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Tien-Kuen Huang
Department of Civil Engineering, National Chung Hsing University, Taichung, Taiwan, R.O.C., huang@dragon.nchu.edu.tw

Chih-Yi Yang
Department of Civil Engineering, National Chung Hsing University, Taichung, Taiwan, R.O.C.

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Acknowledgements

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SIDE FRICTION OF DRILLED PILES IN COBBLE LAYERS

Tien-Kuen Huang[1] and Chih-Yi Yang[2]

Key words: cobble layer, side friction, drilled pile.

ABSTRACT

Full scale vertical load tests have been carried out on drilled piles in cobble layers to evaluate side friction capacity. In the finite element analysis, firstly the property of pile is assumed linear elastic and the elastic-plastic Mohr-Coulomb model is used for the cobble layer. Interface elements are introduced to model interface interaction between pile and cobble layer. The measured data obtained from the field tests are compared with the finite element simulation results using PLAXIS 2D. It shows reasonable agreement between field test data and simulation results. A preliminary evaluation of side friction of drilled piles in cobble layers based on Rollins et al. design equation is also made after assembling the available databases from load tests and numerical analysis. A modified equation is proposed to estimate the ultimate unit side friction of drilled piles in cobble layers for design purpose.

I. INTRODUCTION

Pile foundation is one of the most popular deep foundations adopted for structures in weak soils, as well as in strong soils sustaining heavy loads and moments. The ultimate capacity and maximum settlement of the pile govern the design of vertically loaded piles. These are usually completed by a number of numerical and theoretical approaches. However, the accurate evaluation of the movement and capacity of the piles is particularly difficult due to the complexity of soils. Pile loads are carried by skin friction developed at the side of the pile and point bearing capacity at the tip of the pile simultaneously (Meyerhof, 1976; Tomlinson, 2001). In the load transfer mechanism, the maximum frictional resistance along the pile shaft will be fully mobilized when the relative displacement between the pile and soil is about 5 to 10 mm, while the maximum point bearing will not be mobilized until the pile has moved about 10 to 25% of pile diameter (Van Weele, 1957; O'Neill, 2001; Randolph, 2003). That is, the side friction will take the major part of the pile load under design load for piles (Zhu and Chang, 2002; Comodromos et al., 2009; Said et al., 2009), especially in granular soils. Therefore, the evaluation of side friction is a key issue of the pile design in granular soils. About the study of pile capacity in granular soils, most researches emphasize on the piles installed in the sandy soils (e.g. Reese and O'Neill, 1988; O'Neill and Reese, 1999; O'Neill, 2001; Loukidis and Salgado, 2008; Tosini et al., 2010). A few are related to the side friction of pile shaft in gravelly soils which are classified as gravels and gravelly sands based on Unified Classification System (e.g. Rollins et al., 2005 ; Rabab'ah et al., 2011). Recommended modifications to ultimate side friction of pile shaft are also reported for gravelly soils with different composition of gravels. However, there seems little research about the characteristics of drilled pile in cobble layer which contains higher percentage of particles greater than 7.5 mm or even larger.

On the other hand, the numerical analysis is widely accepted in geotechnical field and finite element calculations. It is also more and more adopted for the design of foundation pile. Although the bearing capacity of a single pile is most usually determined by pile load tests or by empirical correlations, the numerical analysis of a load test on a bored pile may lead to better understanding of the details of pile behavior. This paper presents the results of three field tests carried out on vertically loaded piles embedded in the cobble layer and attempts to study the pile behavior under vertical loads using a two-dimensional finite element model. Emphasis of comparisons between the measured results and the analyzed prediction is on the side friction of pile shaft. A preliminary evaluation of ultimate side friction of drilled piles in cobble layers based on Rollins et al. design equation is also made after assembling the available databases from load tests and numerical analysis.

II. FIELD LOAD TESTS

Three bored cast-in-situ piles studied in this paper are located along the Taichung Metropolis Road No. 74 near the suburb of eastern Taichung. The soils at the site consist of approximately 0m~7m of mixed (gravel, sand, silt, and clay)

Paper submitted 03/02/15; revised 05/24/15; accepted 06/02/15. Author for correspondence: Tien-Kuen Huang (e-mail: huang@dragon.nchu.edu.tw).
[1] *Department of Civil Engineering, National Chung Hsing University, Taichung, Taiwan, R.O.C.*
[2] *Department of Civil Engineering, National Chung Hsing University, Taichung, Taiwan, R.O.C.*

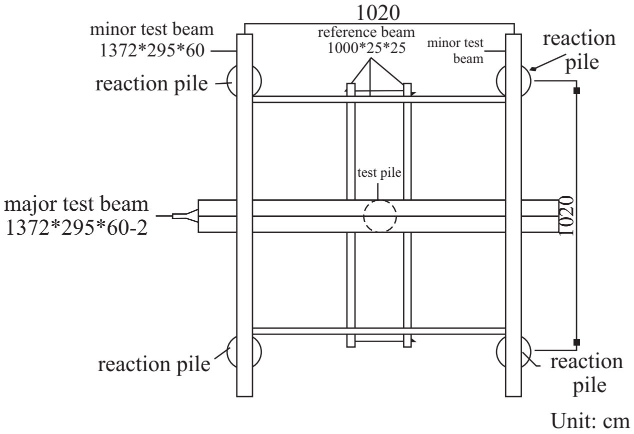

Fig. 1. Layout of pile load test equipment.

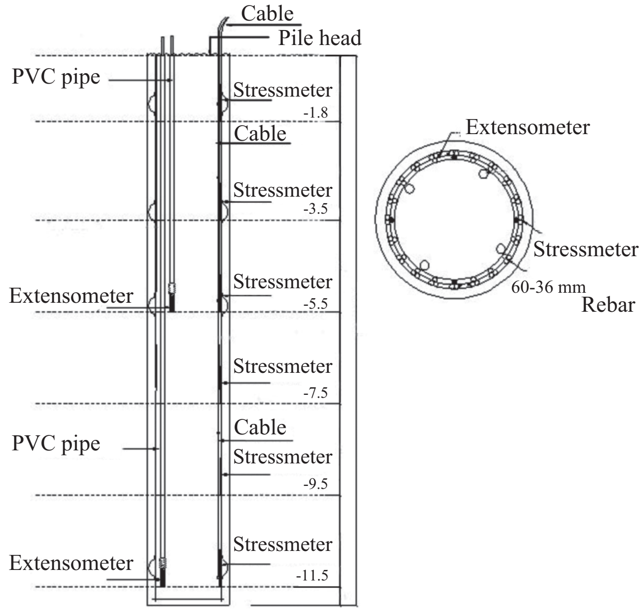

Fig. 2. Arrangement of internal monitoring systems.

deposits overlying a deep deposit of cobble layer. In cobble layer, the percentage of particles greater than 4.75 mm is more than 70%.

The three test piles are all of 1.2 meters in diameter and 12, 15 and 25 meters in length respectively. All the three pile load tests were conducted according to the slowly maintained loading procedure (ASTM D1143, 2007) and maximum loads were 44,100 kN, 39,400 kN and 44,500 kN respectively. None of them was loaded to failure because they were proof testing of working piles. The pile load test arrangement is composed of the test pile and 4 reaction piles. The compressive load was applied by 8 hydraulic jacks having a capacity of 5,886 kN each. The load force was transferred through the test beam to the 4 reaction piles which were placed at a distance of 7.2 m from the test pile (equal to 6-fold diameters of pile). Steel bearing plates were installed between the hydraulic jacks and the pile head to transmit the applied load uniformly to the pile head. All connections are employed by I-beam as shown in Fig. 1, including reaction system, loading system and monitoring system. Hydraulic jack and reaction piles are used as loading system. Monitoring system consists of electronic displacement transducer (LVDT) with automatic signal acquisition data logger. The layout of equipment for pile load test is schematically shown in Fig. 1.

To study axial load transfer mechanism and side friction of piles, a number of rebar stressmeter were attached to the reinforcing bars within the piles at proper depths of the piles. To carry out a test, the load was applied in cumulative equal increment of 4,905 kN except the final stage. Each load increment was maintained for a time interval of not less than one hour and until all displacement had ceased. The arrangement of internal monitoring systems for case 1 is shown in Fig. 2. These instruments are interfacing with automatic IDA data acquisition system. At each increment, the displacement at the top and different depth of pile and rebar stressmeter in the reinforcing bars were recorded and processed in real time. All the tests were continued when the applied load reached the maximum setting load.

1. Load Distribution and Side Friction

According to the data taken from the rebar stressmeters, which are buried inside the test pile, we can analyze the load transfer along the pile depth from the top of pile. The axial load of pile along the depth is derived from the measured data of rebar stressmeter in reinforcing bars with some calculation.

The average unit side friction f_s, in the considered depth is obtained from the load distribution by following equation,

$$fs = (Ptop - Pbottom)/As \tag{1}$$

where f_s = unit shaft resistance (kN/m^2); P_{top} = axial load at the top of considered depth (kN); P_{bottom} = axial load at the bottom of considered depth (kN); A_s = pile surface area (m^2).

2. Load-displacement Curve

Usually, the basic information obtained from the pile load test is the settlement of the pile head under different loading sequence. This can be measured by LVDT installing at the top of pile. According to the measured displacement at each load increment we can build up load-displacement curve.

III. NUMERICAL SIMULATIONS

The mechanical behavior of drilled piles under vertical load applied on the pile head is studied and the finite element package PLAXIS 2D (Brinkgreve et al., 2011), is used to perform the analyses.

1. Model and Properties of Material Sets

From the geotechnical investigation reports (CECI., 2007), the main site soil of field test is composed of about 10 to 30%

Table 1. Parameters of material sets of case 1.

Strata	Depth (m)	Blow counts (N)	γ (kN/m^3)	c (kN/m^2)	ϕ (degree)	E (kN/m^2)	ν	R_{inter}
Overlying soil	0~3.8	> 100	21	12	30	5.5E5	0.25	0.8
Cobble layer	3.8~37	> 100	22.5	15	46	1.0E6	0.3	0.9
Pile	—	—	—	—	—	3.0E7	0.1	—

Table 2. Parameters of material sets of case 2.

Strata	Depth (m)	Blow counts (N)	γ (kN/m^3)	c (kN/m^2)	ϕ (degree)	E (kN/m^2)	ν	R_{inter}
Overlying soil	0~0.875	> 100	21	12	30	5.5E5	0.25	0.8
Cobble layer	0.875~27	> 100	22.5	15	46	1.0E6	0.3	0.9
Pile	—	—	—	—	—	3.0E7	0.1	—

Table 3. Parameters of material sets of case 3.

Strata	Depth (m)	Blow counts (N)	γ (kN/m^3)	c (kN/m^2)	ϕ (degree)	E (kN/m^2)	ν	R_{inter}
Overlying soil	0~2.2	3	21	12	30	5.5E5	0.25	0.8
Cobble layer	2.2~16.5	>100	22.5	12	42	1.0E6	0.3	0.9
Clay	16.5~19.6	18	19	25	20	5.0E4	0.25	0.5
Cobble layer	19.6~34	>100	22.5	15	46	1.0E6	0.3	0.9
Pile	—	—	—	—	—	3.0E7	0.1	—

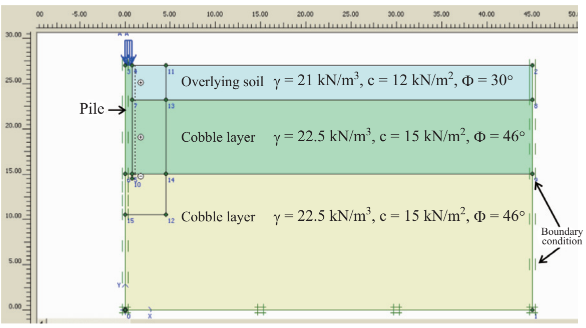

Fig. 3. Soil profile and material properties of case 1.

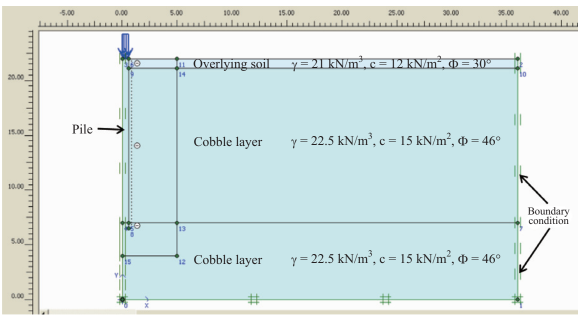

Fig. 4. Soil profile and material properties of case 2.

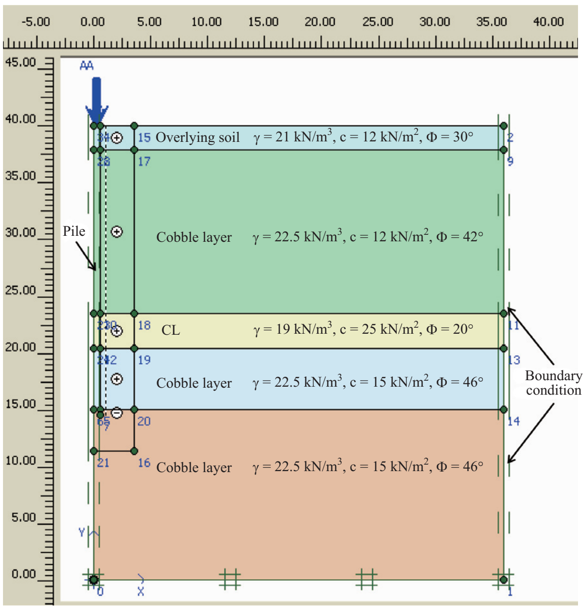

Fig. 5. Soil profile and material properties of case 3.

fine sands and 70 to 90% boulders, cobbles and gravels. The SPT-N value of cobble layer is more than 100. The average density of the site soils γ, is between 20 to 23.5 kN/m^3. There are no experimental data about Young's modulus and strength parameters in the geotechnical investigation reports. They are determined from the test results near the site by some researchers (Chang et al., 1996; Chu et al., 1996). The soil profile and material properties of the three test piles in this paper are summarized in Figs 3-5. The strata are all cobble layers except the top surficial soil and a very thin clay in the third case. Tables 1-3 list the parameters of the material sets for three test sites. The drilled piles under vertical load had been mo- deled as an axisymmetric problem by PLAXIS 2D. 15-node triangular element was chosen which results in a two-dimensional finite element model with two translational degrees of freedom per node. The 15-noded triangle provides a fourth

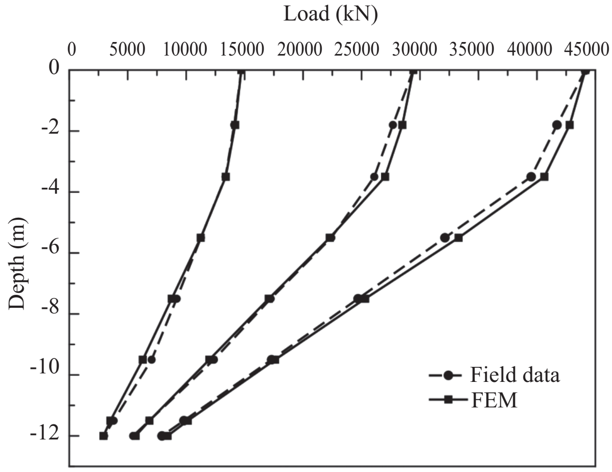

Fig. 6. Load transfer curve of pile for case 1.

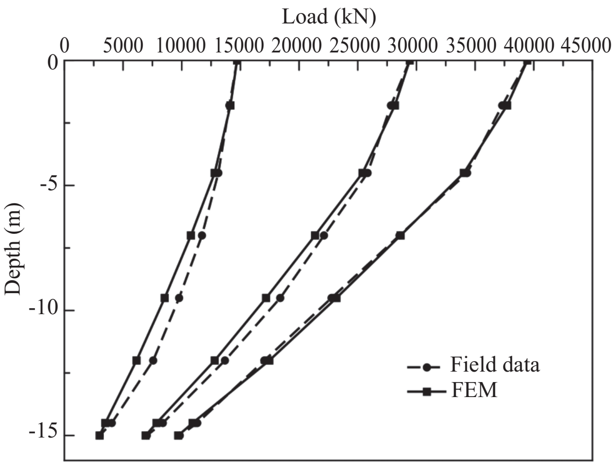

Fig. 7. Load transfer curve of pile for case 2.

order interpolation for displacements and the numerical integration involves 12 Gauss points. The pile is made up of reinforced concrete and the behavior is assumed to be linearelastic. The behavior of cobble layers is described by elasto-plastic constitutive model, in conjunction with Mohr-Coulomb failure criterion.

2. Soil-pile Interface

The material properties of the drilled pile are substantially different from those of the surrounding soil. The contact behavior between pile and surrounding soil is simulated by the interaction of interface element. In PLAXIS 2D, the interface properties are defined with a strength reduction factor R_{inter}, which is used to simulate the strength of the interaction of interface. That is, $c_i = R_{inter}c_{soil}$ and $\tan\phi_i = R_{inter} \tan\phi_{soil}$ where c_i and ϕ_i are the reduced cohesion and friction angle in the interface. If the shear stress in the interface is less than $\sigma_n\tan\phi_i + c_i$ where σ_n is the normal stress in the interface, the interface displays elastic behavior. However, if the shear stress is greater than or equal to $\sigma_n\tan\phi_i + c_i$, the interface exhibits plastic behavior. In

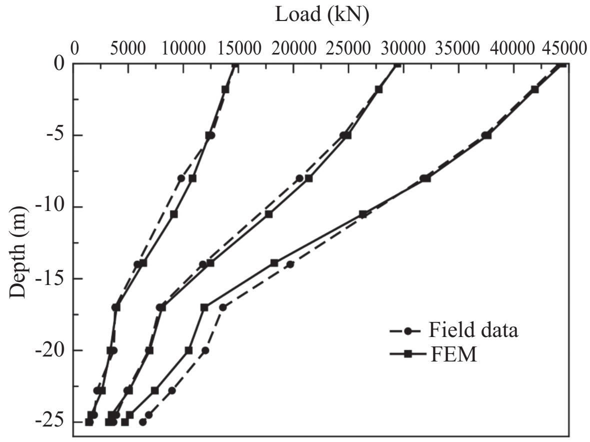

Fig. 8. Load transfer curve of pile for case 3.

this paper, the interface between pile and surrounding cobble layer is relatively rough, so R_{inter} is set to 0.9.

3. Mesh and Boundary Conditions

Axisymmetric model is adopted to analyze the behavior of the drilled pile under vertical load. The x-coordinate is along the radial direction, and the y-coordinate corresponds to the axial line of symmetry. A domain area of 40 times and 18 times the pile diameter in x and y direction coordinates has been selected. In the study, the standard fixities of PLAXIS tool are used to define the boundary conditions. Thus the program will generate full fixities at the bottom and vertical rollers at the vertical sides.

Because the problem is modeled as axisymmetric, only one half (the right half) is taken into account in the analysis. Firstly, set the global coarseness to medium coarse in the mesh option. Then the meshes will be automatically generated by PLAXIS 2D program. For improving the accuracy of the computation, the meshes around the pile of 3 times the pile diameter are refined.

4. Loading Simulation

In simulating the loading of drilled piles with PLAXIS 2D, the staged construction is selected. The type of loading used in the analysis is distributed loads. The input value of every stage is the same as the stage loading value of pile load test except the final stage.

IV. COMPARISON OF MEASURED DATA AND FEM SIMULATION

According to the data taken from the rebar stressmeters, which are buried inside the test pile, we can analyze the load transfer along the pile depth during every load stage. In simulating the loading of drilled piles with PLAXIS 2D, the staged construction is selected. The input load of every stage is the same as that of field test. Figs. 6-8 show the load transfer curves along the pile depth simulated by PLAXIS and calculated from

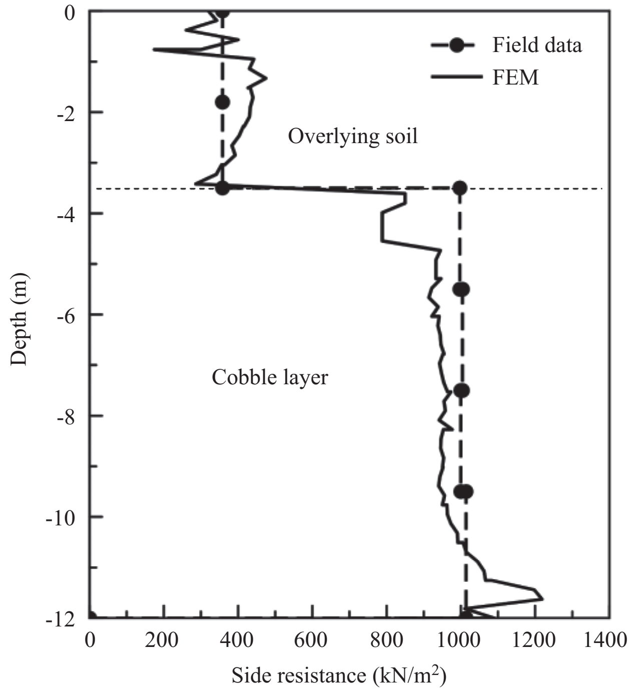

Fig. 9. Side friction along pile depth for case 1.

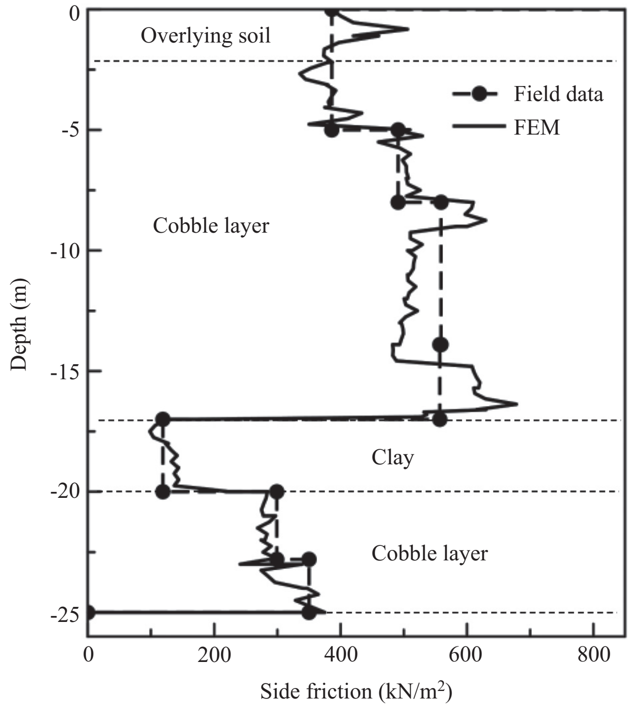

Fig. 11. Side friction along pile depth for case 3.

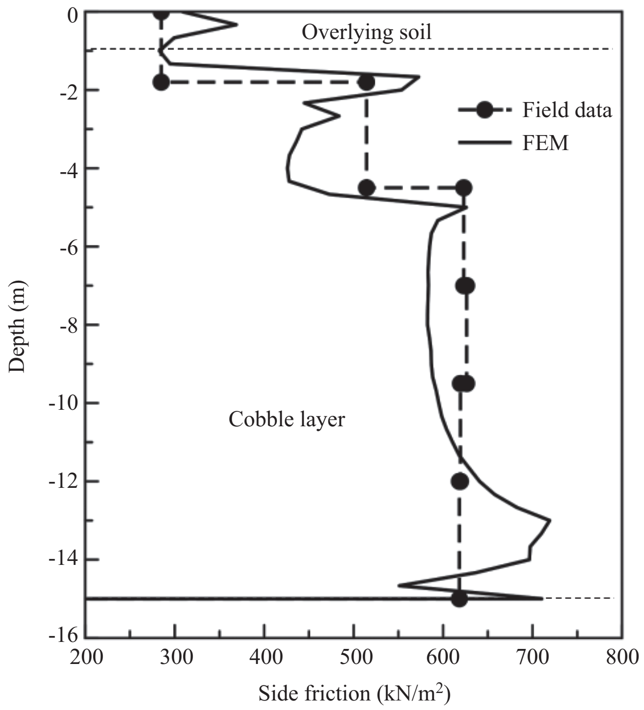

Fig. 10. Side friction along pile depth for case 2.

test data for different applied loads. There is a good compatibility between the results of FEM simulation and pile load test. Figs. 9-11 show the side friction results simulated by PLAXIS and calculated from pile load test data. There still exists a proper coincidence. From Figs. 9-11, it can be found that the lower side friction is in clay layer, and a reduced side friction is in the deep cobble layer. The latter may be attributed to the mild mobilization of side friction at greater depth for case 3. As for the settlement of the pile head under different loading sequence, Figs. 12-14 show the load-displacement curve of pile. It seems to be reasonable agreement between the FEM simulation and pile load test. As for the proportions of shaft resistance and base resistance with respect to the total resistance, Figs. 15-17 show the results of finite element analyses and calculated loading test. It can be seen that the proportions of side friction and base resistance gradually approach to a stable value. The proportions of side friction for the three cases are about 82%, 74%, 86% respectively.

V. ULTIMATE CAPACITIES OF DRILLED PILES

Many methods have been reported and developed to predict the ultimate capacities of pile by pile load test. All of these methods depend on the shape of the load-displacement curve. Owing to the conditions of it's use and applicability, the methods developed by Van der Veen (1953) and Chin (1970) were adopted in this study. Van der Veen's method supposed that load versus displacement of pile is an exponential curve; that is, $P = P_{ult}(1-e^{-\alpha\delta})$, where P is load at any loading level; P_{ult} is ultimate capacity of pile; α is the factor related to load and displacement; and δ is settlement corresponding to load P. The ultimate capacity P_{ult} can be evaluated through the expression of $\ln(1-P/P_{ult})$ vs. displacement of pile δ for different assumed values of P_{ult}. When the relation curve becomes a straight line,

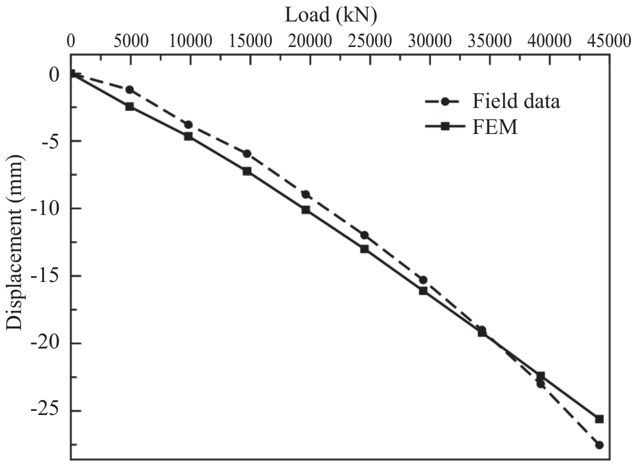

Fig. 12. Load-displacement curve of pile for case 1.

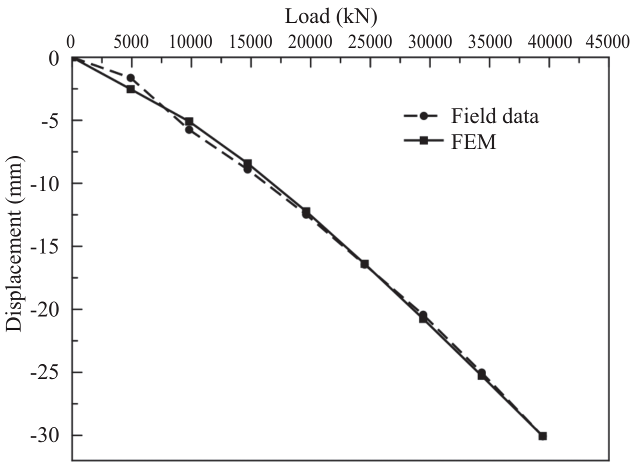

Fig. 13. Load-displacement curve of pile for case 2.

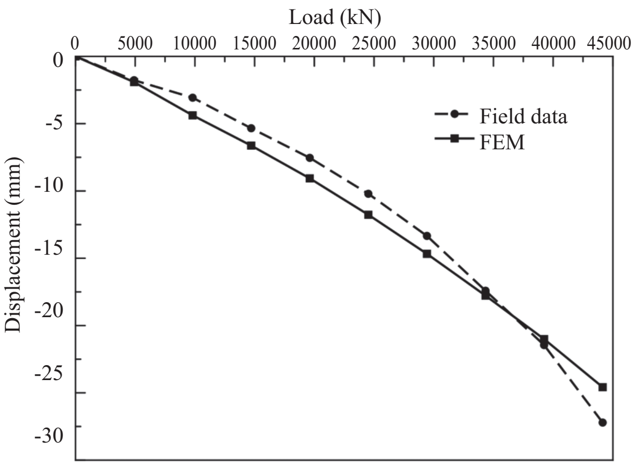

Fig. 14. Load-displacement curve of pile for case 3.

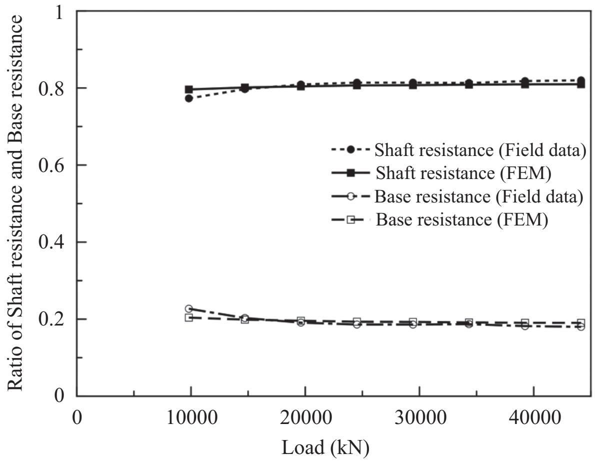

Fig. 15. Ratio of pile resistances for case 1.

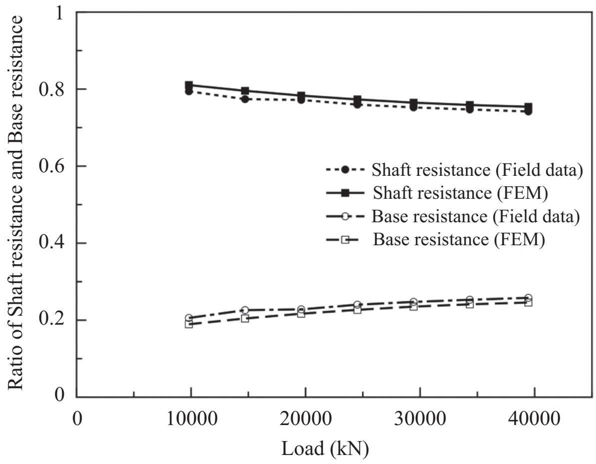

Fig. 16. Ratio of pile resistances for case 2.

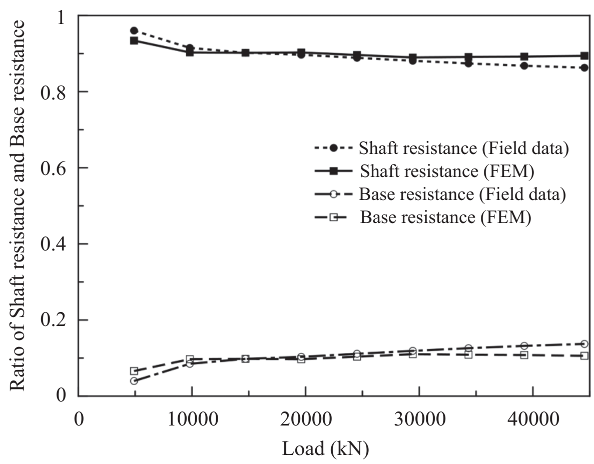

Fig. 17. Ratio of pile resistances for case 3.

the corresponding value of P_{ult} is the ultimate capacity. Fig. 18 shows the searching process of ultimate capacity for pile case 1 using Van der Veen's method. Chin's method supposed that load versus displacement of pile is a hyperbolic curve. Thus the inverse slope of δ/P vs. P will be the ultimate capacity P_{ult}. Fig. 19 shows the relation of δ/P vs. P and evaluation of P_{ult} for

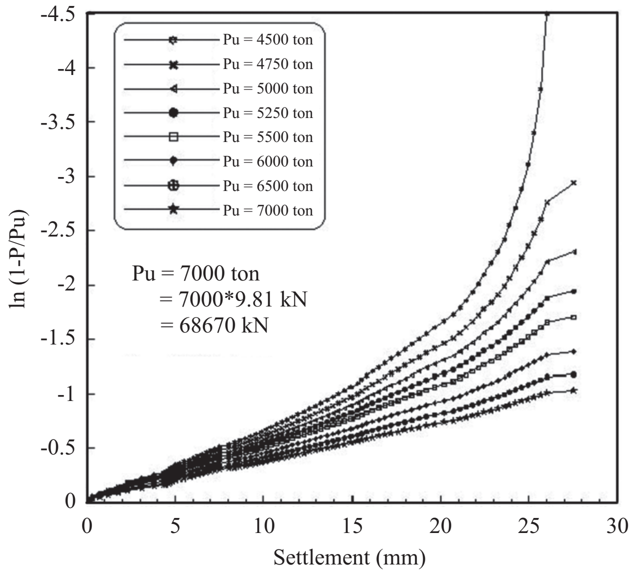

Fig. 18. Ultimate capacities estimated by Van der Veen method (Case 1).

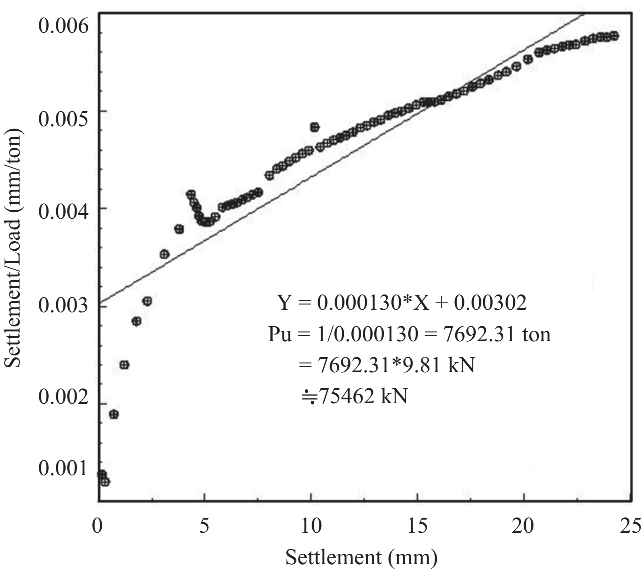

Fig. 19. Ultimate capacities estimated by Chin method (Case 1).

pile case 1 using Chin's method. The ultimate capacities for other two cases are evaluated in the same manner. All the ultimate capacities of three cases using these two methods are summarized in Table 4.

VI. DESIGN EQUATIONS FOR SIDE FRICTION

The ultimate side friction capacity Qs can be written as

$$Qs = \Sigma\, p\, \Delta L\, f \quad (2)$$

where p is the parameter of the pile section, ΔL is the incremental pile length over which p and f are taken to be constant, and f is ultimate unit side friction of pile.

Table 4 Summary of ultimate capacities of drilled pile test for three cases.

Interpretation methods	Case 1 (kN)	Case 2 (kN)	Case 3 (kN)
Chin method	75,462	67,562	80,939
Van der Veen method	68,670	63,765	73,575

It is convenient in pile design to express skin resistance as the product of the vertical effective stress and an appropriate coefficient of β in the granular soils. The procedure proposed by Reese and O'Neill's method (1988) for sands gives

$$f = \beta\sigma_z' \leq 200 \text{ kPa} \quad (3)$$

where σ_z' is the vertical effective stress in soil at depth z; β = 1.5-0.245 $z^{0.5}$ with limits of 0.25 ≦ β ≦ 1.2; and z is the depth below ground surface (m).

Rollins et al. (2005) have modified the above equation for gravelly sands as follows:
for sands with 25 to 50% gravel,

$$f = \beta\sigma_z' \text{ kPa} \quad (4)$$

where β = 2.0-0.15 $z^{0.75}$ with limits of 0.25 ≦ β ≦ 1.8,
for sands with more than 50% gravel,

$$\beta = 3.4\, e^{(-0.085z)} \text{ with limits of } 0.25 \leqq \beta \leqq 3.0.$$

where e is natural base (2.718).

VII. PRELIMINARY EVALUATION OF ULTIMATE SIDE FRICTION OF DRILLED PILES IN COBBLE LAYERS

As indicated previously, Reese and O'Neill equation and Rollins et al. equation are used to compute ultimate side friction in sands and in granular soils with different composition of gravels respectively. Table 5 summarizes the ultimate side friction using the above design equations and the pile load test in cobble layers. The ultimate side friction of piles in cobble layers is obtained from the pile ultimate capacity using Van der Veen's method in Table 4 with the ratios of side friction to the pile capacity as 82%, 74%, 86% respectively for the three cases. The ultimate side frictions of test piles in cobble layers are significantly higher than those of the Reese and O'Neill and Rollins et al. equations. That is, the ultimate side frictions in cobble layers will be underestimated by using the Reese and O'Neill equation or Rollins et al. equation. These design equations seem not be appropriate in the evaluation of ultimate side friction for soil layers with high percentages of gravel and cobbles.

From the evaluated ultimate side friction of field test data, Fig. 20 shows the back-calculated β values of the three pile load tests as a function of depth. The Reese and O'Neill design

Table 5. Summary of ultimate side friction for three cases.

	Length (m)	Diameter (m)	Average SPT-N	Reese and O'Neill (kN)	Rollins et al. (> 50% gravel) (kN)	Pile test (Van der Veen) (kN)
Case 1	12	1.2	> 100	5,415	11,497	56,309
Case 2	15	1.2	> 100	5,712	11,400	47,186
Case 3	25	1.2	> 100	7,916	13,944	63,275

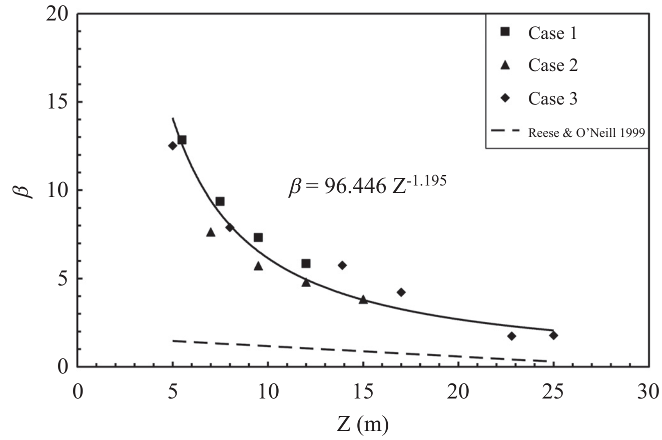

Fig. 20. Back-calculated β values from pile load tests along with existing design curves and proposed curve for cobble layers.

curve is also included in Fig. 20 for comparison. It should be noted that only the cobble layer is considered in the analysis. Back-calculation of β values using regression techniques with an exponentiation fitting as Rollins et al. procedure, the function of β with depth in the evaluation of ultimate side friction of piles in cobble layers can be modified as

$$\beta = 96.446\, Z^{-1.195} \tag{5}$$

VIII. CONCLUSION

In this paper, three full scale vertical load tests are performed on drilled piles in cobble layers. Finite element program PLAXIS 2D is used to analyze the field load test. More study is emphasized on the side friction of piles. Two methods to predict the ultimate capacities of pile by pile load test are used for studying the pile capacity in this paper. A very high ultimate capacity of drilled piles in cobble layers is found comparing with that of pile commonly in granular soils. An approximate 80% of side friction resistance to the full capacity of drilled piles sustains the applied vertical load in cobble layers. Presently, there seems no appropriate design equation to evaluate the ultimate side friction of drilled piles in cobble layers (SPT-N > 100). Based on the results of field tests and finite element analysis, the main conclusions are summarized as below:

(1) A good comparison of load transfer and unit side friction along the drilled pile, and load-displacement of drilled piles in cobble layers is obtained through the field test and numerical simulation.
(2) The ultimate capacity of drilled piles in cobble layers is found much higher than that of pile commonly in granular soils.
(3) The results of pile load test and numerical simulation show that skin friction of drilled piles in the cobble layers takes most part of the total axial load. The ratio of skin friction to the total load capacity is approximately in the range of 74%~86%.
(4) A preliminary equation to evaluate the ultimate side friction of drilled piles in cobble layers is proposed for design purpose in this paper.

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