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Analysis of Shear Load-Transfer Curve of Prebored and Precast Steel Pile

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ABSTRACT: The prebored and precast piles are frequently used in urban areas where the noise and vibration are caused by the pile driving. In this study, the behavior and load-distribution of prebored and precast piles were investigated by pile load tests and an analytical study. Special attention was given to the quantifying skin frictions which developed between the pile-soil interfaces of instrumented 14 test piles. Based on this detailed field tests, the load-settlement curves and axial load distributions of piles were obtained and the load transfer curve (t-z curves) for the steel prebored and precast were proposed. It is found that the piles show two different load transfer behaviors, such as ductile and brittle shapes, and t-z curves according to two shapes are proposed for a hyperbolic- and sawtooth-shape, respectively. It is also shown that the prediction by the proposed load-transfer curve is in good agreement with the general trends observed by in-situ measurements. Therefore, it represents a practical improvement in the prediction of load-settlement of prebored and precast piles.

Keywords: Steel pile, Prebored and precast pile, Field loading test, Shear Load transfer curve, Skin friction

1 INTRODUCTION

The vibration and noise induced during pile installation have become a significant consideration in modern construction projects. For this reason, various construction projects, such as urban construction and highway construction, are using prebored and precast piles (PPP) over conventional driven piles.

A prebored and precast pile is installed by boring a hole in the ground and then placing the precast pretensioned spun high strength concrete (PHC) or steel pile in the borehole and finished by injectingcement around the pile. The preboring process induces significantly less noise and vibration during construction compared to driven piles and it is more cost effective compared to drilled shafts.

Prebored and precast pile was first introduced to Korea in the 1980s. In 1990s, the concept prebored and precast pile is used to designate installation methods such as soil-cement injected precast pile (SIP) method, separation doughnut auger (SDA), special auger SIP (SAIP), reverse circulation (RCD), corex method, and percussion rotary drill (PRD) method. In this study, we used 55.6% SDA method, which is the most used

method among the various pile construction methods.

As shown in Figure 1, the installation process of SDA method is as follows: 1) A borehole for placing the precast pile is bored with casing. 2) 1st cement milk injection is conducted in the borehole. The 1st cement milk is commonly injected through the borehole auger up to four times the pile diameter with an additional 1m in height (4D +1m). The cement milk injected in this step effects the pile tip bearing condition of the prebored and precast pile. 3) The steel pipe pile is placed in the borehole. 4) After the placing of the precast pile, the 2nd cement milk is injected to the top of the borehole. The cement milk injected in this step effects the shaft resistance of prebored and precast pile. 5) The installation process of prebored and precast pile is completed with light hammering on the head of the pile.

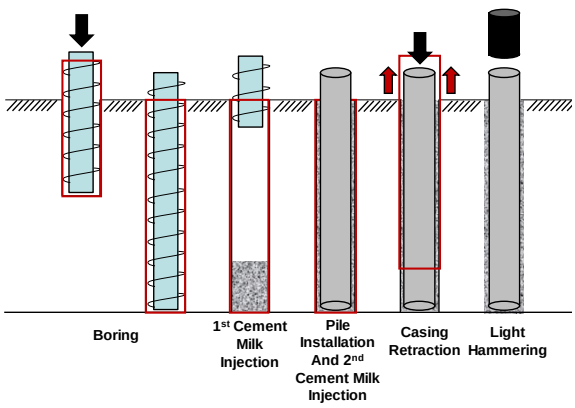


Figure 1. Installation process of prebored and precast piles(Separated Doughnut Auger)

However, the quality of a prebored and precast pile depends highly on the workmanship of the technicians, and it is relatively difficult to predict the behavior and the settlement of the pile due to various uncertainties. Studies and experiments on prebored and precast piles have been limited to only standard penetration test-based empirical methods which roughly estimates the bearing capacity.

The bearing capacity and settlement of a prebored and precast pile are affected not only by the end bearing capacity, but also by the skin friction between the pile and the cement milk poured in the borehole. A number of investigations of the settlement and the interface behavior of various types of pile have been conducted. However, as mentioned above, to date, most of the measurements and experiments were based on driven piles or drilled shafts.

The settlement of a steel pile was measured, and based on the results, the maximum skin friction between the steel pile and dry sand was found to depend on the roughness of the steel pile and the sand particle size(Uesugi et al. 1986, 1989, 1990). Drilled shafts, which have a source of skin friction similar to a prebored and precast pile, showed a relative settlement and high shear stress at the pile-soil interface (O'Neill et al. 1996). Shear load transfer characteristics along the skin of the drilled shaft socketed in rocks were investigated, and modified t-z curves were used to enhance the accuracy of the numerical predictions (Seol et al. 2009, Jeong et al. 2010).

Various studies were conducted to investigate the major factors affecting the skin friction of prebored and precast piles. The unit weight of the soil around the pile was found to be the main influence factor for the skin friction (Lim et al. 2002). Additionally, studies showed that the N-value, the type of soil and the water-cement ratio of the cement milk were the major influence factors affecting the skin friction of prebored and precast piles. Among the major factors, the water-cement ratio of the cement milk was found to have the biggest influence on the skin friction (Hong et al. 2008).

Studies on the soil-structure interface, which greatly affects the behavior of prebored and precast piles, can generally be divided in to three stages (Zhang et al. 2008). The investigation and studies focused on, in stage 1) strength behavior which can be used for practical applications, in stage 2) the stress-strain relationship of the interface along with the significant advancement in the field of measuring techniques and

numerical tools, and in stage 3) systematic investigation regarding the cyclic response, microscopic measurements, various types of interface,improved constitutive models describin g the nonlinear and elasto-plastic behavior and numerous types of numerical methods.

In this study, the load distribution and deformation of prebored and precast piles subjected to an axial load are investigated based on the results of field load tests. From the results of the tests, two different load transfer behaviors, such as ductile and brittle shapes, and t-z curves according to two shapes are proposed for a hyperbolic- and sawtooth-shape, respectively.

2 METHODOLOGY

The definition and the use of the load transfer function is the main factor to the successful understanding to the actual pile-soil response. There has been a considerable amount of studies on the load transfer function

Shear Load Transfer Function

There are several methods available for predicting the shear load transfer (t-z) functions for pile in soils (Kraft et al., 1981; Castelli et al., 1992; Vijayvergiya, 1977) and rocks (Baguelin, 1982; O'Neill and Hassan, 1994; Kim et al., 1999). Studies on the shear load transfer functions for prebored and precast piles have not been thoroughly studied, and assumed to be similar to the drilled shafts in the field design due to the similar shaft condition. All of these methods model the t-z curves similar to the elasto-perfectly plastic models where the ultimate value f_{max} is obtained in the same way as it would be for shaft resistance in bearing capacity estimations.

The t-z relation proposed by Baguelin (1982) is expressed as a bilinear (similar to the elasto-perfectly plastic) function. This model is based on the radial integration of the shear curve obtained from the self-boring pressuremeter curve and was verified by finite-element work

$$f = \frac{E_{SB}}{2r(1+v_S)[1+\ln\left(\frac{L}{2r}\right)]} \quad (1)$$

where, E_{SB} is the initial tangent modulus by a self-boring pressuremeter test, L is the embedded pile length and r is the radius of the pile shaft. Here, E_{SB} correlates favorably with the modulus E_R obtained from an unload-reload cycle in a preboring pressuremeter test. Therefore, this model includes the replacement of E_{SB} by E_R in the recommended slope for the t-z curve. O'Neill and Hassan (1994) suggested the potential t-z behavior in rock and it is shown in Figure 2. If the pile-rock interface is clean so that the cement milk bonds to the surrounding soil, the roughness pattern will be regular, and the asperities are rigid, and the t-z curve will be formed such as OABC. However, in most cases the interface roughness pattern is not regular due to some degree of smear

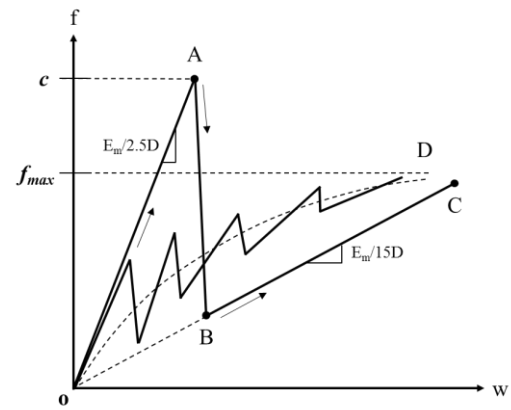


Figure 2. Potential t-z relations for rock

or the infiltration of the injected cement milk due to loss circulation; in addition, asperities are deformable, which results in ductile or brittle-like, progressive failure among asperities. Therefore, they proposed as interim criterion for a hyperbolic, or a sawtooth type, t-z curve in most pile embedded in rocks as below until better solutions become available.

$$f = \frac{w}{\frac{2.5D}{E_m} + \frac{w}{f_{max}}} \quad (2)$$

where, E_m is effective Young's modulus of rock mass. Kim et al. (1999) proposed modified hyperbolic load transfer function for highly weathered rocks based on the numerical analysis and on model-scale tension loading tests performed on nine instrumented test piles. The t-z function of pre-failure response is represented as,

$$f = \frac{w}{\frac{1}{S_i} + \frac{w}{\alpha_i f_{max}}} \quad (3)$$

where, S_i is initial tangent modulus of load transfer function as follows,

$$S_i = \frac{CE_s}{\sqrt{D}} = \frac{C\alpha_1 f_{max}}{\sqrt{D}} \quad (4)$$

where, α_1 is curve-fitting constant larger than 1.0 and C is proportional constant.

In the modified hyperbolic transfer function, f approaches its maximum value α_1 until it reaches its ultimate resistance f_{max}, beyond which it is modeled as a near perfectly plastic behavior. This transfer function is quite similar to the hyperbolic model with the value $\alpha_1 = 1.0$, whereas for α_1 larger than 1.0, the curve shape is similar to elasto-perfectly plastic. Thus, regardless of the surface roughness, the transfer function can be represented by modified hyperbolic transfer function by adjusting the α_1 value properly (rough surface C = 3.86, $\alpha_1 = 1.00$; smooth surface C = 6.26, $\alpha_1 = 1.35$)

These t-z curve models may be reliable if site-specific correlations can be accomplished. Their reliability may be questionable due to the numerous variables which affect the shaft resistance of rock sockets mentioned above. Johnston (1994) warns that t-z curve methods elicit criticism since they do not thoroughly consider failure mechanisms, random surface roughness patterns, rock stiffness and effects of dilation among interfaces on normal stress.

3 FIELD LOADING TEST

3.1 Field load test

To analyze the behavior of a prebored andprecast pile, a field loading test on an actual pile was conducted in the southern region of the Korean peninsula. After a series of field investigations and laboratory tests, the ground condition was shown to reflect the typical soil conditions of the Korean peninsula, consisting of land fill, a sedimentary layer, weathered soil and weathered rock layers.

The diameter of the steel pile used in the loading test was 0.508m, and the thickness was 0.012m. The strain gauges indicating the settlement and the load-transfer curve of the test pile were installed at the surface of the test pile, as shown in Figure 3. The gauges were placed on two sides of the pile for each depth to prevent the loss of records due to gauge failure, with exceptions on the boundary of layers, where the spacing differs to 0.5m.

Static load tests were conducted in compression and based on the ASTM D1143 protocol. A total of 6 earth anchors were installed at the test site to serve as a reaction. A schematic representation of the loading system for the static load test is shown in Figure 4. The instruments used for the static load tests included a load cell, a hydraulic jack, a pump, LVDTs, and beams. The maximum capacity of the load cell and the hydraulic jack was 6000 kN.

The period of the loading was based on four steps, by loading 25%, 50% and 75%, sustaining the load until the settlement of the head was less than 0.25mm per hour (maximum duration of two hours). The loading process was carried up to the ultimate state of the pile. The settlement of the pile head due to axial loading is recorded based on the measurement through the LVDT (Linear Variable Differential Transformer) installed on the test pile.

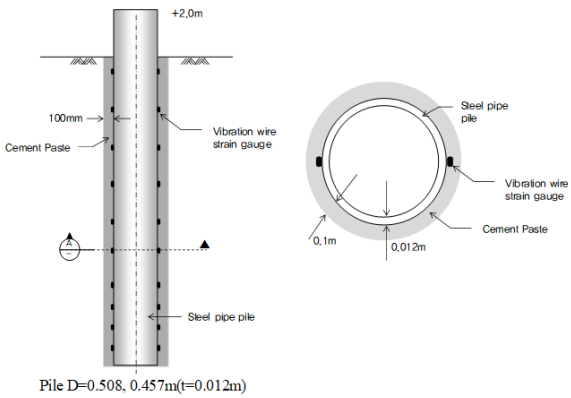


Figure 3. Schematic representation of instrumented piles

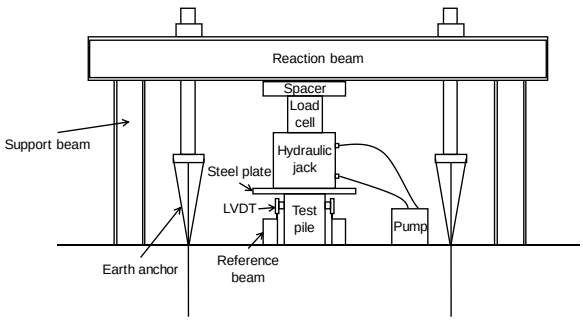


Figure 4. Schematic of loading system for static loading test

3.2 Site investigation

Due to the complexities of the behavior of prebored and precast piles, a comprehensive geotechnical investigation was performed to define the soil profile and properties at the test site as accurately as possible. To identify the subsurface materials, a subsurface investigation was performed on three boreholes (BH-1, BH-2 and BH-3) by using conventional sampling near the test piles. Figure 5 shows an image of the subsurface soil profile with the borehole and embedment for the test piles.

The soil profile near the surface consists of 9.7 m of fill, sedimentary, weathered soil underlain by weathered rock. The results of the SPTs indicate that SPT N values ranging from 5 to 50. The soil properties based on these site investigations are summarized in Table 1. According to the unified Soil Classification System (ASTM 2011), the sand is classified as with round angularity.

Table 1. Physical properties of in-situ soil

Soil	USCS	r_t (kN/m^3)	c (kPa)	φ (deg)	E (MPa)	υ
Fill	SM	17.0	0	29	10	0.3
Sediments	GP	20.0	5	29	40	0.3
Weathered soil	SM	20.0	22	39	127	0.3
Weathered rock	-	21.5	35	32	185	0.3

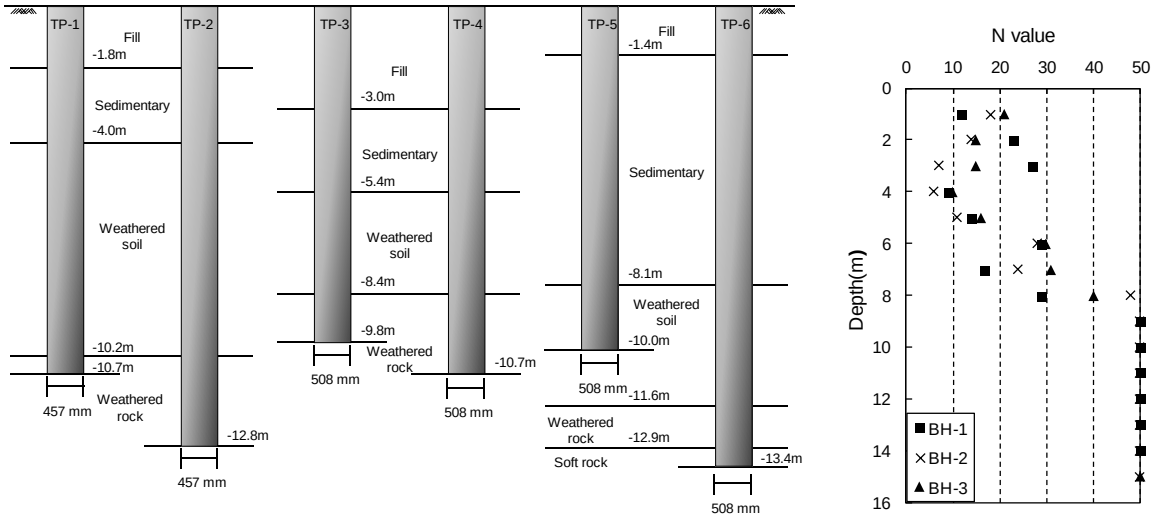
4 TEST RESULTS AND DISCUSSION

4.1 Static load test results

A series of tests was performed in the field (14 cases). Typical test results are presented in Table 2 in terms of the ultimate strength versus settlement. Table 2 can be categorized typically into two different types. Ductile group was selected as a criterion that yield stress occurs at greater than 10mm of settlement. Conversely, the brittle group was selected as the criterion that the yield stress occurs at a settlement of 10mm or less.

4.2 Load transfer mechanism

Through the measurements of the pile strain along the shaft of the test piles due to axial loading. The strain of the pile at each depth, at each loading stage, was converted in to load acting on a certain point by a correlation process of the raw data from the field and multiplying the Young's modulus (E) and area of the test pile (A). From this, the load distribution curves by depth is obtained.



(a) subsurface soil profile (b) N value from SPT

Figure 5. Schematic of loading system for static loading test

Table 2. Summary of field pile loading test

Pile Num.	Type	Ultimate strength (kN)	Final settlement (mm)
Test-1	Ductile	5,150	66
Test-2	Ductile	5,625	126
Test-3	Brittle	2,440	70
Test-4	Brittle	2,625	62
Test-5	Brittle	3,375	60
Test-6	Ductile	4,125	90

4.3 Load distribution curve

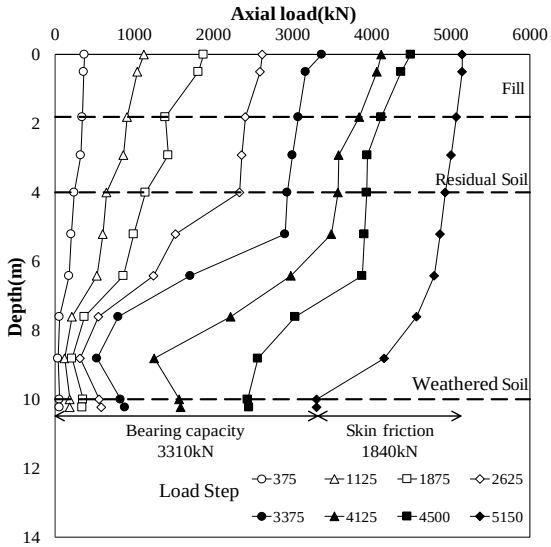
The axial load distribution profiles are obtained by analyzing measured strain gauge data along the pile. Ductile group show that higher bearing capacity and settlement than brittle group. Becasue, the ductile behavior, the load transfer is successfully transferred from the skin surface to the tip. In addition, it was shown that the bearing capacity of the tip was larger than the skin friction, and the load transfer occurs to the tip (Figure 6(a)). However, the brittle behavior, the fracture occurred in the skin surface and the load transfer did not to the tip. Also, the skin friction was larger than the bearing capacity of tip. It is shown that the settlement rapidly after the failure occurred on the skin surface(Figure 6(b)).

4.4 Load transfer curve

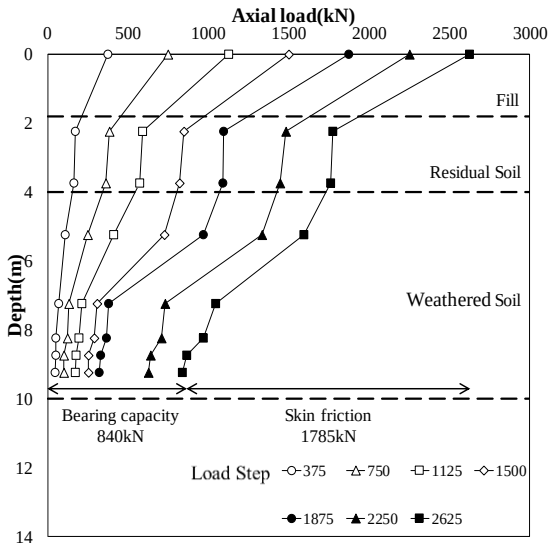
As shown in Figure 7, shear transfer curves can be categorized typically into two different types. Ductile group composed of eleven test datas and brittle groups composed of seven test datas were embedded in weathered soil and weathered rock. the piles show two different load transfer behaviors such as ductile and brittle shapes and corresponding t-z curves are proposed for a hyperbolic- and sawtooth-shape, respectively, produced by soil-cement paste interface.

The condition of the shaft along the test pile was investigated through the borehole image profile system (BIPS). The BIPS investigated the shaft of the pile by drilling a borehole at the side of the test pile and taking images of the wall along the borehole. The image obtained from the BIPS investigation is displayed along the depth of the borehole. The result is shown by the borehole depth, from top left to the bottom right. The BIPS images actually shows that the cement milk injected in the borehole has penetrated into the surrounding ground.

Figure 8 shows the BIPS investigation result of TP-5 test case. The smooth grey portion of the BIPS image is observed widely along the shaft. The cement milk infiltrated portion (light grey) makes up the majority of the shaft below the fill layer (about 2m below the surface). This can be the cause of the excessively high shaft resistance and brittle behavior observed through the t-z curves.

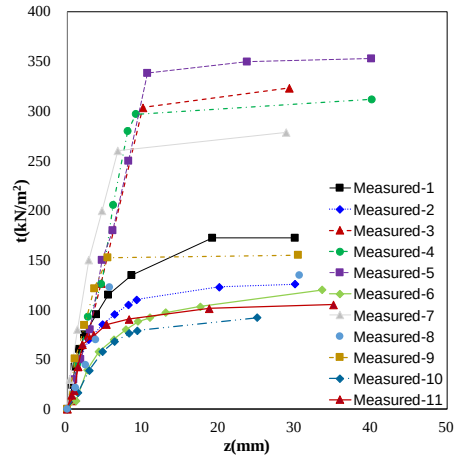


(a) Ductile

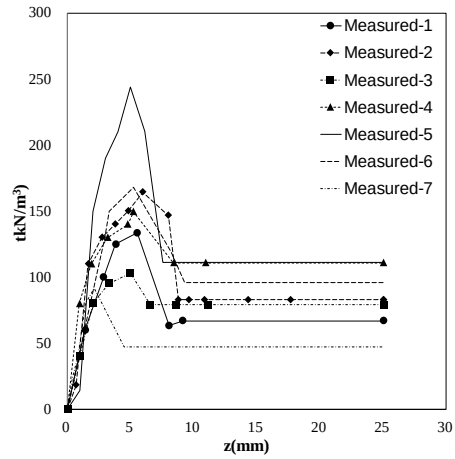


(b) Brittle

Figure 6. Load-distribution curve



(a) Ductile



(b) Brittle

Figure 7. Shear load-transfer curve

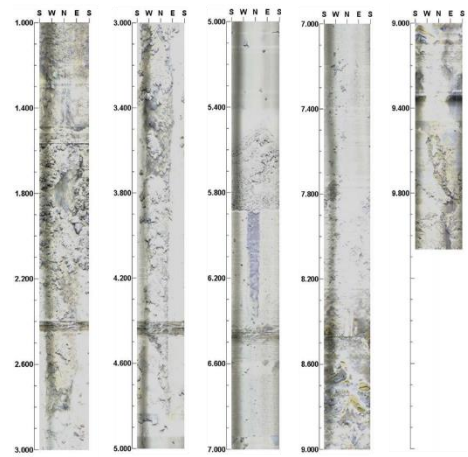


Figure 8. BIPs results with cement milk infiltration

5 CONCLUSIONS

In this study, we conducted a field static load test on prebored and precast steel pile and analyzed the load-settlement and load-transfer curve. As a result, we analysis fo the shear load-transfer curve (t-z curve) of a steel pile. The following conclusions were obtained:

1. The piles show two different load transfer behaviors, such as ductile and brittle shapes, and t-z curves according to two shapes are proposed for a hyperbolic- and sawtooth-shape.

2. The hyperbolic shape form of ductility behavior was found in the general soil with no cement milk injected. The bearing capacity of the tip was larger than the skin friction, and the load transfer occurs to the tip.

3. The behavior of the sawtooth type is dependent on the brittle behavior in the soil - cement layer formed by the water - cementing process by smear. Also, the skin friction was larger than the bearing capacity of tip.

6 ACKNOWLEDGEMENTS

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