NUMERICAL PREDICTIONS OF THE LOAD-DISPLACEMENT CURVES OF ROCK-SOCKETED PILES

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ABSTRACT

Since the settlement limit concept is generally adopted as design criteria of rock-socketed pile foundations, the load-displacement(σ - δ) behavior of the rock-socketed piles should be well understood at the design stage, which, however, is hard to achieve due to its complexity. To help this out, field pile load tests are executed on cast-in-situ concrete piles, first, to figure out the σ - δ behavior of rock-socketed piles. Next, the σ - δ relations of the piles are simulated numerically using commercial package program (FLAC) varying a couple of input data which are sensitive in shaping the σ - δ curves. Finally, the relations between the best input data for the numerical simulations and the geotechnical field data are cultivated to generalize the numerical simulation procedures, which enables geotechnical engineers to predict the σ - δ behavior at the design stage, if appropriate geotechnical field data are provided.

KEYWORDS

Rock-socketed pile, Load-displacement curve, Pile load test, Interface element, Peak cohesion, Subgrade reaction modulus

INTRODUCTION

It is known that the allowable bearing capacity of rock-socketed piles is governed exclusively by the settlement, and not by strength, unless the strength of the intact rock is extremely low. Thus, to be able to predict the capacity of the rock-socketed piles, the load displacement(σ - δ) behavior of the piles should be well understood.

To this end, a series of field load tests are executed on cast-in-situ concrete piles to obtain the σ - δ curves, which are, then, simulated by numerical manipulation. In the manipulation, input data of a couple of geotechnical constants are varied to find out the best values to simulate the field curves. After obtaining a set of the best input data, the relationships between the input data and geotechnical properties of base rocks are cultivated so that the field σ - δ curves could be numerically predicted, if the field geotechnical properties are given.

PILE LOAD TESTS

The base rock of the test site is Gneiss which is one of the most popular types of rocks in Korea. 8 of 40cm cast-in-situ piles are constructed on the site with 2 anchor piles for each test pile. Plan view of the test pile arrangement as well as the location of boreholes is shown in Fig.1, and the boring logs, in Fig.2.

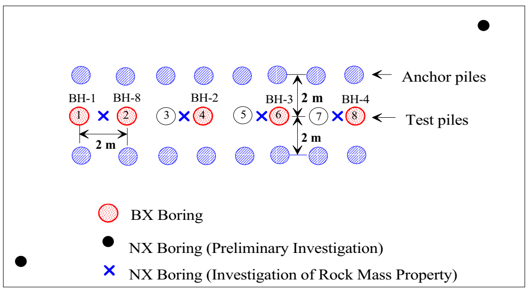

Fig. 1 Plan view of test piles

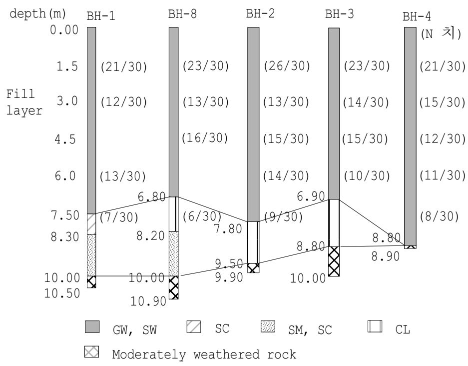

Fig. 2 Soil profile

Pile No.1 is intended to penetrate the base rock by one pile diameter and, to eliminate the point resistance, styrofoams of 10cm thickness are stacked at the bottom. Rock penetration depthes of Piles No.2～No.7 vary from 0.5～2.7 times of the pile diameter as listed in Table 1(Pile No.3 is eliminated in the table because it is constructed for different purpose). Pile No.8 is constructed through soil layers only, and styrofoam of 10cm thickness are stacked at the bottom. Pile load tests are executed following ASTM D 1143-81. The whole test results are shown in Fig.3.

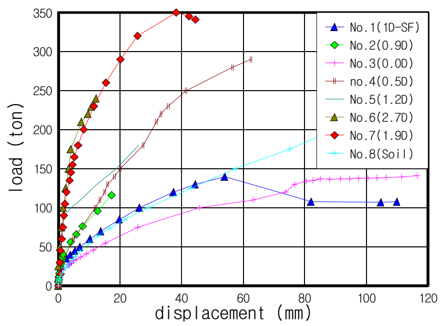

Fig. 3 Pile load test results

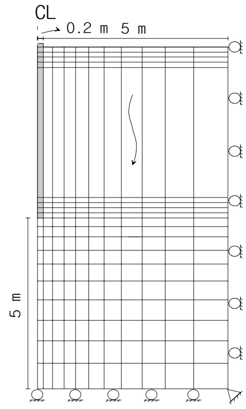

Fig. 4 Finite difference mesh

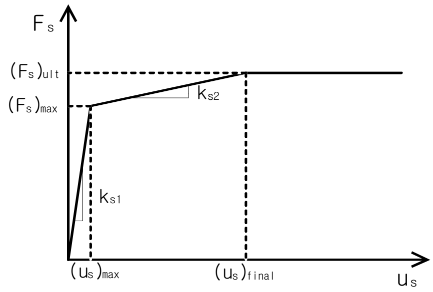

Fig. 5 Shear force versus displacement

NUMERICAL MANIPULATION

FLAC

Package program FLAC has been utilized to reproduce the field vertical load-displacement curves by numerical manipulation. In the manipulation, elastic piles are modeled axisymmetrically, and surrounding soils are simulated by elasto-plastic constitutive models obeying Mohr-Coulomb's failure criteria. Interface spring elements are inserted at the pile-soil boundaries. The finite difference grid used in the numerical simulation is shown in Fig.4.

INTERFACE ELEMENTS

To simulate the slip of piles along the pile-soil boundaries, interface spring elements are employed at the boundaries. The normal(k_n) and shear(k_s) stiffnesses are defined in the spring elements respectively. Both stiffnesses of the interface are assumed to have two different values depending upon whether slip along the pile-soil boundary has occurred. Before the slip occurring, k_{s1} is calculated assuming $(u_s)_{max}$ (Fig.5) to be as small as 0.1mm, which will make the resulting load-displacement curve to follow the elastic compression line of a pile. Meanwhile, k_{s2}, after the slip, is calculated using $(u_s)_{final}$ (Fig.5) of several millimeters, and the residual shear resistance coming from dilation of the slip surface, $\varDelta F_s$, is evaluated from the expression(Cundall, 1976)

$$\varDelta F_s = F_s - (F_s)_{max} = k_{s2}\varDelta u_s = k_{s2}(u_s - (u_s)_{max}) \qquad (1)$$

,where F_s : shear force, u_s : relative displacement, k_{s2} : shear stiffness after slip

The values of u_s can be compared with the model test results of Pells et.al(1980), in which $(u_s)_{max}$ ranged from 0.5 to 3mm, and $(u_s)_{final}$, 10mm. For the k_n before the slip, lateral deformation modulus, k_h, obtained from pressuremeter test(PMT) is used, and after the slip, it is assumed to be 10 times of the value of k_s as suggested by Belytschco et. al(1984).

NUMERICAL SIMULATION OF FIELD LOAD TEST CURVES

To care of the effect of soil layers around the pile shaft on the load-displacement behavior, the load-displacement curve of pile No.8 which is embedded only in the soil layers with styrofoam stacked at the bottom is simulated first. As seen in Fig.6, the fore part of the curve can be simulated reasonably well by the numerical manipulation under the assumption of no point resistance. The rear part of the curve is affected by the point resistance exerted from the styrofoam(Fig.7) after it is deformed to a critical state. After the effect of the styrofoam resistance is superimposed to the numerically predicted curve, the resulting σ - δ curve(Fig.6 – dashed line) shows similar trend with that of field load test.

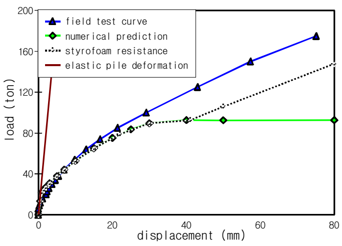

Fig. 6 Pile No.8

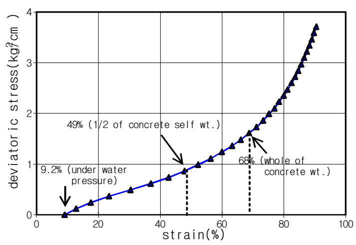

Fig. 7 Triaxial test result on styrofoam

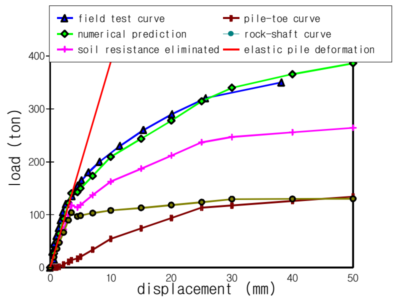

Fig. 8 Pile No.7

Generally, it is known that the very first part of the σ - δ curve of rock-socketed piles is affected by the lateral deformation modulus of the wall rock, and the elastic limit, by the magnitude of the cohesion of the rock mass, and the σ - δ behavior after slip, by the vertical subgrade reaction modulus of the pile base(Carter & Kulhawy(1988), Kulhawy & Goodman(1987)). Therefore, the peak cohesion before slip, c_{peak}, and the vertical reaction modulus of the base, k_b, are varied to produce similar σ - δ curves with those of field test. Fig.8 shows one of the resulting σ - δ curve obtained numerically, which compares very well with the field curve. In the same figure, the σ - δ curve of the rock-socketed portion of a pile is also shown which is obtained by eliminating shaft resistance of soil layers from the total σ - δ curve. The σ - δ curve of rock-socketed portion of the pile is, then, separated into the σ - ε curve of the shaft and the point of the pile as shown in the same figure. The σ - δ curve of the shaft shows the elastic-perfectly plastic behavior, and that of the point increases monotonically to a great extent of the pile settlement, which agrees with the observation of Carter and Kulhawy(1988).

TABLE 1 GEOTECHNICAL PROPERTIES AND MANIPULATION DATA OF THE ROCK MASSES

Pile No.(Embedment depth) / Result of experimental and numerical manipulation		1 (1.0D)	2 (0.9D)	4 (0.5D)	5 (1.2D)	6 (2.7D)	7 (1.9D)
Field & laboratory test data	Lateral deformation modulus of rock mass from PMT (GPa)	2.81	2.81	6.11	9.35	9.35	-
	Unconfined compressive strength of intact rock, q_u(MPa)	23.0	23.0	78.0	49.5	49.5	43.5
	RQD (%)	12	12	37	47	47	45
	Rock Mass Rating (RMR)	28	28	35	40	40	40
Input data for simulation	Pile-soil adhesion, c (kPa)	150	600	800	1200	1200	1200
	Reaction modulus (kPa/inch)	-	1400	3000	1000	4000	6000
	Friction angle, ϕ(°)	40°					
	Dilation angle, ψ(°)	10°					

In Table 1, the input data used to simulate the σ-δ curves of field tests are tabulated with the geotechnical properties of the base rock masses. As stated in the table, the internal friction angle and the dilation angle are fixed at 40° and 10° , respectively, which is justified because the variation of those angles has very limited influence on the result.

PREDICTED σ-δ CURVES

To generalize the numerical simulation procedure, it is pursued to find out the relations of the input data used in the simulation with the geotechnical field data such as the deformation modulus from PMT, the unconfined compressive strength of rocks(q_u), RQD, and the Rock Mass Rating(RMR) of Bieniawski(1976). The equations of the respective regression lines are shown in Eqs.2～4 for c_{peak}, and Eqs.5～7 for k_b. A couple of repredicted σ-δ curves are shown in Figs.9～11. From the comparisons of the field curves with the predicted ones in the figures, and from the inspection of the correlation coefficients of the regression equations, it is believed that any of the equations can be used to simulate the σ-δ curves of rock socketed piles in compliance with the availability of geotechnical field data.

$$c(\text{kPa}) = 109 \times 1.06^{RMR} \quad (2) \qquad k_b(\text{kPa/inch}) = 378 + 398 \times E_r(\text{GPa}) \quad (5)$$

$$c(\text{kPa}) = 297 + 95 \times E_r(\text{GPa}) \quad (3) \qquad k_b(\text{kPa/inch}) = 895 \times 1.04^{RQD} \quad (6)$$

$$c(\text{kPa}) = 339 + 18 \times RQD(\%) \quad (4) \qquad k_b(\text{kPa/inch}) = 77 \times 1.11^{RMR} \quad (7)$$

,where E_r = deformation modulus from PMT, k_b = vertical reaction modulus of the base

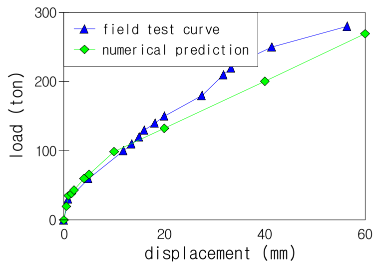

Fig. 9 Pile No.4

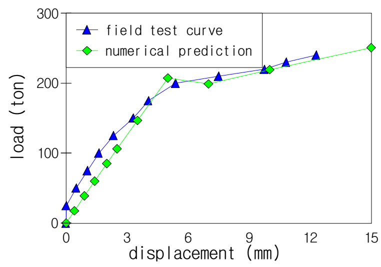

Fig. 10 Pile No.6

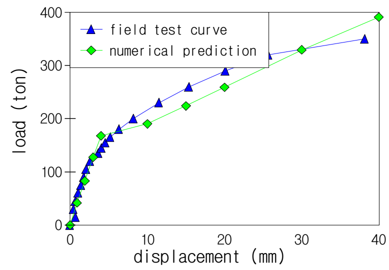

Fig. 11 Pile No.7

CONCLUSIONS

From a series of field pile load tests on the rock-socketed cast-in-situ concrete piles of 40cm diameter, and numerical simulation works, the followings are drawn as conclusions.

1) In simulating the load-displacement behavior of rock-socketed piles numerically, the roll of the peak cohesion intercept, c_{peak}, and the vertical subgrade reaction modulus of the base rock, k_b, are found to be very important, which can be estimated reasonably well from geotechnical field data, such as the deformation modulus from PMT, RQD, or RMR values, using the empirical expressions as suggested in this paper.

2) Since the empirical expressions suggested here are verified only for the cases from which the empirical relations are extracted, further verification processes are needed.

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