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Vertical bearing capacity of bored pre-cast pile with enlarged base considering diameter of the enlarged excavation around pile toe

K. Kobayashi & H. Ogura
Engineering Development Division, Japanpile Corporation, Tokyo, Japan

ABSTRACT: In Japan, most pile foundations using pre-cast concrete piles are installed by the bored pre-cast piling method. In this method, the pre-cast piles are placed in holes that have been excavated beforehand and filled with cement milk. The bored pre-cast piling method with enlarged base, in which a portion near the pile toe is enlarged during boring operation, is widely used because of the cost benefits and the increased vertical bearing capacity that this method offers. The Hyper-MEGA method, further developed as an improved bored pre-cast piling method with enlarged base, is reported here. This report gives an overview of the Hyper-MEGA method and the evaluation of vertical bearing capacity based on examples of application of the method. As a result, this method can give a vertical bearing capacity coefficient at the toe of $\alpha = 423$(for base enlargement ratio $\omega = 1.2$, where ω is defined as the ratio of enlarged base diameter to the pile nodule diameter). This is more than twice the corresponding value of $\alpha = 200$ for a conventional bored pre-cast pile.

1 INTRODUCTION

In Japan, most pile foundations using pre-cast concrete piles are installed by the bored.pre-cast piling method, wherein pre-cast piles are inserted in holes that have been excavated beforehand and filled with cement milk (term used for cement water slurry in Japan). The bored pre-cast piling method with enlarged base, in which a portion near the toe is enlarged during boring operation, is widely used because of the cost benefits owing to the increased vertical bearing capacity that this method offers. Conventionally, the diameter of the grouted base was enlarged to 1.1 to 1.15 times.

The Hyper-MEGA method (hereafter referred to as "the HM method") is a type of the bored pre-cast piling method with enlarged base. Its features include the use of nodular piles, the option of setting the enlargement excavation for the grouted base in the range of 1 to 2 times, and a longer enlarged excavation.

This report gives an overview of the HM method and the study on its vertical bearing capacity. Since the HM method makes use of various enlarged excavation diameters, the relationship between the end bearing capacity and the enlarged excavation diameter based on the results of loading tests is the focus of this report.

2 OVERVIEW OF THE HM METHOD

2.1 *Nodular pile installation method*

Nodular pile is a pre-cast concrete pile having protrusions in the form of nodes at fixed intervals over the pile shaft. Since its first appearance in the 1920s, the material has changed from reinforced concrete (RC) to pre-stressed spun concrete (PC) to pre-stressed spun high-strength concrete (PHC). The cross sections have changed from square/octagonal to triangular/hexagonal to cylindrical (spun pre-cast pile). In spite of the above, the pile diameter at the node had remained 440 mm (shaft diameter 300 mm) for along period. The method used to install this pile until 1970 was generally the "sealing method" in which the pile was driven into the ground while filling gravel around it.

From the 1970s, nodular piles could not be installed in urban areas by the "sealing method" because of the noise and vibration problems and the bored pre-cast piling method began to be used instead. The GMTOP method was developed in the year 2000. It is a method for installing nodular pile in a hole formed by stirring and mixing excavated earth with cement milk. It uses a rotating blade with a slit in the helical part of the blade for excavation.

The HM method is an improved version of the GMTOP method. It was developed in 2006 in response to the demand for high bearing capacity piles in recent years. The maximum diameter of the pile at the node has been increased to 1200 mm. A straight shaft pile with diameter equal to the diameter at the node can also be joined above the nodular pile (enlarged head nodular pile). This arrangement enables full utilization of all the advantages of the GMTOP method while also making its application as bearing piles possible. The end bearing capacity of the "bored pre-cast piling method with enlarged base" has been increased by enlargement excavation around the toe of 1 to 2 times. Furthermore, by enlarging the excavation over a range of up to 50% of the pile length, the skin friction resistance can also be increased significantly to realize high bearing capacity.

2.2 *Applicable piles*

Piles to which the HM method can be applied include nodular piles (including enlarged head nodular piles) with maximum diameter at the node of 1200 mm (shaft diameter 1000 mm), and straight piles (including enlarged head piles) such as PHC piles, PRC (prestressed and reinforced spun high strength concrete) piles, SC (steel encased spun high strength concrete) piles, and steel piles of maximum diameter 1200 mm that can be joined to nodular piles. Combinations of these piles can be freely used as long as a solid contact with the grouted base is achieved with nodular pile (including enlarged head nodular pile) at lower section. Figure 1 shows the form of nodular piles with diameter 800–1000.

Large horizontal resistance corresponding to the vertical bearing capacity can be obtained by using enlarged head nodular pile or enlarged head pile and by joining straight piles of diameter larger than that of the lower nodular pile at the top. Three kinds of piles with concrete strength F_c of 85 N/mm^2 (long-term permissible compressive stress level of 24 N/mm^2), 105 N/mm^2 (long-term permissible compressive stress level of 30 N/mm^2), and 123 N/mm^2 (long-term permissible compressive stress level of 35 N/mm^2) have been utilized.

2.3 *Ground conditions and installation depth*

The ground conditions and the maximum installation depth applicable to the HM method are given below.

(1) Soil around pile toe: Sandy soil, gravelly soil, clayey soil (including silty soil)
(2) Soil around the pile shaft: Sandy soil (including gravelly soil) or clayey soil (including silty soil)

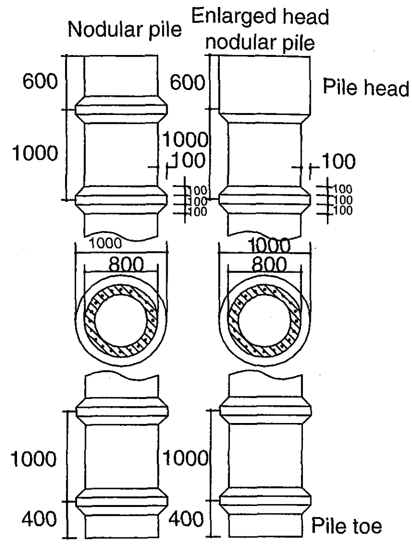

Figure 1. Shape of HC-TOP pile (ϕ1000–800).

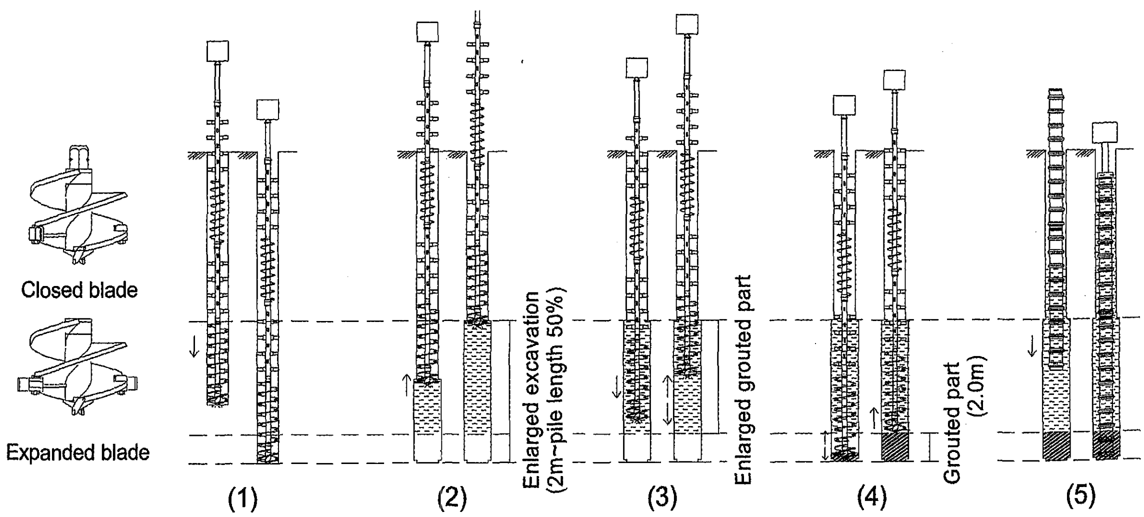

Figure 2. Installation sequence.

(3) Maximum installation depth: 68.5 m when the soil around the pile toe is sandy soil or gravelly soil, and 60.0 m when it is clayey soil.

2.4 *Installation method*

An overview of the installation procedure for the HM method is shown in Figure 2. Stages (1) to (5) in the Figure are briefly explained below.

(1) Centering of pile – normal excavation
(2) Expand blade – enlarged excavation; pumping grout (cement milk)
(3) Re-excavation as mixing and stirring continues
(4) Forming the grouted base- withdrawing the auger
(5) Pile placement and setting

The grout(cement milk) pumped before stirring and mixing may include expansive admixture in addition to the usual cement milk with the aim of increasing the shaft friction resistance. Use of grout without expansive admixture is referred to as "standard type," while the latter is referred to as "expansion type."

The enlargement head for boring may also be of two types – the mechanical type in which the blade is expanded by reverse rotation and the hydraulic type in which oil is pumped from the appropriate facility to expand the blade. Figure 2 shows the mechanical type of head. If the standard excavation diameter (diameter at the node + 50 mm) is D_s, and the enlarged excavation diameter is D_e, then the enlargement factor ω is defined as $\omega = D_e/D_s$.

Work monitoring is extremely important to ensure quality of the pile at the base in enlarged pre-boring and grouting method. The key points especially are checks to confirm that the expanded blade has correctly expanded and the grout has been properly pumped.

3 LOADING TEST AND INSTALLATION CONFIRMATION

Conventional static loading and pile toe loading test (Ogura et al. 2005) were carried out at 53 locations in various parts of the country from Hokkaido to Kagoshima in Japan. The pile diameter varied widely from 300 mm to 1200 mm, the pile length from 4 m to 68.5 m, the average *N*-value of the soil around the pile toe from 0 to 64.6 and the enlargement ratio ω from 1 to 2.

The load-displacement curve from the loading test (nodular pile with nodule diameter 500 mm and shaft diameter 400 mm, $\omega = 2.0$, overall pile length = 30 m, embedment depth = 29.2 m, standard type, Saitama Prefecture) in which the pile toe is positioned in gravelly soil is shown in Figure 3, as an example. In principle, staged loading system (maintained load duration of new load cycle is 30 minutes) in which creep deformations occur until the first limit resistance was utilized. Subsequently, a continuous loading method in which the behavior during large displacement can be accurately tracked was utilized. The principle assumed was that loading continues until the pile toe settlement became greater than 10% of the enlarged excavation diameter. The toe resistance and the skin friction resistance were obtained from the measurement of strain gauges fitted along the shaft of test pile.

Furthermore, to confirm the condition of the grouted base of the pile as far as possible, the ground was excavated at a few locations on site where the pile was installed, and dimensions and strength of soil cement of the grouted base were investigated. Photo 1 shows a section of the grouted base when a pile of embedment depth 5 m, diameter at node of 440 mm, shaft diameter of 300 mm and $\omega = 2$ was excavated. A cross section (outside diameter 300 mm, inside diameter 180 mm) of the nodular pile shaft can be seen at the center. The core strength and measured dimensions of piles excavated at this and other sites were found to satisfy the requirement for soil cement shaft strength and dimensions.

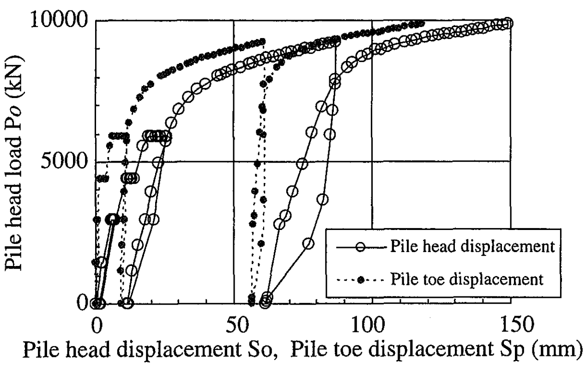

Figure 3. Load-displacement curve.

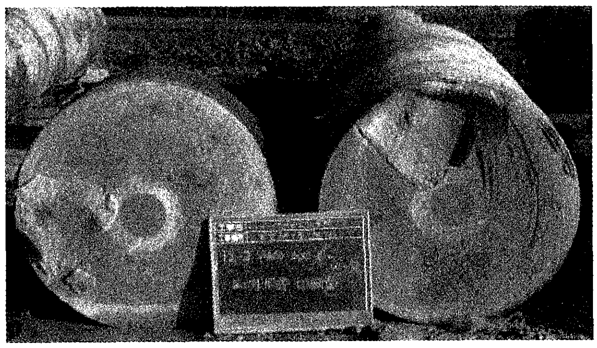

Photo 1. Building the grouted base.

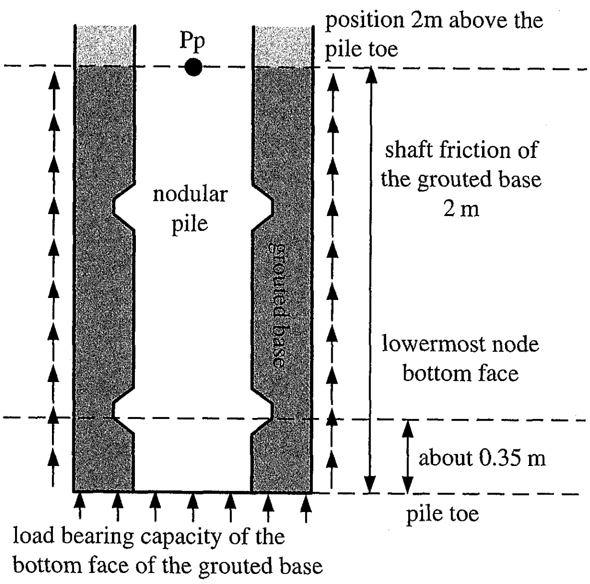

Figure 4. Evaluation position of pile end bearing capacity P_p.

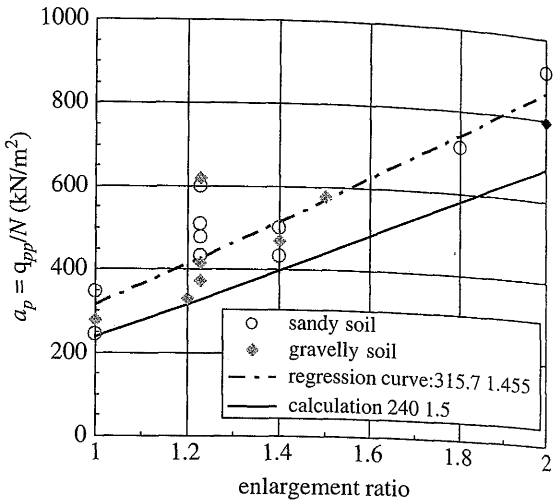

Figure 5. Relationship between α_p and ω.

4 STUDY ON VERTICAL BEARING CAPACITY

4.1 *Study on end bearing capacity*

As shown in Figure 4 and similar to other base enlarged pre-boring and grouting methods, the end bearing capacity P_p is evaluated by the axial force near the upper end of grouted base (at a position 2 m above the pile toe in this HM method). For this reason, the "end bearing capacity" includes the skin friction of the grouted base in addition to the load bearing capacity of the bottom face of the grouted base. As the two components are not evaluated separately in the conventional methods, relative contribution of these components in terms of bearing capacity mechanism has remained unclear.

The end bearing capacity P_p in the HM method is regarded as constituted by the combination of resistance P_{pp} of the pile toe bottom face and the shaft friction of the grouted base P_{pf} and evaluated accordingly. Actually, the axial force at the lowermost node position (about 0.35 m above the pile toe face) is used for P_{pp}.

The value obtained by dividing P_p and P_{pp} by the closed section area of the node A_p is taken as the end bearing pressure q_p and the bearing pressure of the lowermost node bottom face q_{pp} respectively.

To derive the equations for end bearing capacity in the HM method, the diameter of the grout base should be considered to vary in the range of 1 to 2 times corresponding to the standard excavation diameter. The conventional α is expressed by equation (1), where the coefficients a, b, c are determined from the data of loading tests.

$$a = \alpha_p + \alpha_f = a\omega^b + c\omega \quad (1)$$

here α_p = bearing capacity coefficient due to load bearing capacity at the pile toe face (lowermost node bottom face); α_f = bearing capacity coefficient due to skin friction resistance of grouted base; ω = enlargement ratio = $D_e / (D_o + 0.05)$; D_e = enlarged excavation diameter (grouted base diameter) (m); D_o = nodal diameter at node at the grouted base position (m).

Figure 5 shows the relationship between α_p and ω. The soil around the pile toe is taken as sandy soil and gravelly soil, and α_p plotted along vertical axis is the value obtained by dividing q_{pp} by average N-value of pile toe region (Ogura & Kobayashi 2005). The chained line in the figure is a regression curve approximated by applying the least squares method assuming exponential function. The regression equation in case of sandy/gravelly soil is given by $\alpha_p = 315.7\omega^{1.455}$. The value of b in equation (1) is taken as 1.5.

If the load-bearing capacity of the bottom face of the grouted base is considered proportional to the bottom area, then b is likely to be 2. However, the regression values of 1.455 for sandy/ gravelly soil and 1.259 for clayey soil are smaller. Three possible reasons given below may be considered for the small values.

(1) Skin friction resistance between the bottom face of the grouted base and the lowermost node bottom face (about 0.35 m) is included in P_{pp}.
(2) The ultimate load-bearing capacity is being evaluated as the load bearing capacity when S_p has reached 10% of the pile diameter (diameter at the node) D_0 and not 10% of the grouted base diameter D_e. For this reason, the increase in the toe resistance until S_p becomes $D_e/10$ from $D_o/10$ has not been reflected in q_{pp}.

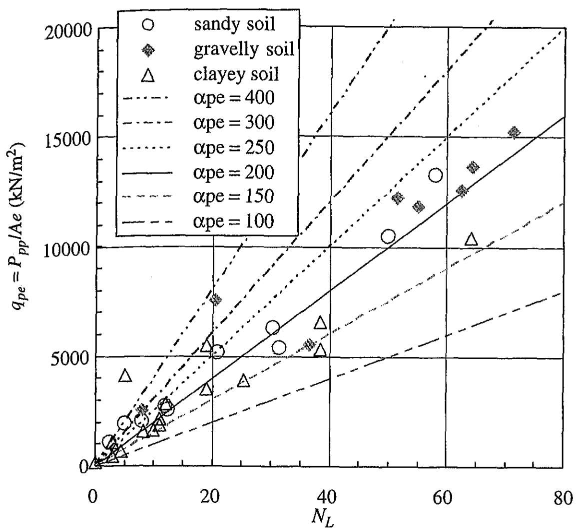

Figure 6. Relationship between q_{pe} and N_L.

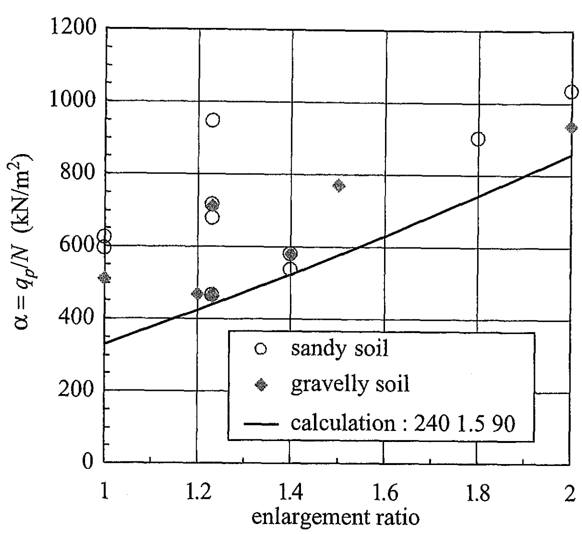

Figure 7. α–ω relationship.

3) A phenomenon by which the load bearing capacity is not proportional to the contact area (the so-called "dimensional effect") may also have its effect.

If a value of a that does not exceed data is determined, then it becomes 245.5 in sandy/ gravelly soil, and 219.2 in clayey soil. The a values for bearing capacity evaluation are taken as 240 and 210 respectively. As a result, the equations are determined as $a_p = 240\omega^{1.5}$ and $\alpha_p = 210\omega^{1.25}$. The curve obtained from the equations above is also plotted in Figure 5. The physical significance of α_p may be studied here. For $\omega = 1$ when the bottom area of the grouted base is almost equivalent to A_p, the values of α_p are 240 kN/m² and 210 kN/m² respectively. Figure 6 shows the relationship between the value q_{pe} (obtained by dividing P_{pp} not by A_p but by the bottom area of the grouted base A_e) and the mean N-value N_L (obtained for N-value from the pile toe to the depth of D_o+D_e). It can be observed that most of the values of $q_{pe}/N_L = \alpha_{pe}$ lie between 200 to 300 for sandy soil/gravelly soil, and between 150 to 250 for clayey soil. That is, even in the base enlarged pre-boring and grouting method wherein α is apparently large, when evaluated by the bottom area of the grouted base, this value is the same as 200 kN/m² or 250 kN/m², which is the value of α in the conventional pre-boring method, which corresponds to values of α_p when $\omega = 1$.

Since the grouted base behaves as an integral part of the pile until the soil around the pile toe of the enlarged grouted base reaches ultimate load-bearing capacity, the settings for the strength of the grouted base were studied using the results of FEM analysis (Ogura & Yamazaki 2006).

The shaft friction resistance of the grouted base is considered to be proportional to the shaft area of the grouted base. The result of plotting the values of the shaft friction resistance divided by ω with respect to the average N-value in pile toe region shows that the slope of this relationship is c in equation (1). Its lower limit is 98.2 for sandy/ gravelly soil, and is 99.8 for clayey soil. Thus, the value of c for both kinds of soil is taken as 90 from here onwards.

Based on the above, the equations of α for the different soils are as given in (2) and (3) below.

Sandy/ gravelly soil $\quad \alpha = 240\omega^{1.5}+90\omega \quad (2)$

Clayey soil $\quad \alpha = 210\omega^{1.25}+90\omega \quad (3)$

When $\omega = 2$, the value of α for sandy/gravelly soil becomes 858, and for clayey soil it becomes 679. Figure 7 shows the α–ω relationship based on the loading test data for sandy/ gravelly soil. The values in the data for both kinds of soil exceed the values obtained by the equations.

5 SHAFT FRICTION RESISTANCE OF PILE

Equations for calculating the shaft friction resistance are derived using the friction resistance in pile f_i, determined from the results of loading tests. The calculation equations are as shown in (4) and (5).

Sandy soil

$$P_{fs} = f_s L_s D\pi \quad \text{(kN)} \quad (4)$$
$$f_s = \beta N_s \quad \text{(kN/m}^2\text{)}$$

Clayey soil

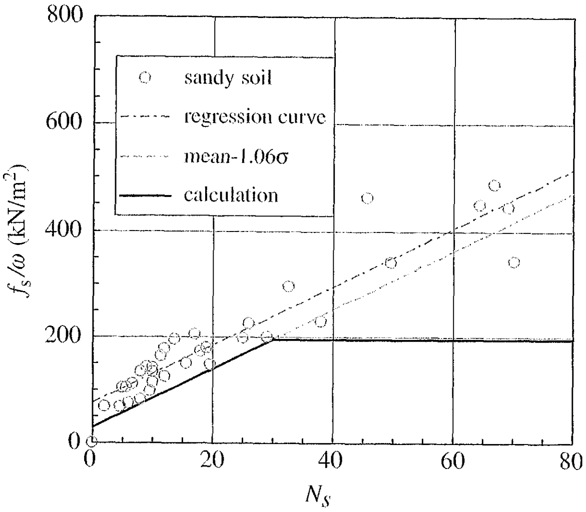

Figure 8. f_s–N_s relationship. (nodular pile; standard; sandy soil)

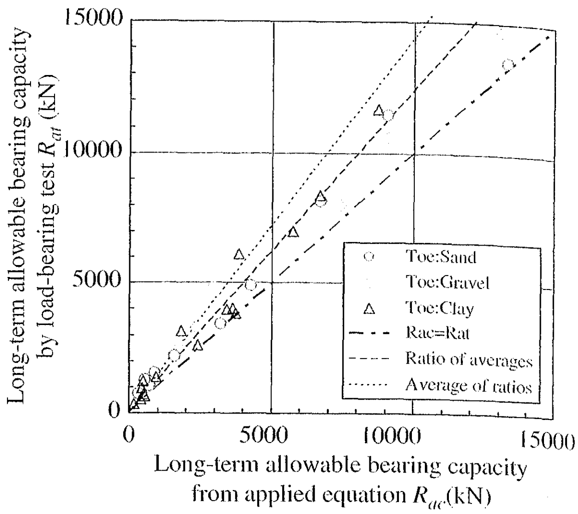

Figure 9. Correlation diagram for long-term permissible load-bearing capacity.

$$P_{fc} = f_c L_c D\pi \quad \text{(kN)} \tag{5}$$

$$f_c = \gamma q_u \quad (\text{kN/m}^2)$$

where P_{fs} = shaft friction resistance of sandy soil (kN); f_s = shaft friction in sandy soil (kN/m^2); L_s = total length of pile in contact with sandy soil (m); D = pile diameter (diameter at node in case of nodular pile) (m); β = shaft friction resistance coefficient of sandy soil; N_s = average N-value of sandy soil; P_{fc} = shaft friction resistance of clayey soil (kN); f_c = shaft friction in clayey soil (kN/m^2); L_c = total length of pile in contact with clayey soil (m); γ = skin friction resistance coefficient of clayey soil (kN); q_u = mean value of axial compressive strength (kN/m^2)

Load bearing mechanism has been studied based on the loading test data and values of shaft friction at ultimate load-bearing capacity. In case of nodular piles, the occurrence of slip between pile shaft and cement slurry due to bearing pressure of the node cannot be considered. Moreover, slip occurred between the cement slurry and the soil in the observations during loading tests. Accordingly, the skin friction resistance of nodular pile is decided by the frictional resistance between the cement slurry and the soil, and is considered to be proportional to the outer diameter (excavation diameter) of the cement slurry. Thus, the skin friction stress of nodular pile was estimated using the excavation diameter (taken as ωD_o) here and not the nodal diameter D_o.

Figure 8 shows the f_s–N_s relationship of sandy soil when standard type (no expansive admixture) slurry is used around the pile. The number of data points is 33. Since the equation for (mean − 1.06σ) is $f_s/\omega = 30.3 + 5.50\ N_s$, the calculation equation for β becomes (6).

$$\beta N_s = (30 + 5.5 N_s \omega) \tag{6}$$

Studies where carried out similarly for calculation equations for shaft friction resistance of other piles according to pile type, soil, and cement slurry around pile shaft (Ogura et al. 2007).

6 LONG-TERM PERMISSIBLE CAPACITY

P_p and P_f are determined from the calculation equations derived in the studies in Sections 4 and 5, and the long-term permissible vertical bearing capacity R_a was calculated taking a safety factor of 3.

Figure 9 shows the correlation between the long-term permissible load-bearing capacity R_{ac} from calculation, and the long-term permissible load-bearing capacity R_{at} obtained from loading tests (one-third the ultimate load-bearing capacity). In all the loading tests, R_{at} exceeds R_{ac}. The mean value of the ratio between the two parameters was 1.44, while the ratio of the mean values of the two parameters was 1.24. From the above, it is concluded that the proposed equation offers safe long-term permissible load-bearing capacity.

7 CONCLUSION

In the last few years, a new pre-cast pile installation method for high load-bearing capacity is being actively developed in Japan. This development is in response to the demand for an installation method that enables the design of one-column one-pile system by designers in order to reduce pile installation costs.

The Hyper-Mega method has the advantage that a pile diameter of 1 to 2 times can be arbitrarily selected without keeping the base enlargement factor at a fixed value. For this reason, the authors have collected valuable data during the development stage related to the effects of the excavation diameter on the end bearing capacity and shaft friction resistance. These characteristics of vertical bearing capacity have been explained and discussed in this report.

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