Canadian Geotechnical Journal
Revue canadienne de géotechnique

**Field study on the axial bearing capacity and load transfer
characteristic of waveform micropile**

Journal:	<i>Canadian Geotechnical Journal</i>
Manuscript ID	cgj-2017-0155.R1
Manuscript Type:	Article
Date Submitted by the Author:	28-Aug-2017
Complete List of Authors:	Jang, Young-Eun; University of Science and Technology, Geospace engineering Han, Jin-Tae; Korea Institute of Civil Engineering and Building Technology, Geotechnical Engineering
Keyword:	Micropile, Waveform micropile, Jet grouting micropile, Field load test, Shaft resistance

Field study on the axial bearing capacity and load transfer characteristic of waveform micropile

Young-Eun Jang^a, Jin-Tae Han^{b,*}

Manuscript submitted to *Canadian Geotechnical Journal*, March 13th 2017

Revised manuscript submitted to *Canadian Geotechnical Journal*, August 28th 2017

Updated manuscript submitted to *Canadian Geotechnical Journal*, September 13, 2017

^a *School of Geospace Engineering, University of Science & Technology, 283 Goyangdae-ro, Ilsanseo-gu, Goyang 10223, Republic of Korea. E-mail: yejang@kict.re.kr*

^b *Korea Institute of Civil Engineering and Building Technology, 283 Goyangdae-ro, Ilsanseo-gu, Goyang 10223, Republic of Korea. E-mail: jimmyhan@kict.re.kr*

*Corresponding author

Phone: +82-31-910-0259, Fax : +82-31-910-0561

Field study on the axial bearing capacity and load transfer characteristic of waveform micropile

Young-Eun Jang, Jin-Tae Han

Abstract

The waveform micropile is an advanced construction method that combines the concept of the conventional micropile with the jet grouting method. This new form of micropile was developed to improve shaft resistance, and has enabled a higher bearing capacity and greater cost efficiency. Two field tests were conducted to examine the field applicability and to verify the effects of bearing capacity enhancement. In particular, waveform micropile construction using the jet grouting method was performed to evaluate the viability of waveform micropile installation. After testing, the surrounding ground was excavated to check the shape of the waveform micropile. The result showed that waveform micropiles can be installed by adjusting the grouting time and pressure. In the loading tests, the bearing capacity of the waveform micropile increased by 1.4 to 2.3 times that of the conventional micropile, depending on the shape of the micropile. The load transfer analysis using the strain gauge data showed that the waveform micropile increases the shaft resistance in the soil layer. This not only decreases the pile settlement but also contributes to the increase of overall bearing capacity. The overall results clearly demonstrated that the waveform micropile is an enhanced construction method that can improve bearing capacity.

Keywords: Micropile, Waveform micropile, Jet grouting micropile, Field load test, Shaft resistance

Introduction

Since micropiles were first used in Italy in the 1950s, they have been widely used not only for the foundation of new buildings but also for the reinforcement of existing structures and the restraint of settlement. The construction of micropiles involves installing reinforcement in a bore hole with a diameter that varies from a minimum of 100 mm to a maximum of 300 mm, and then injecting grout into the hole. Reinforcement such as a steel rebar and a hollow bar with high strength transmits the upper load to the grout body and supports the load through the bond strength between the grout body and the ground. Generally, when calculating the design capacity of a micropile, only the skin friction of the pile is taken into account, and the effect of the end bearing capacity is not included due to the small diameter of the pile. In practice, the Federal Highway Administration (FHWA) guidelines for micropile design and construction suggest the range of bond strength (α_{bond}) values at the pile-to-ground interface as the main factor for calculating the design capacity considering the grouting method (FHWA 2005). Several researchers have carried out various studies to derive high bond strength by modifying the construction method and the structure compositions of the pile to improve the bearing performance of the micropile compared to the conventional pile (Gomez et al. 2008; Bennet and Hothem 2010; Drbe and El Naggar 2014; Abd Elaziz and El Naggar 2015; Vickars and Clemence 2000; El Sharnouby and El Naggar 2012; Papadopoulou et al. 2014).

In Korea, the need for the construction of new buildings, the reinforcement of aging structures, and seismic retrofitting is highly increasing. Recently, the government launched a

new project to construct artificial ground (i.e. reinforced concrete deck) on the upper part of a railway and lagoon in order to relieve the density of the buildings in urban areas. The micropile was expected to satisfy the technical requirements for construction of artificial ground. However, since the construction cost of micropiles using steel rebar has increased compared to other foundation types, several studies have been conducted to improve the bearing capacity and economic efficiency of micropiles through the development of a new micropile method (Choi and Cho 2010; Kim et al. 2016; Kyung et al. 2017)

The waveform micropile was developed in order to provide additional shaft resistance by forming the grout into a waveform shape by jet grouting method. The bearing capacity enhancement by the waveform micropile has been verified through 2D numerical analysis and model tests (Jang and Han 2014; Jang and Han 2015). However, since the behavior of the waveform micropile was not evaluated in the in-situ condition, the constructability and performance of the waveform micropile from a field test were investigated in this study. The goal of the study is to evaluate the constructability of the waveform micropile in an actual soil condition, and analyze the axial behavior of the waveform micropile based on the load-settlement curves and strain gauge data that were measured from the two sets of full-scale loading tests.

Waveform micropile

The waveform micropile method was developed to increase both the bearing capacity and economic efficiency of the conventional micropile by securing the additional shaft resistance even in the upper soil layer. This method thus differs from the conventional micropile method, which only considers the bond strength of the lower ground where the pile socket length is

placed (Jang et al. 2014). In the design of the micropile, the shaft resistance of the casing penetration depth in the soil layer is generally considered to be negligible, but only the bond strength of the supporting layer is included as a design factor. On the other hand, the waveform micropile by a simplified construction method enables drilling and grouting work without the need to install a casing in the soil layer. The waveform micropile was developed to improve the shaft resistance at the contact surface between the grout and soil by partially enlarging the grout to a waveform shape through the pressurized jet grouting method, which is used not only as a pile foundation but also as ground reinforcement and waterproofing, tunnel reinforcement etc. (Bell 1993; Covil and Skinner 1994).

The jet grouting method can be applied to a wide range of soil types such as sandy and gravelly soil with the exception of very hard cohesive soil and rock (Peplow et al. 1999).

Figs. 1(a) and 1(b) show the general configuration of conventional micropile and the conceptualized drawing of waveform micropile, respectively. The casing of the conventional micropile is substituted by waveform grout composed of shear keys as shown in Fig. 1(b). The aim is to maximize the bearing capacity in the soil layer. It has been reported that the vertical bearing capacity and pull-out resistance are increased in a partially under-reamed pile, similar to a waveform micropile (Martin and DeStephen 1983; Cai et al. 2006; Qian et al. 2015). When the diameter of the micropile is D , the diameter of the waveform micropile is divided by the diameter D_1 of the shear key and the diameter D_2 of the pile shaft without the shear key. Here, the shaft diameter D_2 is 300 mm, which does not exceed the maximum diameter range of the general micropile, and the diameter of shear key D_1 is 500 mm, about 1.7 times larger than the shaft diameter. Previous studies related to under-reamed piles showed a significant increase in bearing capacity when the ratio of the diameter of the pile shaft to the bulb was 1.5 ~ 2.0 (Zhang

and Yangin 1992; Peter et al. 2006). The shape of the waveform can be determined from the length of the shear key (L) and the spacing of the shear key (S). The shape of the shear key is expected to be a key factor in determining the load bearing characteristics of the waveform micropile.

As shown in Fig. 2, the waveform micropile can be constructed through (a) ground drilling using a water jet, (b) waveform grout formation by controlling grout injection pressure and time, and (c) reinforcement insertion process. The jet grouting work is a significant part of the installation of the waveform micropile, as the primary purpose of grouting is to strengthen the soil layer and shaft resistance by making shear keys on the pile shaft.

Test conditions

Two sets of field tests were carried out in this study. The purpose of the 1st test was to confirm the constructability of the waveform micropile from the installation test. A full-scale loading test was also performed to compare the compressive and pull-out resistances on the conventional micropile, two types of waveform micropiles, and a jet grouting micropile, where the pile shaft has no shear keys, but the same construction process is used as that for the waveform micropile. For the 2nd test, the load-settlement and load transfer mechanism of the waveform micropile were analyzed in more detail based on the strain gauge measurement data.

Site investigation and test piles preparation

Figs. 3(a) and 3(b) show the standard penetration test (SPT)-N values for the 1st test and 2nd test sites, respectively, located north of Gyeonggi province, South Korea. The 1st test site shown in Fig. 3(a) consisted of alluvial soil with some silty sand and gravel from the ground surface to the

depth of 1.8m. A weathered soil layer was observed with a medium to very dense sand and gravel layer extending to a depth of 10 m, followed by weathered rock. In the 2nd test site shown in Fig. 3(b), a soft to medium dense fill layer was observed, with some silty sand and gravel appearing between 0 and 4.5 m. A deposit layer at a depth of from 4.5 to 7.5 m was also present, consisting of gravelly sand. The unexpected large N-value (50/2) measured at a depth of 3.0 m was considered to be due to partially embedded cobblestones. The ground water table was found at an elevation of 1.4m and 4.7 for the 1st and 2nd field test sites, respectively.

Fig. 4 and Table 1 show the type of piles used in the 1st and 2nd field tests. In the 1st test, a conventional micropile (MP-A), a jet grouting micropile (JP-A), and two types of waveform micropile with different shapes (WM-A1, WM-A2) were constructed. MP-A was installed to a depth of 9 m and placed on a hard layer with an N value of 50 or more. The casing of MP-A was removed after the installation to compare the shaft resistance in the soil layer with that of waveform micropiles. The jet grouting micropile and waveform micropiles were installed at 6 m to the top of the weathered soil layer, so as to have a similar bearing capacity to the conventional micropile. The shape of the waveform micropiles in the 1st test was determined to be the same as that of the model piles that showed the highest bearing capacity from the previous laboratory test and numerical analysis results (Jang and Han 2013). Accordingly, the length (L) and spacing (S) for WM-A1 were determined with a ratio of 1:1, where L and S were the same as D_1 (500 mm) for the shear key diameter, while WM-A2 had a ratio of 1:2 since S was two times D_1 as shown in Fig. 4(a).

For the 2nd test, the shape of the waveform micropile (WM-B) was selected to be the same as that for WM-A1 from the 1st test, which showed higher bearing capacity between the two waveform micropiles. The test micropiles for the 2nd test were designed to have the same

length in order to quantitatively compare the behavior of the piles under the same length. The strain gauges were used for the 2nd test piles to evaluate the axial load transfer mechanism. A total of 18 gauges were attached to the surface of the thread bar in pairs at 1 m spacing from the length of pile of 0.5 m to 7.5 m, and additional gauges were attached at 7.9 m to examine the load transfer characteristic at the pile tip.

The design capacity ($P_{G-allowable}$) of each test pile was calculated using eq. (1), as suggested by FHWA (2005).

$$(1) P_{G-allowable} = \frac{\alpha_{bond}}{F.S} \cdot \pi \cdot D_b \cdot L_b$$

where α_{bond} is the grout-to-ground ultimate bond strength, D_b is the diameter of the drill hole, L_b is the bond length, and F.S is the factor of safety. L_b and D_b of each pile used in the calculation are shown in Fig. 1. In consideration of the range of α_{bond} values for silty sand soil, the range of 95 kPa ~ 215 kPa for a conventional micropile constructed with gravity grout (A type) and 120 kPa ~ 360 kPa for a jet grouting micropile constructed with pressurized grout (B type) were applied for each layer. When calculating the design capacity of the conventional micropile, the entire penetration depth was considered as the supporting layer since the pile was installed without the casing. The bearing capacity of the waveform micropile was conservatively calculated in a same manner to that of the jet grouting micropile since the design method considering the shape effect of the wave-formed grout has not yet been established. Therefore, it was assumed that the waveform micropile with multiple shear keys behaved the same as the jet grouting micropile under the same design load. Table 1 shows the design capacity of each test pile calculated using a safety factor of 2.0. For the 1st test piles, the design capacity of JP-A,

WM-A1, WM-A2 was estimated at 520.3 kN at a length of 6 m, which is considered to be similar to that of the MP-A with a pile length of 9 m. Therefore, the conventional micropile and the other two piles (JP-A, WM-A1, and WM-A2) were constructed to a depth of 9 m and 6 m, respectively, to compare the support performance at different pile lengths. On the other hand, the 2nd test piles (MP-B, JP-B, and WM-B) were all installed at a length of 8 m in order to examine the effect of the increase in the bearing capacity provided by the shear key and load transfer mechanism at the same length. The bearing capacities of the MP-B and JP-B (WM-B) piles were 320.6 kN and 634.7 kN, respectively.

The ultimate structural strength of the test piles were calculated to examine the structural stability using eq. (2) and eq. (3), respectively, as specified by FHWA (2005).

$$(2) P_{ult_compression} = [0.85f_{c-grout}' \cdot A_{grout} + f_{y-casing} \cdot A_{casing} + f_{y-bar} \cdot A_{bar}]$$

$$(3) P_{ult_tension} = [f_{y-casing} \cdot A_{casing} + f_{y-bar} \cdot A_{bar}]$$

where $f_{c-grout}$, $f_{y-casing}$, and f_{y-bar} are the strength of the grout, casing, and steel rebar, respectively, and A_{grout} , A_{casing} , and A_{bar} indicate the area of the grout, casing, and steel rebar, respectively. The diameter of the deformed bars used as a reinforcement in the 1st and 2nd tests was 50 mm ($f_{y-bar} = 500$ MPa) and 63.5 mm ($f_{y-bar} = 555$ MPa), respectively. A 10 MPa grout strength ($f_{c-grout}$) was used, while the casing was not used in the test. The calculation results corresponding to all types of testing piles are summarized in Table 1.

Waveform micropile installation

Fig. 5(a) shows the instrumented steel rebar with strain gauges (TML-WFLA type) for the test piles MP-B, JP-B, and WM-B. The gauge surface was double coated by using two different types of butyl rubber tape and butyl waterproof tape to be protected from the damage during the pile installation and loading test. Then, a spacer was installed at every gauge location in order to prevent damage from the bore hole during the insertion of the rebar.

Figs. 5(b), 5(c), 5(d), and 5(e) show the construction process of the waveform micropile of the ground boring, the forming of the waveform grout via jet grouting method, and the steel rebar insertion, respectively. Immediately after the jet grouting, the steel rebar with the strain gauges was inserted into the grout-filled holes when the grout was still fresh.

The construction test of the waveform micropile was carried out successfully and the construction time was reduced by about 30% compared to the conventional micropile. This difference in construction time occurred because a waveform micropile can be constructed without the casing installation and rock drilling steps that are generally required in the construction of the conventional micropile.

Evaluation of constructability of the waveform micropile

To ensure the successful installation of the waveform micropile, the jet grouting pressure and time step of the rod ascent capable of forming diameters corresponding to D_1 (500 mm) and D_2 (300 mm) were investigated. Among the various fluid systems of the jet grouting method, the double tube type drilling system was used in this test. Generally, when the jet grouting is performed using a double tube method at a grouting pressure of 20 to 40 MPa, the possible diameter in sand ($N < 40$) is about 1,000 to 2,000 mm (YBM, 2010). Since the time step of the rod

ascent is 6 to 24 min/m, it is expected that shortening the ascent time of the grouting rod could form a waveform grout. Therefore, a preliminary test was performed to find the operation time for the shear key formation of the waveform micropile in the 1st test site. In the preliminary test, the grout injection pressure was fixed at 40 MPa and the grouting time was varied. After the construction, the ground was excavated to confirm the formation of the waveform grout as shown in Fig. 6. Although the obtained shape of the waveform grout did not perfectly matched the idealized drawing shown in Fig. 1(b), it was found that the waveform grout can be formed by controlling grouting time for this ground condition.

Pile load test

After installation of the waveform micropile and the other test piles, the grout was cured for 28 days and the load test was carried out as shown in Table 2. The four reaction piles having the same condition as that for MP-A were placed for each test pile. The test piles were spaced at more than 4 times the maximum diameter of 500 mm of the waveform micropile to prevent the interaction between the piles during the load test, as shown in Fig. 7.

Prior to the loading test, the signal of each strain gauge was checked and it was confirmed that some gauges of the JP-B and WM-B were damaged during the pile installation. All gauges that were connected to a digital data logger showed that the resistance values of the strain gauges in the data logger are balanced to $0 \pm 3 \mu\text{s}$.

For the 1st test, three compression (MP-A, WM-A1, WM-A2) loading tests, and a tensile (JP-A, WM-A1, WM-A2) loading test were performed. For the 2nd test, three compression (MP-B, JP-B, WM-B) loading tests were performed. The cyclic loading test method was adopted by a series of loading and unloading to the maximum test load in accordance with the American

Society for Testing and Materials test standards ASTM D 1143-81 (ASTM, 1994). The test was conducted with loading cycles equal to 20%, 40%, 60%, 80%, and 100% of the maximum load corresponding to twice the design load. In each cycle, the load was applied in increments of 100 kN every 20 minutes, and the load was removed in decrements equal to the loading increments. After maintaining the load cycle for about an hour, the next cycle was reapplied.

Field test result and analysis

Analysis of load-settlement relationship

Table 3 summarizes the results of ultimate bearing capacity obtained from FHWA's ultimate capacity equation and loading tests. Fig. 8 shows the load-settlement curves of the 1st test piles obtained from the compression loading test. The final loadings of MP-A, WM-A1, and WM-A2 were 1,225 kN, 882 kN, and 980 kN, respectively, and the corresponding displacements were 63.4 mm, 29.1 mm, and 24.5 mm, respectively. In the graph, the capacity of the waveform micropiles, WM-A1 and WM-A2, was superior to that of the conventional micropile, MP-A in terms of the settlement. However, the waveform micropiles, WM-A1 and WM-A2, experienced rapid increase in displacement, resulting in discontinuation of the load test at loads between about 900 kN and 1,000 kN. It was found that the part of the reinforcement connected to the loading system was buckled as the load reached the yield strength of 982 kN of the rebar. Therefore, it was not possible to evaluate the ultimate load of each pile and the settlement of the pile was compared with the initial load section of about 882 kN before the buckling occurred. The settlement of MP-A, WM-A1, and WM-A2 at the load of 882 kN was 23.3 mm, 10.4 mm and 17.7 mm, respectively, indicating that the settlement of the waveform micropiles was

reduced by 55.3% (WM-A1) and 24.0% (WM-A2) compared to the conventional micropile. From this result, it can be assumed that if the steel rebar buckling did not occur, the bearing capacity of the waveform micropile could increase significantly compared to the conventional micropile.

In the tensile load test on the jet grouting micropile (JP-A) and the two waveform micropiles (WM-A1, WM-A2), the pull-out resistance of the waveform micropiles increased compared to JP-A, as shown in Fig. 9. The allowable bearing capacity of each pile with the safety factor of 2.0 was 490 kN for JP-A and WM-A1, and 573.5 kN for WM-A2, respectively, which were estimated from the log P-log S curves in Fig. 10. The bearing capacity of JP-A and WM-A1 was the identical, but the displacement of WM-A1 at yielding state was 14.13 mm, which was 32% lower than the displacement of JP-A at 20.77 mm. In addition, the bearing capacity of the WM-A2 pile increased by 17% compared to that of the JP-A pile. However, the capacity improvement of the waveform micropile compared to jet grouting pile was less than compressive test. It is because adequate resistance has not been provided by the upper shear key during the tensile load test. Even though the pull-out resistance of the waveform micropile was mainly governed by the upper part, the effect of the upper shear key was not developed enough because of low confining pressure and disturbance during construction.

However, it was difficult to directly estimate the exact bearing capacity including end bearing resistance from the compression test of the waveform micropiles due to the buckling of the steel rebar during the 1st test. Therefore, an additional field test was performed in order to evaluate the effect of increasing the bearing capacity by the waveform micropile. The 2nd field test was conducted on the conventional micropile (MP-B), jet grouting micropile (JP-B), and waveform micropile (WM-B). In the second field test, a concrete cap was installed on the head

of the pile as shown in Fig. 11, to prevent the steel rebar from buckling and the load was transferred through the load plate placed on the pile cap.

The maximum load and displacement for MP-B, JP-B, and WM-B were 882 kN and 31.0 mm, 1,372 kN and 33.9 mm, and 1,862 kN and 35.1 mm at the final loading step of the 2nd load test, respectively. Fig. 12 shows the load-settlement responses of micropiles, showing that the waveform micropile has a higher bearing capacity than the conventional micropile and the jet grouting micropile. In particular, the bearing capacity of the waveform micropile increased compared to that of the jet grouting micropile. As explained above, the construction process of the jet grouting micropile is the identical with that of the waveform micropile, though with the omission of a shear key. Therefore, it is evident that the additional shaft resistance was generated by the shape of the waveform grout.

In order to examine the behavior of each pile in detail, the yield bearing capacity was determined based on the log P-log S curve method, as shown in Fig. 13. The allowable bearing capacity of each pile with the safety factor of 2.0 was 392 kN, 620 kN, and 882 kN for the MP-B, JP-B, and WM-B piles, respectively. From this result, the allowable bearing capacity of JP-B and WM-B increased by 38.5% and 55.5% compared to MP-B, respectively. These results indicate that the waveform micropiles can contribute to the increase of the bearing capacity compared with the conventional micropile, and consequently lead to economic efficiency by reducing the length of the pile.

The design capacity calculated in the above section for the pile design was 321 kN for MP-B, and 635 kN for JP-B and WM-B while the experimentally estimated allowable bearing capacity was 392 kN for MP-B, 620 kN for JP-B, and 882 kN for WM-B. These values calculated using the method from FHWA (2005) and estimated from the load test were in

reasonable agreement with those for both the conventional micropile and jet grouting micropile. However, the FHWA (2005) method appears to have under-predicted the allowable bearing capacity by about 27.7% for the waveform micropile, which was assumed to be equal to the design capacity of the jet grouting micropile. This is because the design method proposed by FHWA (2005) does not reflect the bearing effect of the bearing capacity increase by shear keys. Therefore, an appropriate method to reasonably predict the design load of the waveform micropile needs to be developed.

Analysis of axial load transfer mechanism

In the 2nd test, the axial load transfer mechanism of the waveform micropile was analyzed using the strain gauge measurement results. The axial force, P , at different depths was calculated using eq. (4).

$$(4) P = \varepsilon \cdot A_p \cdot E_p$$

Where ε is the measured strain, A_p is the cross-sectional area of the pile and E_p is the elastic modulus of the pile.

The exact elastic modulus of grout of the JP-B and WM-B piles constructed with the jet grouting method was difficult to estimate due to the mixture of grout slurry and in situ soil. Accordingly, the elastic modulus of jet grouting was back-calculated from the relationship between the measured strains ε near the pile heads and the average stress on the grout section σ_c using eq. (5) (Han and Ye 2006).

$$(5) \sigma_c = \frac{P_p - P_{rb}}{A_c} = \frac{P_p - E_{rb} \cdot \varepsilon \cdot A_{rb}}{A_p - A_{rb}}$$

where P_p is the load applied on the pile head, P_{rb} is the load applied on the steel rebar, A_p is the area of the pile cross section, A_c is the area of the grout portion, A_{rb} is the cross section area of the steel rebar, and E_{rb} is the Young's modulus of the steel rebar. Here, the strains from measured data of JP-A were used, and the Young's modulus of steel rebar was 200 GPa. The grout elastic modulus of 3.04 GPa was obtained from the relationship between the average stress on the grout section and the measured strain as shown in Fig. 14. It was found that the elastic modulus of the grout estimated in this study was in a reasonable range compared to the previous studies, in which the elastic modulus of the jet grouting columns was defined as from 1.8 GPa to 8 GPa (Axtell and Stark 2008; Hamidi et al. 2010; Tinoco et al. 2014).

Fig. 15 shows the axial load transfer of the 2nd test piles with loading steps from the initial load of 98 kN to 784 kN, where the failure occurred at MP-B. In this analysis, only the gauge measurement data near the shear key was used to focus on the analysis of the increase in shaft resistance at the shear key zone of the waveform micropile. Each location of strain gauge is divided into four sections. The JP-B pile shows only the result up to the depth of 3.5 m because the strain gauges were damaged. In the early stage of the loading step shown in Fig. 15(a), the MP-B, JP-B, and WM-B piles showed similar behaviors of the distribution for axial forces. As the load was increased, as shown in Figs. 15(b) to 15(f), the distribution of axial forces along the piles started to present different behaviors.

For MP-B and WM-B, it can be seen that a relatively large axial force appeared for the MP-B compared to WM-B over all of the loading steps. Even though a fairly small load was applied as shown in the graphs in Figs. 15(b) and 15(c), the shaft resistance of WM-B was higher

than that of MP-B in the soil layer corresponding to sections A and B. Although the MP-B pile used in this test was a full-bonded pile without casing installation in the soil layer, a smaller shaft resistance was mobilized compared to the WM-B. These results indicate that the increased shaft resistance at the soil layer of the waveform micropile can contribute to the decrease of the pile settlement, which consequently leads to an increase of bearing capacity.

The increase in the resistance of the WM-B can be confirmed by comparison of the axial forces with the JP-B pile. In the previous load-settlement curves, the bearing capacity of the WM-B pile increased compared to that of the JP-B pile due to the additional shaft resistance mobilized by shear keys. It was also confirmed through the load transfer curve that the shaft resistance of WM-B is larger than that of JP-B. In particular, the shaft resistance of WM-B in the relatively soft soil increased, and the result indicates that the shear key added to the WM-B pile strengthened the shaft resistance in the soil layer. In addition, the WM-B pile presented a relatively large frictional resistance from the early stage of loading compared to MP-B and JP-B. This phenomenon occurs for two reasons: the shape of the waveform micropile as mentioned above and the densification of the ground due to the large compressive stress transmitted to the surrounding ground during the pressurized grouting for the shear key construction. In other words, WM-B contributed to mobilize a large shaft resistance, even in the soil layer at section A with the lowest confining stress.

In order to examine the tip resistance of the waveform micropile, the axial load distribution curves are given, corresponding to each loading step from 94 kN to 1,764 kN of failure load for WM-B as shown in Fig. 16. When compared with the conventional micropile, the base of the waveform micropile was extended to acquire additional end bearing along with an increase in shaft resistance. However, as shown in Fig. 16, the end bearing marked 134.3kN,

which is only 7.6% of 1,764 kN corresponding to the failure load of the WM-B pile. This result indicated that the WM-B pile supports most of the load from the shaft resistance and transfers less load to the pile base.

Fig. 17 shows the shaft resistance of the test piles normalized to the conventional micropile. Normalized curves for MP-B, JP-B, and WM-B were obtained of each section for the failure load of a conventional micropile. Section A shows the weak soil layer near the ground surface, and section D shows the pile tip laid on hard weathered soil. In section A, the shaft resistance of WM-B increased more than two times that of MP-B at a load of 588 kN. This tendency is similar to that already seen in the above axial load distribution graph. On the other hand, the shaft resistance of WM-B was very small in section B and C of a greater depth. In section D, which shows the pile tip, the load transfer to the tip was initiated when the load of 294 kN was applied, but the supporting portion in section A was still significant. This result means that the WM-B pile supports the load through the shaft resistance occurring at the ground surface, even when the MP-B pile reaches the ultimate state at the load of 784 kN. Also, it is confirmed that the waveform micropile increases the shaft resistance in the soil layer, and contributes to not only a decrease of pile displacement but also an increase of bearing capacity.

Fig. 18 shows the shaft resistance of the waveform micropile carried by each section. Here, the final loading step of 1,764 kN indicates the failure load of WM-B. In the early stage of loading, section A showed the highest shaft resistance. After 1,176 kN was loaded, the load ratio shared by section A decreased and the load began to be transmitted to a deeper zone. Also, the slope of section A sharply decreased, then gradually increased again. It is considered that the strength of the grout diminished because the upper shear key was partially broken at the load of 1,176 kN. Even though the original strength of the upper shear key was compromised, the shaft

resistance still occurred in the subsequent loading steps. At the same time, the load distribution in the deeper sections B, C, and D increased. Also, section D, which was close to the pile tip, showed the most dominant shaft resistance from the load of 1,568 kN. These results indicate that the shaft resistance of the shear key gradually developed from the upper part to the lower part according to the settlement of pile. Furthermore, it is expected that the required bearing capacity could be satisfied by simply installing one or two shear keys at a specific depth without installing multiple shear keys along the pile. Therefore, in further study, the relationship between the confining pressure and the load carrying characteristic of a shear key location will be analyzed using centrifuge model test results.

Conclusions

In this study, field construction and full-scale load tests were carried out to evaluate the constructability and load carrying behavior of the newly developed waveform micropile. The pile installation was successfully performed following the steps of boring a hole, grouting, and inserting a steel rebar. It was also shown that the construction time could be reduced by about 30% with the use of a waveform micropile compared with the conventional micropile. After construction, the ground was excavated to verify that the grouting work was accomplished as intended in this study, and it was concluded that the waveform micropile could be made by controlling the grouting pressure and time during the jet grouting work.

The 1st series of full-scale loading tests were conducted to investigate the load-settlement response of the waveform micropile under compression and tensile load. In the compression test, the waveform micropile displayed strong behavior even though it was impossible to evaluate the ultimate bearing capacity of test piles due to the buckling of the steel rebar. The pull-out

resistance of the waveform micropile was also increased by up to 17% compared to the jet grouting micropile, and it was found that the waveform micropile contributed to a decrease of the pile settlement and enhanced the load carrying behavior. From the result, it was confirmed that the bearing capacity of the pile increased when shear keys were added to piles with the same shaft diameter. The allowable bearing capacity was estimated from the 2nd test on the conventional micropile, jet grouting micropile, and waveform micropile. The allowable bearing capacity of the waveform micropile was 56% greater than that of the conventional micropile. Compared to the jet grouting micropile, the allowable bearing capacity increased by up to 28% due to the shear key effect. It was also concluded that an appropriate design method for the waveform micropile needs to be developed since the FHWA (2005) method under-predicts its bearing capacity.

In the axial load distribution analysis, a higher shaft resistance for the waveform micropile was observed from a relatively early stage of loading compared to the other micropiles. In addition, shaft resistance was analyzed at different shear key locations, the results of which showed that the shaft resistance of the waveform micropile increased by more than two times that of the conventional micropile, even in the relatively weak soil layer. The tip resistance of the waveform micropile was marked at 134.3kN, which is only 7.6% of the failure load. From this result, it was found that the waveform micropile supports most of the load from the shaft resistance and transfers less load to the pile base. The shaft resistance of the shear key gradually developed from the upper part to the lower part of the pile according to the settlement of the pile. Also, when the load reached 88.8% of the failure load, the load sharing ratio was superior at the lowest shear key. In conclusion, it was confirmed that the waveform micropile increases the shaft

resistance in the soil layer. This not only decreases the pile settlement but also contributes to the increase of overall bearing capacity.

However, since this study was carried out in limited ground and pile shape conditions, it is necessary to analyze the behavior of the waveform micropile through model test and numerical analysis considering various soil and pile configurations. In addition, as the conventional method underestimated the bearing capacity of the waveform micopile, detailed research should be performed to develop more reasonable design methods.

Acknowledgements

This research was supported by a grant from “Design and construction of artificial ground with the vertical spaces above existing urban infrastructure” which is funded by The Korea Institute of Civil Engineering and Building Technology.

References

- Abd Elaziz, A. Y., and El Naggar, M. H. 2015. Performance of hollow bar micropiles under monotonic and cyclic lateral loads. *Journal of Geotechnical and Geoenvironmental Engineering*, 141(5): 04015010.
- ASTM. 1994. Standard test method for piles under static axial compressive load, ASTM D1143-81. In *Annual book of ASTM Standards*, West Conshohocken, Pa, Reapproved 1994.
- Axtell, P. J., and Stark, T. D. 2008. Increase in shear modulus by soil mix and jet grout methods. *DFI Journal-The Journal of the Deep Foundations Institute*, 2(1): 11-21.

- Bennett, J. K., and Hothem, N. L. 2010. Hollow bar micropiles for settlement control in soft clay. In Proceedings of the 10th International Workshop on Micropiles, Washington DC, USA, 22-25 September 2010.
- Bell, A. L. 1993. Jet grouting-ground improvement. Ed. Moseley, E.D., Chapman & Hall, Glasgow, pp. 149-174.
- Cai, L., Li, Y., and Zhou, H. 2006. Discussion on the bearing capacity of a bored pile with reamed enlargements. *Foundation Analysis and Design: Innovative Methods*, pp. 137-144, ASCE.
- Choi, C., and Cho, S. D. 2010. Field verification study for micropile load capacity. In Proceedings of the 10th International Workshop on Micropiles, Washington DC, USA, 22-25 September 2010.
- Covil, C. S., and Skinner, A. E. 1994. Jet grouting – a review of some of the operating parameters that form the basis of the jet grouting process. In *Proceedings of Grouting in the Ground*, Ed. Bell, A. L., Thomas Telford, Lon.
- Drbe, O. F. E., and El Naggar, M. H. 2014. Axial monotonic and cyclic compression behaviour of hollow-bar micropiles. *Canadian Geotechnical Journal*, 52(4): 426-441.
- El Sharnouby, M. M., and El Naggar, M. H. 2012. Field investigation of axial monotonic and cyclic performance of reinforced helical pulldown micropiles. *Canadian Geotechnical Journal*, 49(5): 560-573.
- FHWA. (2005). *Micropile Design and Construction: Reference Manual*, Federal Highway Administration (FHWA) U.S. Department of Transportation, Washington, D.C. Publication No. FHWA-NHI-05-039.

- Gómez, J.E., Rodriguez, C.J., Mikitka, J., and Robinson, H.D. 2008. Hollow core bar micropiles - design parameters interpreted From 404 Load Tests. In Proceedings of the 33rd Annual Conference on Deep Foundations, New York, USA, October 2008.
- Hamidi, B., Krzeminski, M., Berthier, D., Vincent, P., and Yates, M. 2010. The application of jet grouting for the construction of sydney international airport runway end safety area. *Australian Geomechanics*, 45(4): 21.
- Han, J., and Ye, S. L. 2006. A field study on the behavior of micropiles in clay under compression or tension. *Canadian Geotechnical Journal*, 43(1): 19-29.
- Jang, Y. E., and Han, J. T. 2014. Development on the Micropile for Applying to Artificial Ground above Railroad Site. *Advanced Science and Technology Letters*, 55: 43-46.
- Jang, Y. E. Han, J. T., and Choi, C. 2014. Development on the micropile for applying to artificial ground above railroad site. In Proceedings of the 12th International Workshop on Micropiles, Krakow, Poland, 11-14 June 2014.
- Jang, Y.E., and Han, J.T. 2015. Study of load capacity of waveform micropile by centrifuge test, In Proceedings of the 25th International Offshore and Polar Engineering Conference. Hawaii, USA. 21-26, 2015.
- Kim, S. H., Choi, J., Lee, J. M., Lee, J., and Jung, C. Y. 2016. Improved load carrying behavior of perfobond rib-installed partially expanded micropiles, *Geotechnical Testing Journal*, 39(4): 633-647.
- Kyung, D., Kim, D., Kim, G., and Lee, J. 2017. Vertical load-carrying behavior and design models for micropiles considering foundation configuration conditions, *Canadian Geotechnical Journal*, 54(2): 234-247.

- Martin, R. E., and DeStephen, R. A. 1983. Large diameter double underreamed drilled shafts, *Journal of Geotechnical Engineering*, 109(8): 1082-1098.
- Papadopoulou, K., Saroglou, H., and Papadopoulos, V. 2014. Finite element analyses and experimental investigation of helical micropiles, *Geotechnical and Geological Engineering*, 32(4): 949-963.
- Peplow, A. T., Jones, C. J. C., and Petyt, M. 1999. Surface vibration propagation over a layered elastic half-space with an inclusion, *Applied acoustics*, 56(4): 283-296.
- Peter, J. A., Lakshmanan, N., and Devadas Manoharan, P. 2006. Investigations on the static behavior of self-compacting concrete under-reamed piles. *Journal of materials in civil engineering*, 18(3): 408-414.
- Qian, Y., Wang, J., and Wang, R. 2015. The analysis of the vertical uplift bearing capacity of single CEP pile, *Open Civil Engineering Journal*, 9: 598-601.
- Tinoco, J., Correia, A. G., and Cortez, P. 2014. A novel approach to predicting Young's modulus of jet grouting
- Vickars, R.A., and Clemence, S.P. 2000. Performance of helical piles with grouted shafts, In *New Technological and Design Developments in Deep Foundations*, In Proceedings of Sessions of Geo-Denver 2000, American Society of Civil Engineers, Reston, Va. 2000.
- YBM. 2010. Jet grouting equipment (catalog), Saga, Japan.
- Zhang, J., and Yangin, X. 1992. Reinforced concrete load-bearing pile with multi-branches and enlarged footings, and means and method for forming the pile, US Patent No. 5,122,013, Patent and Trademark Office, Washington D.C., 1992.

LIST OF FIGURES

Fig. 1. Schematics of micropiles: (a) Conventional micropile, (b) Waveform micropile.

Fig. 2. Construction process of waveform micropile: (a) Boring, (b) Jet grouting, (c) Reinforcing, (d) Completion.

Fig. 3. SPT-N values: (a) 1st test site, (b) 2nd test site.

Fig. 4. Micropile types: (a) 1st test, (b) 2nd test.

Fig. 5. Field installation procedure of waveform micropile: (a) Instrumented steel rebars, (b) Boring, (c) Jet grouting, (d) Inserting a steel rebar, (e) Completion of construction.

Fig. 6. Side view of waveform grouting column through excavation.

Fig. 7. Layout of test micropiles in the test area.

Fig. 8. Load-settlement curve for MP-A, WM-A1, and WM-A2 (1st test).

Fig. 9. Tensile load-vertical displacement curves for JP-A, WM-A1, and WM-A2 (1st test).

Fig. 10. Estimation of yield load using log P-log S curve method for MP-A, WM-A1, and WM-A2 (1st test).

Fig. 11. Concrete capping at head of steel rebar: (a) Before reinforcing, (b) After reinforcing.

Fig. 12. Load-settlement curve for MP-B, JP-B, and WM-B (2nd test).

Fig. 13. Estimation of yield load using log P-log S curve method for MP-B, JP-B, and WM-B (2nd test).

Fig. 14. Back-calculated elastic modulus of micropiles installed with jet grouting method.

Fig. 15. Axial load transfer curves along pile depth at corresponding loads: (a) 98 kN, (b) 196 kN, (c) 294 kN, (d) 392 kN, (e) 588 kN, (f) 784 kN.

Fig. 16. Axial load distribution of waveform micropile at different loading steps (2nd test).

Fig. 17. Normalized shaft resistance for MP-P, JP-B, and WM-B (2nd test).

Fig. 18. Comparison of shaft resistance at different depths of waveform micropile.

Draft

Fig. 1. Schematics of micropiles: (a) Conventional micropile, (b) Waveform micropile.

Fig. 2. Construction process of waveform micropile: (a) Boring, (b) Jet grouting, (c) Reinforcing, (d) Completion.

Fig. 3. SPT-N values: (a) 1st test site, (b) 2nd test site.

Fig. 4. Micropile types: (a) 1st test, (b) 2nd test.

Fig. 5. Field installation procedure of waveform micropile: (a) Instrumented steel rebars, (b) Boring, (c) Jet grouting, (d) Inserting a steel rebar, (e) Completion of construction

Note:

---- Wave-formed shape

Fig. 6. Side view of waveform grouting column through excavation.

Fig. 7. Layout of test micropiles in the test area.

Fig. 8. Load-settlement curve for MP-A, WM-A1, and WM-A2 (1st test).

Fig. 9. Tensile load-vertical displacement curves for JP-A, WM-A1, and WM-A2 (1st test).

Fig. 10. Estimation of yield load using log P-log S curve method for MP-A, WM-A1, and WM-A2 (1st test).

Fig. 11. Concrete capping at head of steel rebar: (a) Before reinforcing, (b) After reinforcing.

Fig. 12. Load-settlement curve for MP-B, JP-B, and WM-B (2nd test).

Fig. 13. Estimation of yield load using log P-log S curve method for MP-B, JP-B, and WM-B (2nd test).

Fig. 14. Back-calculated elastic modulus of micropiles installed with jet grouting method.

Fig. 15. Axial load transfer curves along pile depth at corresponding loads: (a) 98 kN, (b) 196 kN, (c) 294 kN, (d) 392 kN, (e) 588 kN, (f) 784 kN.

Fig. 16. Axial load distribution of waveform micropile at different loading steps (2nd test).

Fig. 17. Normalized shaft resistance for MP-P, JP-B, and WM-B (2nd test).

Fig. 18. Comparison of shaft resistance at different depths of waveform micropile.

LIST OF TABLES

Table 1. Design bearing capacity values for test micropiles.

Table 2. Load test program.

Table 3. Predicted and measured capacities of test piles.

Draft

Table 1. Design capacity values for test micropiles.

Description	Design capacity (kN)	Ultimate structural strength (kN)	
		Compression	Tension
1 st test	MP-A ¹	540.0	1,600
	JP-A ²	520.3	2,400
	WM-A1 ³	520.3 <	2,400 <
	WM-A2 ⁴		
2 nd test	MP-B ¹	320.6	2,000
	JP-B ²	634.7	2,400
	WM-B ⁵	634.7 <	1,800 <

¹Conventional micropile, ²Jet grouting micropile, ³Waveform micropile(L=D₁, S=D₁)

⁴Waveform micropile(L=D₁, S=2D₁), ⁵Waveform micropile(L=2D₁, S=2D₁)

Table 2. Load test program.

Description	Loading test	
	Compression	Tension
1 st test	MP-A	✓
	JP-A	-
	WM-A1	✓
	WM-A2	✓
2 nd test	MP-B	✓
	JP-B	✓
	WM-B	-

Table 3. Predicted and measured capacities of test piles.

Pile type	Ultimate bearing capacity (kN)					
	1 st test			2 nd test		
	Predicted	Measured		Pile type	Predicted	Measured
		Compression	Tension			
MP-A	540	Test failed (Buckling of steel rebars occurred)	-	MP-B	320.6	392
JP-A	520.3		490	JP-B	634.7	620
WM-A1	520.3 <		490	WM-B	634.7 <	882
WM-A2			573.5			