand. The model ground was prepared in a rigid cylindrical soil container having a diameter of 566 mm and a height of 580 mm. The model ground was prepared with 11 soil layers of 50 mm thick and one layer of 30 mm thick. The sand for each soil layer was put into the soil box, and the sand layer was compacted by hand tamping to have a designated relative density, D_r = 80%. The procedure was repeated to complete the model ground of 530 mm high.

For the preparation of saturated ground, the soil box was filled with water. Then the sand was poured into the soil box and compacted, similarly to the case of the dry ground. Although D_r = 80% was intended also for the saturated ground, D_r = 70% was achieved in the saturated ground.

Table 1 Physical properties of the sand

Soil particle density, ρ_s	(t/m^3)	2.679
Minimum dry density, ρ_{dmin}	(t/m^3)	1.366
Maximum dry density, ρ_{dmax}	(t/m^3)	1.629
Maximum void ratio, e_{max}		0.962
Minimum void ratio, e_{min}		0.645
Mean particle size, D_{50}	(mm)	0.52

2.3 Experimental procedure and cases

Figures 2 and 3 show the experimental apparatuses. During pile penetration tests (PPT), the pile was installed by means of a motor jacks or a vibratory hammer. When the pile was penetrated to a depth of 400 mm, PPT was finished. Subsequently, during static load test (SLT), the pile head was pushed in two or three loading cycles. At the end of SLT, the pile extraction force was measured by extracting the pile slightly. Finally, Cone penetration test (CPT) was carried out to investigate the soil strength profile of the model ground.

Table 2 shows the test cases. Case 1 to Case 4 were carried out in the dry ground, while Case 5 to Case 7 were carried out in the saturated ground.

For each ground condition, three kinds of penetration methods were employed. Jack-in method in Case 1 and Case 5 is the monotonic pile installation at a penetration rate of 0.2 mm/s. Surging method in Case 2 and Case 6 means the cyclic installation with repetition of 2 mm jack-in and 1 mm pull-out. In these methods, the pile was installed by using the jack without pile rotation. In the vibration method in Cases 3, 4 and 7, a vibratory hammer model was employed. As shown in Figure 3, the hammer has a tandem rotating eccentric weights which generates vertical vibrations and cancels horizontal vibrations. The vibratory hammer has a weight of 300 N and a maximum frequency of 60 Hz. The close-ended pile was used in Case 4, while the open-ended pile was employed in the other cases.

SLT was carried out subsequent to PPT. In all cases, the pile penetration rate was 0.1 mm/s.

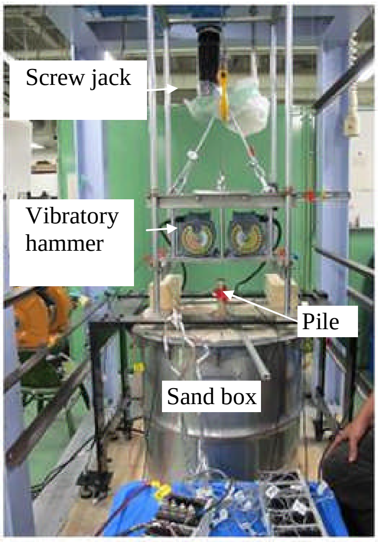


Figure 2 Loading apparatus

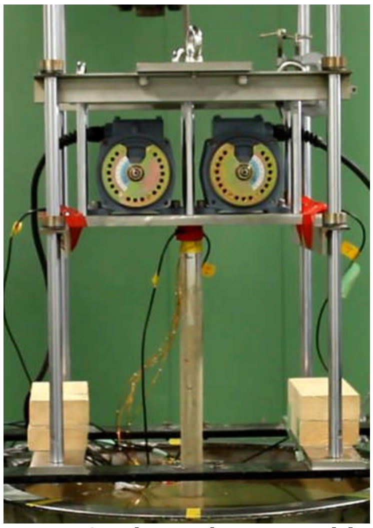


Figure 3 Vibratory hammer model

Table 2 Experimental cases and conditions

Case No.	1	2	3	4	5	6	7
Model ground	Dry	Dry	Dry	Dry	Saturated	Saturated	Saturated
Relative density, D_r (%)	79.9	79.9	80	80	69.5	64.3	69.5
Dry density, ρ_d (t/m^3)	1.568	1.568	1.568	1.568	1.538	1.524	1.538
Pile tip condition	Open	Open	Open	Close	Open	Open	Open
Penetration method	Jack-in	Surging	Vibration	Vibration	Jack-in	Surging	Vibration
Penetration speed during PPT (mm/s)	0.2	0.2	-	-	0.2	0.2	-
Penetration speed during SLT (mm/s)	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Vibration frequency (Hz)	-	-	20 to 35Hz	20 to 35Hz	-	-	15 to 20Hz
Test sequence	PPT*1 ↓ SLT*2 ↓ CPT*3	PPT*1 ↓ SLT*2 ↓ CPT*3	Static loading by V.H. *4 weight ↓ Vibratory penetration ↓ SLT*2 ↓ CPT*3	Static loading by V.H. *4 weight ↓ Vibratory penetration ↓ SLT*2 ↓ CPT*3	PPT*1 ↓ SLT*2 ↓ CPT*3	PPT*1 ↓ SLT*2 ↓ CPT*3	Static loading by V.H. *4 weight ↓ Vibratory penetration ↓ SLT*2 ↓ CPT*3

*1 PPT: Pile Penetration Test, *2 SLT: Static Loading Test, *3 CPT: Cone Penetration Test, *4 V.H.: Vibratory Hammer

2.4 Measurements

During PPT and SLT, the pile head load, pile head displacement and pile strains were measured. In the cases of jack-in and surging, the pile head load was measured by a load cell set between the pile head and motor jack. On the other hand, in the cases of vibration, the axial force was estimated from the strain measured near the pile head (i.e., SG level 1 in Figure 1(a)). Furthermore, a pair of acceleration transducers was attached near the pile head. In the saturated sand cases, pore water pressures were measured by the transducers located as shown in Figure 4.

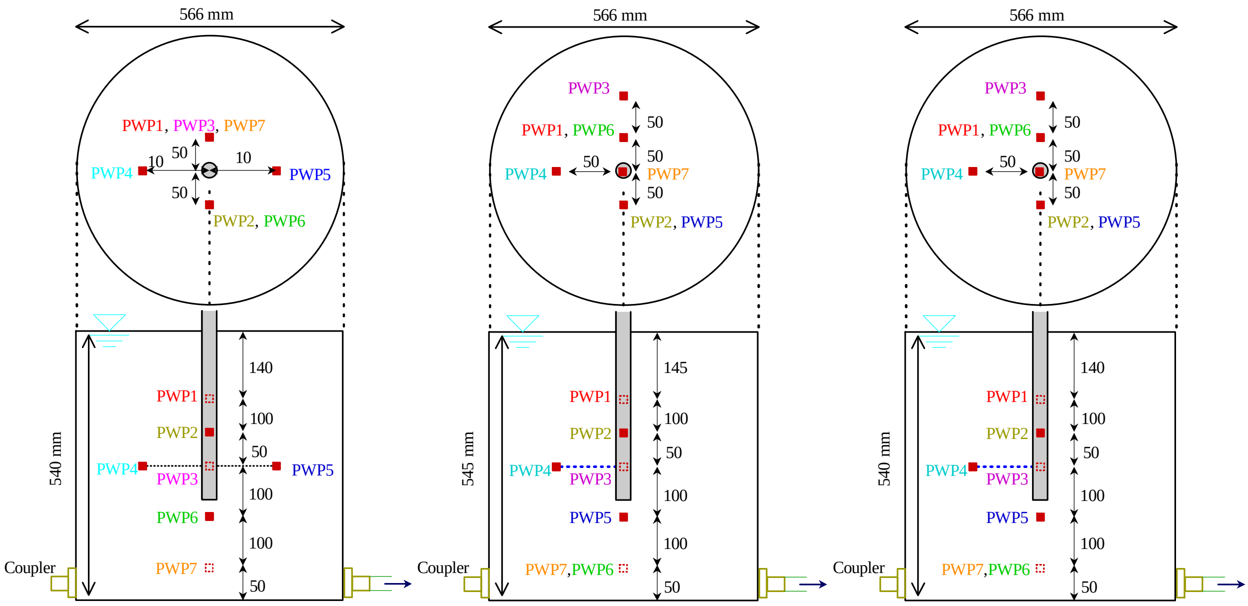
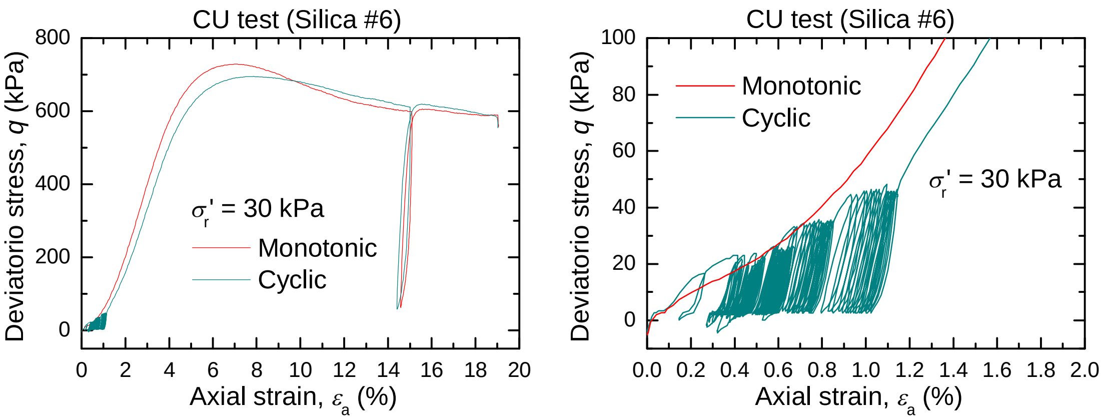


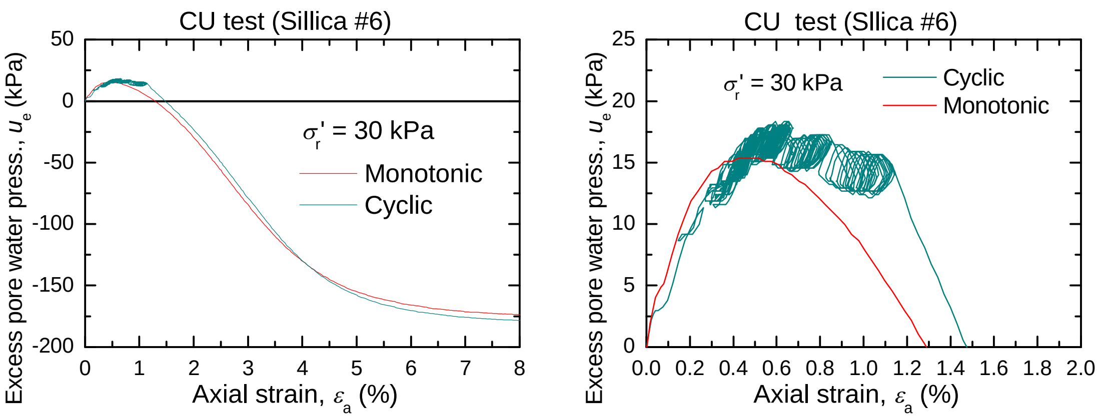
Figure 4 Locations of 7 pore water pressure transducers in Cases 5 to 7



(a) General view

(b) Enlarged view

Figure 5 Relationship between the axial strain, $\varepsilon_{a,}$, and the devlatoric stress, q



(a) General view

(b) Enlarged view

Figure 6 Relationship between the axial strain, ε_a, and the excess pore water pressure, u_e

2.5 Monotonic and cyclic triaxial shear tests of the sand

In order to investigate the mechanical property of the sand, a monotonic and a cyclic consolidated undrained (CU) shear tests of the sand were carried out.

Figure 5 shows the relationship between the axial strain, ε_a, and the deviatoric stress, q. Figure 6 shows the relationship between ε_a and the excess pore water pressure, u_e. Figure 7 shows the effective stress paths. The confining pressure of these tests was about 30 kPa. The internal frictional angle was 38 to 40 degrees, and the shift phase angle was 28 degrees.

During cyclic loading, it is seen from Figure 5 that ε_a increased with accumulation of the number of cycle loading with each constant q. At the same time, cyclic loading generated excess pore water pressure higher than that in monotonic loading. At the monotonic loading stage after the cyclic loading, the increment of u_e

turned from positive to negative, because the negative dilation changed to positive one. The change point in the cyclic loading (ε_a = 1.2 %) was delayed behind the case of monotonic loading (ε_a = 0.4 %). Therefore, the cyclic loading may have an action to prevent the positive dilation. The soil behaviour is referred to discuss the experiment results later.

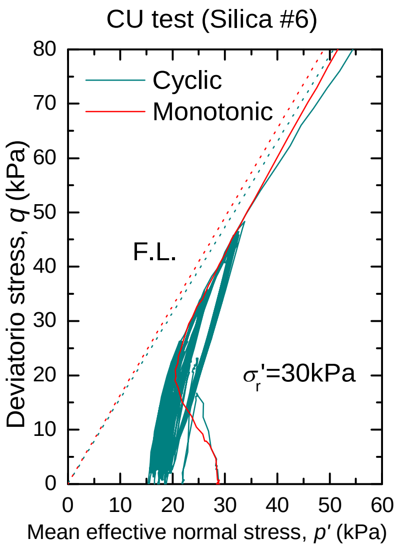


Figure 7 Effective stress channel paths

3. EXPERIMENT RESULTS

3.1 Results of Cone Penetration Tests in the model ground

Figure 8 shows the vertical effective stresses, σ_v', with depth in the cases of dry and saturated grounds. It is noted that ground water levels in Cases 5 and 7 were 100 mm below from the ground surface. Figure 9 shows the CPT tip resistance, q_c, measured in Case 2 (dry ground) and Case 6 (saturated ground). The CPTs were carried out after SLT. CPT profiles of the other cases were similar to Figure 9. The q_c in the saturated ground (Case 6) was smaller than that of the dry ground (Case 2) due to smaller σ_v' in the saturated ground. Furthermore, q_c may be influenced by the soil behaviour during PPT and SLT, because CPT was carried out after the PPT and SLT.

3.2 Results of the pile penetration tests

Figure 10 shows the time history of the accelerations of the pile head, the ground surface and the vibratory hammer during the vibratory pile penetration in Case 4. The downward acceleration is taken as positive. As shown in the figure, the acceleration of pile head was much larger than those of the ground surface and the vibratory hammer. The frequency of the vibratory hammer was changed from 20 Hz to 35 Hz to enhance the pile penetration ability of the vibratory hammer during PPT.

Figure 11 shows the relationship between the pile head load, P_h, and pile head displacement, w_h, in Cases 1 to 4 (dry ground). It can be seen that P_h in Case 3 and Case 4 during PPT was almost similar. In Cases 3 and 4, at first, the pile was installed by the self-weight of the vibratory hammer from the ground surface. When the pile could not be penetrated by the self-weight at w_h = 75 mm, the operation of the vibratory hammer was started. Until the pile reached w_h = 400 mm, the frequency of the hammer was increased if the pile penetration was degraded. P_h in Case 3 and Case 4 (vibration) was smaller than that of Case 1 and Case 2 after w_h exceeded 200 mm.

Figure 12 shows the relationship between P_h and w_h in Cases 5 to 7 (saturated ground). During PPT, P_h in Case 6 (surging) was slightly smaller than that in Case 5 (jack-in). In Case 7 (vibration), P_h decreased significantly, when f was increased to 18.3Hz at w_h = 400 mm.

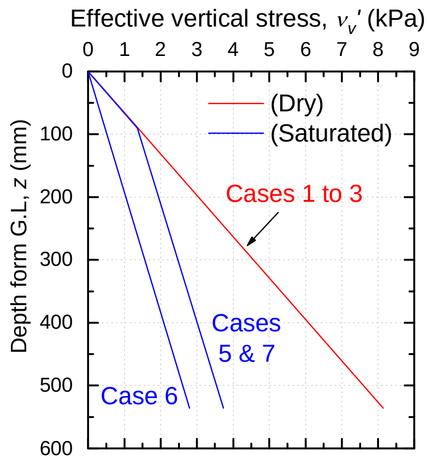


Figure 8 Vertical effective stress

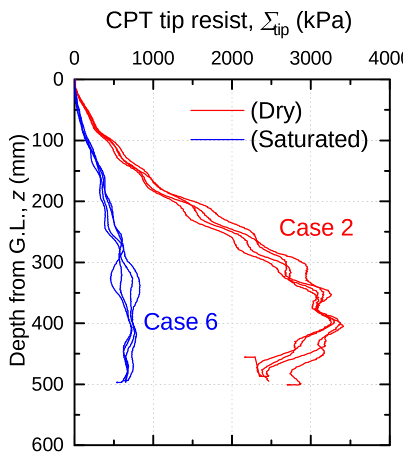


Figure 9 CPT results

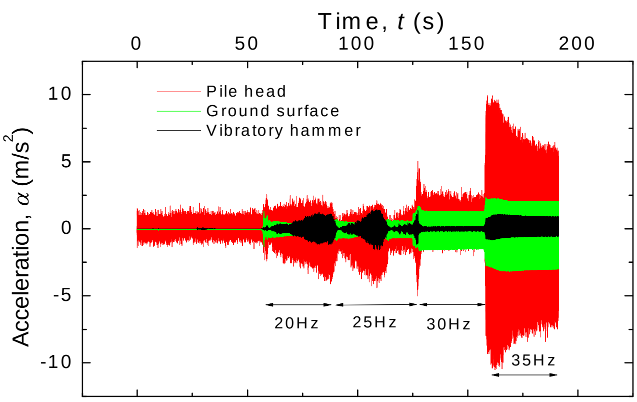
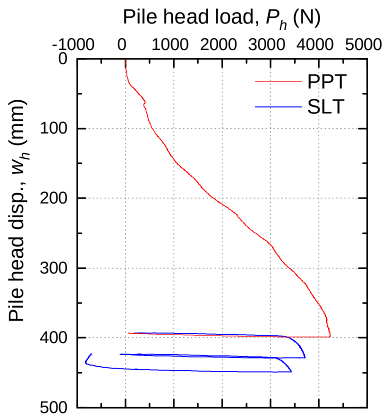
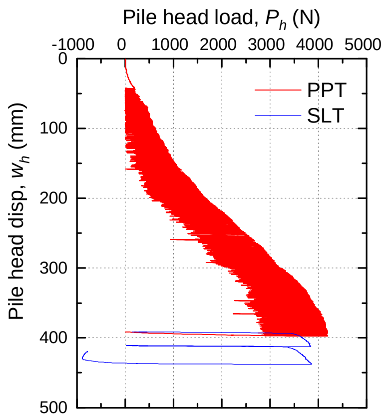


Figure 10 Acceleration during vibratory penetration

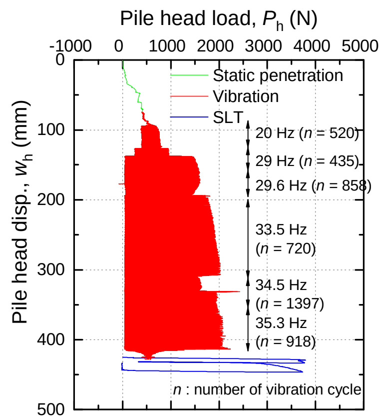
Figure 13 shows the relationship between the pore water pressure, p, and pile head displacement, w_h, during PPT. In Case 7, p increased with increases in w_h, and p increased dramatically when f was increased to 18.3 Hz at w_h = 400 mm. It is seen that soil liquefaction was caused, because the excess pore water pressure corresponded to the effective vertical stress, σ_v, shown in Figure 8. Therefore, P_h decreased with the reduction of σ_v related to the soil liquefaction. As mentioned earlier, the CU test shows that cyclic shearing generated higher excess pore water pressure than monotonic loading, as shown in Figure 6. If the degree of the excess pore water pressure depends on the number of cyclic shearing, the vibration provided a large number of cyclic shearing (3800 times) than surging (350 times).



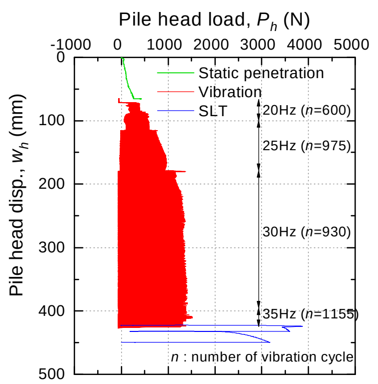
(a) Case 1 (Dry, Push-in)



(b) Case 2 (Dry, Surging)

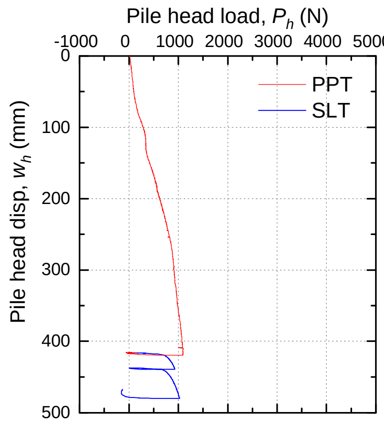


(c) Case 3 (Dry, Vibration)

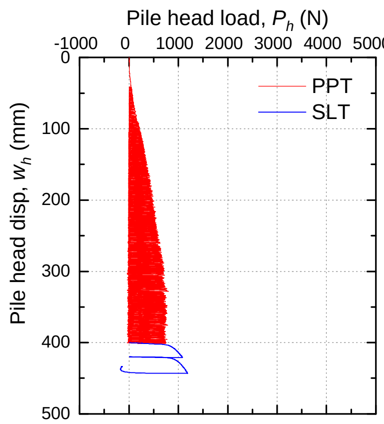


(d) Case 4 (Dry, Vibration, close-ended pile)

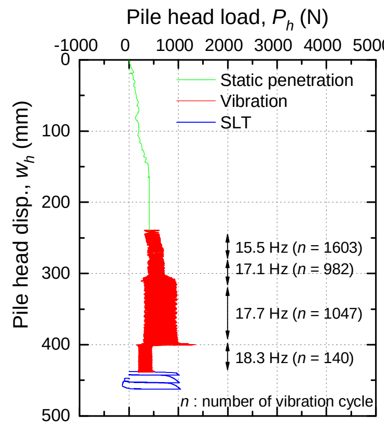
Figure 11 Relationship between the pile head load, P_h, and the pile head displacement, w_h, in Cases 1 to 4



(a) Case 5 (Saturated, Push-in)



(b) Case 6 (Saturated, Surging)



(c) Case 7 (Saturated, Vibratory)

Figure 12 Relationship between the pile head load, P_h, and the pile head displacement, w_h, in Cases 5 to 7

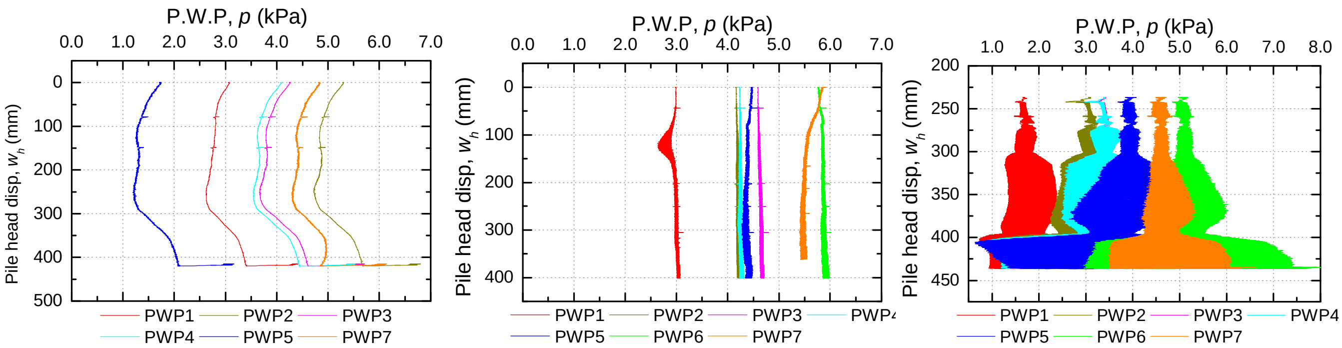
3.3 Results of the static load test

The static load test was carried out subsequent to the pile penetration test. As shown in Figure 11 (a) (b), the pile head load, P_h, during SLT was smaller than that during PPT. A possible reason is the difference of the pile penetration rate. Watanabe et al. (2012) investigated that the failure strength and deformation modulus of sand increases with increasing the loading rate. The similar behaviour may appeared. Conversely, in the Case 3 and Case 4, P_h during SLT was higher than that during PPT. A possible reason is the generation of excess pore air pressure during PPT. Watanabe et al. (2013) investigated that excess pore air pressure is generated in dry sand when it is sheared very rapidly in the triaxial compression testing. In order to clarify the behaviour, more study is needed.

Figure 14 (a) shows the comparison of load-settlement relations between Case 1 (jack-in) and Case 2 (surging) in dry sand condition. It should be noted that the origin of vertical axis of Figure 14 (i.e, pile head displacement during SLT, w_{h_SLT}) is the end of w_h during PPT due to adjust the difference end of w_h among cases. #1 in each figure means the yield point in the 1st loading cycle, and #2 is the end point of increasing load. As shown in Figure 14 (a), the initial pile head stiffness in Case 2 (surging) is higher than that in Case 1 (jack-in), and the yield point (#1) in Case 2 is larger than that in Case 1. Figure 15 shows the change of pile shaft resistance and pile base resistance from #1 to #2. The pile base resistance, P_b, is the axial force obtained from strain SG5, and the pile shaft resistance is the difference between the pile head load, P_h, and the pile base resistance. The pile base resistance in Case 2 (surging) is higher than that in Case 1. This is similar to Moriyasu et al. (2016).

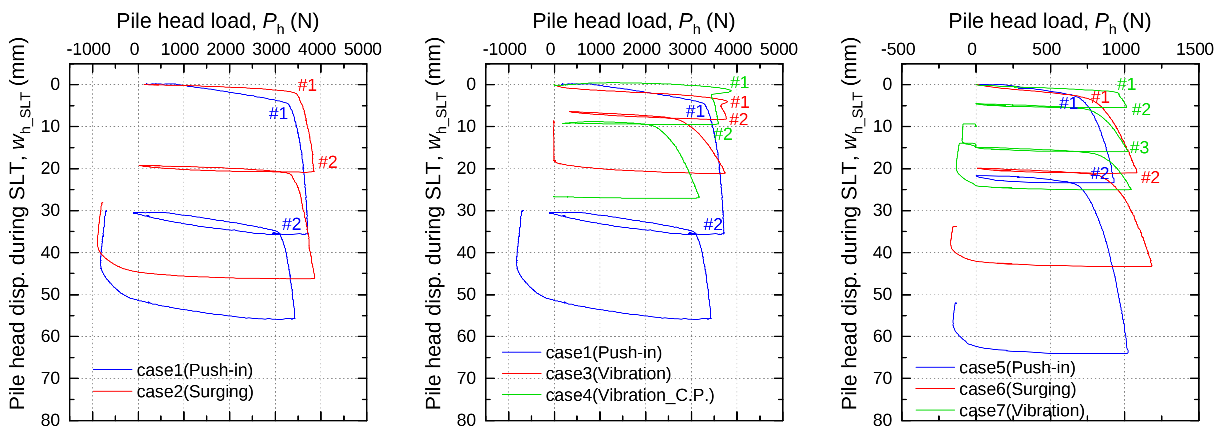
As Bolton et al. (2013) pointed out, the cavity created below the pile tip collapses during the upward movement of the pile, and the soil below the pile tip is compacted during the following downward movement. This cyclic compaction enhanced the pile base resistance largely.

Figure 14 (b) shows the comparison of load-settlement relations among Case 1 (jack-in), Case 3 and Case 4 (vibration). The yield loads (#1) in Case 3 and Case 4 (vibration) are larger than that in Case 1. However, a softening behaviour occurred: i.e., P_h increased to 3878 N at #1, and dropped immediately to P_h = 3604 N with a little increasing of w_{h_SLT}. It can be seen from Figure 15 (a) that the pile shaft resistance in Case 4 decreased with increasing w_{h_SLT} (from #1 to #2), while the pile base resistance increased. This is not related with soil plugging because the behaviour was occurred both condition of open-ended pile (Case 3) and close-ended pile (Case 4).



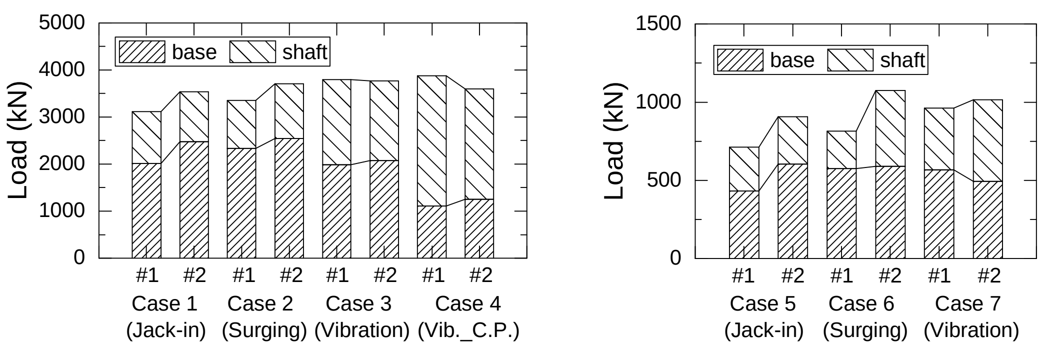
(a) Case 5(Jack-in) (b) Case 6 (Surging) (c) Case 7 (Vibration)

Figure 13 Relationship between the pile head displacement, w_h, and the water pressure (P.W.P.), p, during PPT



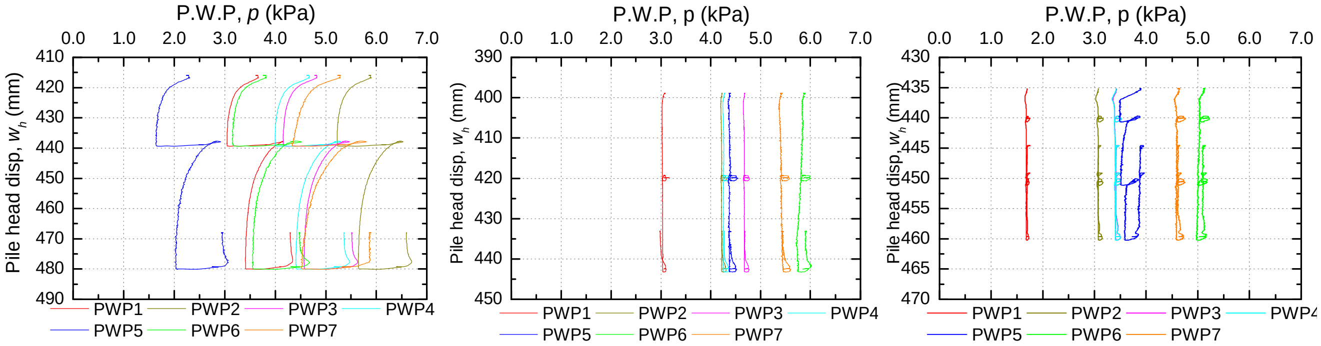
(a) Cases 1 and 2 (dry ground) (b) Cases 1, 3 and 4 (dry ground) (c) Cases 5, 6 and 7 (saturated ground)

Figure 14 Relationship between the pile head load, P_h, and the pile head displacement, w_{h_SLT}



(a) dry ground (b) saturated ground

Figure 15 Change of pile shaft resistance and pile base resistance from #1 to #2 marked in Figure 14



(a) Case 5 (Jack-in) (b) Case 6 (Surging) (c) Case 7 (Vibration)

Fig. 16 Relationship between the pile head displacement, w_h, and the water pressure (P.W.P.), p, in SLT

Figure 14 (c) shows the load-displacement curves in Cases 5 to 7 (saturated ground). The initial pile head stiffness in Case 6 (surging) was comparable to that in Case 5 (jack-in), and that in Case 7 (vibration) was higher than that in Case 5 and Case 6. Conversely to the result in Cases 3 and 4 (dry ground), the decrease of P_h in the 1st loading cycle did not occur in Case 7. As shown in Figure 15 (b), the pile base resistance in Case 6 (surging) and Case 7 (vibration) was larger than that in Case 5 (jack-in). And, the pile shaft resistance increased with increase in w_{h_SLT} (from #1 to #2). Additionally, when the pile was extracted at the final stage of SLT, the extraction resistance was obtained. It can be seen from Figure 14 (c) that the extraction resistance in Case 6 and Case 7 was comparable to that in Case 5.

Figure 16 shows the pore water pressure, p, during SLTs. It can be seen from Figure 16 (a) that p in Case 5 (jack-in) is closely related to the dilation of the ground. The negative p (i.e., positive dilation of the ground) increments were generated when the pile was loaded into the compressed direction, while positive p (i.e., negative dilation of the ground) increments were generated when the pile was unloaded to zero. Although the magnitudes of the pore water pressures in Case 6 (surging) and Case 7 (vibration) are smaller than those in Case 5 (jack-in), they have similar trend to those in Case 5. Furthermore, p in Case 7 was comparable to the other cases, although the soil liquefaction occurred at the final stage of PPT. Hence, it is thought that excess pore water pressure dissipated and the ground was reconsolidated during the preparation work for SLT.

As mentioned in 2.5., it can be seen in the CU test that cyclic shearing has the effect to prevent the positive dilation. If the similar situation occurs in the laboratory pile load test, the cyclic pile movement during PPTs in Case 6 (surging) and Case 7 (vibration) may prevent the positive dilation during SLTs.

4. CONCLUSIONS

In order to investigate the influence of cyclic pile movement on the penetration resistance and bearing capacity, a series of laboratory model tests were conducted with the different piling methods, i.e., monotonic jack-in, surging and vibratory pile driving, in dry or saturated sand grounds. Furthermore, triaxial CU tests of the sand used for the model ground were carried out to compare the soil behaviours under monotonic or cyclic shearing. The findings are as follows.

1) In the pile penetration tests in the dry ground, the pile head load (penetration resistance) in the surging piling method was comparable to that in the jack-in piling method. The pile penetration resistance in the vibratory driving was smaller than those in the jack-in and the surging piling methods. A possible reason for this is the generation of pore air pressure in the dry ground, although more investigation is needed for this aspect.
2) In the pile penetration tests in the saturated ground, the pile head load (penetration resistance) in the surging piling method was smaller than that in the jack-in piling method. The pile penetration resistance in the vibratory piling method was significantly reduced due to the occurrence of soil liquefaction. The cyclic pile movements may have caused negative dilation of the soil surrounding the pile, judging from the results of the cyclic shearing in the triaxial CU test. The vibratory pile driving may have caused negative dilation higher than surging due to a great number of loading cycles in the vibratory pile driving.
3) In the static load tests in the dry and saturated grounds, the pile head load of the pile installed by surging or vibratory pile driving was the same or larger than that of jack-in. Furthermore, the pile base resistance of the pile installed by surging or vibratory pile driving was relatively large, especially in the saturated ground. A reason for this result is that the cyclic compaction increased the soil resistance below the pile tip.
4) During the SLT in the saturated ground, the pore water pressures were rarely changed in the cases of surging and vibratory pile driving, while the negative pore water pressures were observed clearly in the case of jack-in piling method. Since the negative pore water pressure means positive dilation, the pile installed by jack-in caused the positive dilation of the ground during SLT. In contrast, surging and vibratory pile driving prevented the dilation of the ground during SLT due to the cyclic movement during PPT.

From these findings, it can be said that surging and vibratory pile driving install the pile with causing the negative dilation of the ground. And, the bearing capacity of the vibratory driven pile is comparable to that by jack-in piling method, because the cyclic pile movement enhances the pile base resistance.

5. ACKNOWLEDGEMENT

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6. REFERENCES

Bolton, M.D., Haigh, S.K., Shepley, P., and D'Arezzo, F.B. (2013) "Identifying ground interaction mechanisms for press-in piles". Press-in Engineering, Proc. 4th IPA International Workshop in Singapore, pp84-95.

Moriyasu, S., Meguro, H., Matsumoto, T., Kobayashi, S., and Shimono, S. (2016) "Influence of Surging and Jack-in Pile Installation Methods on Pile Performance Observed in Model Load Tests in Dry Sand Ground", 19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference (19SEAGC & 2AGSSEA), Kuala Lumpur.

Watanabe, K., Sahara, M. (2012) "Effect of Loading Rate on Bearing Capacity and Soil Spring of Pile Foundations", report of Obayashi Corporation Technical Research institute, No. 76.

Watanabe, K. and Kusakabe, O. (2013) "Reappraisal of Loading Rate Effects on Sand Behaviour in View of Seismic Design for Pile Foundations" Soils and Foundations, 53(2), pp.215-231.