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LOAD TESTS ON BORED PHC NODULAR PILES IN DIFFERENT GROUND CONDITIONS AND THE BEARING CAPACITY BASED ON SIMPLE SOIL PARAMETERS

多様な地盤に施工された埋込みPHC節杭の現場載荷試験結果と地盤定数による支持力の評価

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PHC nodular piles are considered to be a favorable foundation option for low to medium-rise buildings at deep soft ground sites. A comprehensive load test program was undertaken to evaluate the bearing capacity of these piles in different site conditions. A convenient and adequate approach to evaluate bearing capacity of PHC nodular piles is proposed based on the test results.

キーワード：
PHC節杭，埋込み工法，載荷試験，杭支持力，地盤定数，支持力の評価

Keywords：
PHC nodular pile, bored pile, load test, bearing capacity, soil parameters, bearing capacity evaluation

1 Introduction

Nodular precast piles have come a long way since the idea was perceived more than six decades ago[1] as a means of enhancing shaft friction. Over the years, nodular piles of different cross-sectional shapes made of reinforced concrete and prestressed concrete have been in use. Recently, prestressed precast high strength concrete (PHC) nodular piles have proved to be a favorable foundation option for low-rise to medium-rise buildings located at deep soft soil sites. These piles are unique in that the load bearing resistance is primarily derived from adhesion and friction along the pile shaft. A rational approach to the development of a reliable method for the design of PHC nodular piles considering readily available soil parameters is presented.

Previous studies[1)~4)] on model and full-scale tests have clarified the load bearing mechanism of nodular piles. However, the load bearing capacity is influenced by the method of installation. PHC nodular pile installed by inserting into a drilled hole and bonded to the ground by cement grout, often referred to as the bored pile, is considered in this study. Considering that the soil type and the standard penetration test (SPT) N-values constitute the basic information from site exploration records in Japan, attempt is made to develop a simple correlation between these parameters and the bearing capacity.

Table 1 Pile Load Test Sites and Summary of Test Results

No.	Pile test location	Test pile		Soil type at pile toe	Full excess soil disposal (FES)					Low excess soil disposal (LES)				
		L (m)	Type		No.	N_P	R(ton)	Shaft%	Toe%	No.	N_P	R(ton)	Shaft%	Toe%
1	Edogawa-ku	12	B	Sandy silt	F1	1.5	78.0	93	7	L1	2.3	97.0	94	6
2	Kawaguchi City	12	B	Sandy silt	F2	28.0	207.6	71	29	L2	20.7	220.0	74	26
3	Fujishima-cho	12	B	Silt	F3	5.0	120.9	90	10	-	-	-	-	-
4	Otaru City	6	A	Sand	F4	15.0	128.6	64	36	L3	15.0	96.2	67	33
5	Iwamizawa City	12	B	Humus	F5	4.0	46.5	95	5	-	-	-	-	-
6	Togo-cho	18	B	Sand	F6	10.8	240.0	90	10	-	-	-	-	-
7	Isshiki-cho	10	B	Coarse sand	F7	12.3	130.7	71	29	-	-	-	-	-
8	Shimizu City	8	B	Sand	-	-	-	-	-	L4	5.0	39.8	73	27
9	Fuji City	24	B	Silt	F8	2.9	125.5	94	6	L5	2.9	153.4	95	5
10	Hayato-cho - 1	20	B	Silty sand	F9	12.0	219.5	87	13	L6	12.0	272.0	88	12
11	Hayato-cho - 2	9	B	sandy silt	F10	2.3	70.0	91	9	L7	2.0	100.8	90	10
12	Sapporo City	10	A	Fine sand	F11	2.0	50.3	85	15	L8	2.0	62.8	84	16
13	Soka City	12	B	Silt	-	-	-	-	-	L9	1.5	70.0	94	6
14	Ebetsu City	12	B	Sandy silt	-	-	-	-	-	L10	26.5	200.0	70	30
					No. of field load tests =11					No. of field load tests = 10				

2 Pile Installation Methods

The pile installation technique considered in this paper is also known as pre-boring method. Two types of augers were utilized for drilling. One was the normal multiple-

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flight auger in which full excess soil (FES) from the drilled hole needs to be disposed of. The other was a specially designed auger that reduces the excess soil from drilled hole. When this auger is used, the soil around the drilled hole gets compacted, resulting in low excess soil (LES). Excess soil volume reduction is of the order of 30 to 60% depending on the soil types. Drilled hole diameter was 500 mm in both the FES and LES methods. Of the 21 field tests carried out as part of this study, 11 consist of installations by FES method and are numbered F1 to F11 in Table 1. The remaining 10 installed by LES method are numbered L1 to L10. The locations, the pile lengths and the type of soil around pile toe are also shown in Table 1. The designation of A and B types of pile in Table 1 concerns the difference in the concrete strength and the effective prestress, the cross-sectional dimensions remaining the same. The B type has higher concrete strength and highere effective prestress than the A type. The field test locations cover 14 different sites from Hokkaido to Kyushu in Japan.

The bearing capacity of bored PHC piles is derived from the transfer of pile load to surrounding soil by the hardened cement grout. So the grout needs to be of superior strength than the surrounding soil. Cement grout of higher target strength was injected around pile toe region in recognition of generally higher soil strength there. The water-cement ratio of grout in the pile toe region was 100% and that around the pile shaft was 140%. The minimum 28 day target strength of hardened cement grout was 75 kg/cm^2 and 30 kg/cm^2 respectively.

To ensure that the strength of the cement grout hardened within the ground after pile installation met the target strength requirements, two 5m long piles installed by FES and LES methods were dug out for the investigation. Samples of hardened cement grout were cut out from the pile shaft and the pile toe regions and tested for unconfined compressive strength. Table 2 shows the 21 day average strength of the cement grout samples cut out from the dug out pile. Comparatively higher strength of the cement grout around the pile shaft in one of the dug-out piles is probably because the cement grout injected at the pile toe got mixed with that around the pile shaft during installation. A comparison with correlation of strength with age in days, based on laboratory test results, is also shown in Table 2. It is seen that the corresponding strengths of the cut out samples are equivalent or higher than those estimated based on laboratory tests. It was also noted that the cement grout is bonded well with the pile shaft.

Table 2 Unconfined Compression Test on Samples from Dug-out Piles

21 day average compressive strength F_{21} of samples cut out from the dug out pile (kg/cm^2)			
FES	LES	FES	LES
37.9	98.5	154.1	128.1
For W/C Ratio=140% F_{days}=-1.5+13.7ln(days) F_{21}=40.1 kg/cm^2		For W/C Ratio=100% F_{days}=7.9+33xln(days) F_{21}=108.5 kg/cm^2	

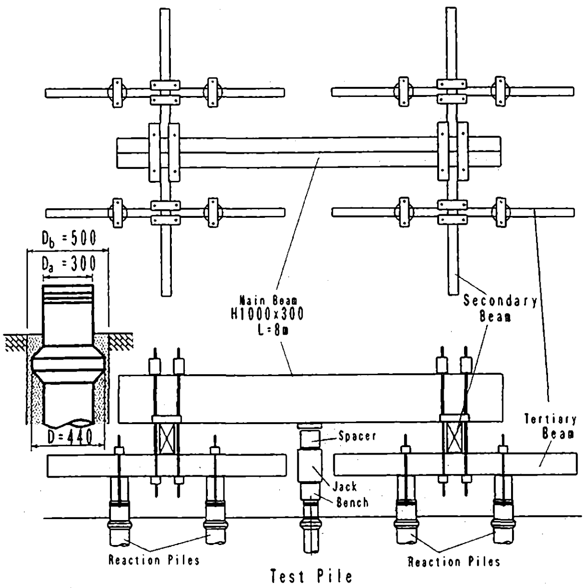


Fig. 1 Example of Field Load Test System

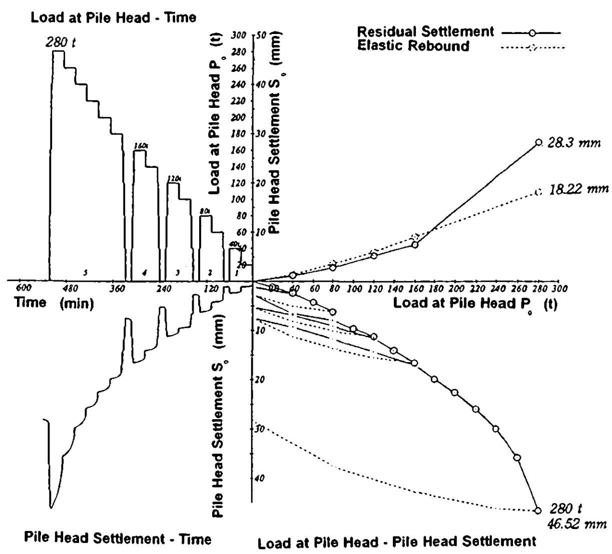


Fig. 2 Typical Example of Load-Settlement-Time Plot

3 Pile Load Tests

Slow maintained-load tests were carried out following the recommendations in JSSMFE Standard[5]. A typical field load test set up is shown in Fig. 1. The incremental loads, applied by jacking against reaction piles, was maintained at constant values for a period of 30 minutes, and the load was also cycled back to zero incrementally as recommended in the standard[5]. The load at pile head P_0, the pile head settlement S_0 and the pile toe settlement S_P were measured directly. The axial force distribution, including the bearing resistance at pile toe P_P, was derived from strain gage readings. The horizontal displacement at pile head and the movement of reaction piles was also monitored for accuracy of load tests. The load was increased until creep deformations were excessively large. A typical load-settlement-time plot from the load test is shown in Fig. 2, where five cycles of load with a maximum maintained load of 280t was applied.

Fig. 3 shows an example of the arrangement of strain gage layout along the pile length shown by circled numbers, along with the soil profile. Two piles installed by FES method (F9 in Table 1) and LES method (L6 in Table 1), but having the same layout were tested at this site. The axial force distribution at different maintained load levels, based on the strain measurements during the load tests, are shows in Fig. 3 by the broken line for test F9 and by the solid line for test L6.

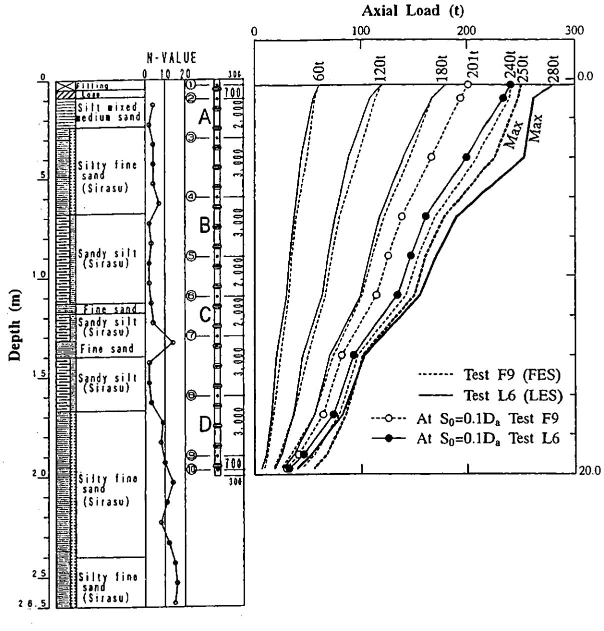


Fig. 3 Typical Strain Gage Layout and Axial Force Distribution

4 Load-Settlement Characteristics

Fig. 4 (a) and 5 (a) show the P_0-S_0 characteristics for piles installed by FES and LES methods respectively. The numerals in square brackets show pile lengths in meter. It is clear that the load bearing characteristics depend on the pile length as well as on the ground condition. Horizontal dotted lines correspond to $S_0=0.1D_a$, where D_a is the diameter of pile axis (Fig. 1). Tests F6 in Fig. 4 (a) and L10 in Fig. 5 (a) were not loaded to pile head settlement of $0.1D_a$, but both the $P_0 \sim S_0$ curves indicate increasing trend with the increase in S_0. Also, the load at pile head P_0 is seen to increase with S_0 well beyond $S_0=0.1D_a$ for all the other tests.

Fig. 4 (b) and 5 (b) show the variation of the pile shaft frictional resistance P_f with S_0, and Fig. 4 (c) and 5 (c) show the $P_P \sim S_P$ relationship (load-displacement at pile toe). The numerals in square brackets in Fig. 4 (c) and 5 (c) show average N-value N_P in the region between $1D$ above and $1D$ below the pile toe, where D is the diameter of the pile node (Fig. 1). Test F5 in Fig. 4 (c) shows exceptionally low pile toe resistance. This is because the soil around pile toe region is humus (Table 1), which offers low resistance to sustained bearing load. However, it is clear from Fig. 4 (c) and 5 (c) that N_P is a good indicator of the bearing resistance at pile toe for soil types other than humus. Two distinct groups of $P_P \sim S_P$ curves seen in Fig. 4 (c) and 5 (c) correspond to those with N_P less than 5.0 and those with N_P greater than 10.0.

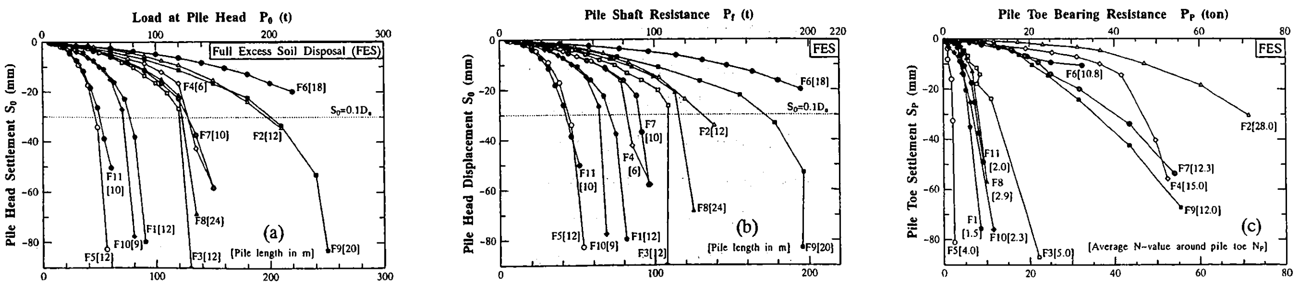


Fig. 4 Dependence of Load Bearing Resistance on Settlement for FES Method

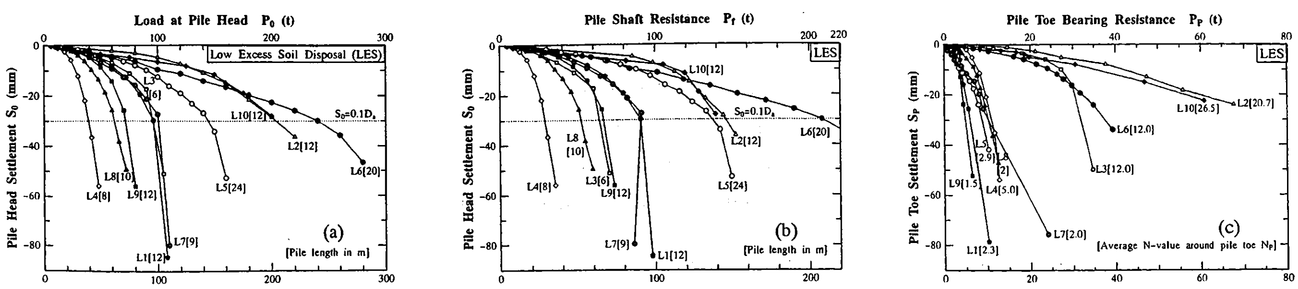


Fig. 5 Dependence of Load Bearing Resistance on Settlement for LES Method

5 Analysis of the Test Results

The JSSMFE Standard[5] defines the ultimate resistance R as smaller of the load P_0 at pile toe settlement S_P of 10% of the pile toe diameter and the load at which $P_0 \sim S_0$ curve is regarded to be almost parallel to S_0-axis. In case of the PHC nodular pile, three possible definitions of pile toe diameter exist: outside diameter of the hardened cement grout D_b (Fig. 1) that corresponds to the diameter of the hole drilled by the auger, the diameter at the nodule D and the diameter of the pile axis D_a. For the lack of a clear guideline, conservatism was exercised and D_a was used for this purpose. In addition, R was defined as P_0 at $S_0=0.1D_a$ instead of that at $S_P=0.1D_a$, adding further to the conservatism. The values of R in Table 1 are those of P_0 at $S_0=0.1D_a$, except for tests F6 and L10 in which case they are maximum values of P_0 because they were not actually loaded to $S_0=0.1D_a$ as noted above.

Attempt was also made to estimate the yield load P_y from the test results. Values of S_0 at estimated yield loads were found to vary from about 10 mm to more than 20 mm. The fact that the shaft resistance S_f continues to increase even after $S_0=20$ mm for all the tests can also be noted from Fig. 4(b) and 5(b). Considering the specification[5] that the long term allowable design load is smaller of $P_y/2$ and $R/3$, the ultimate load R was found to control the pile bearing capacity in all the cases.

Table 1 also shows percentages of R borne by the shaft resistance and the bearing at pile toe. Overall, 70 to 90% of the ultimate load is borne by pile shaft depending on the average N-value N_P around pile toe. In most cases the share of shaft resistance is more than 85%.

As the cement grout, bonded well with the pile shaft, is of considerably higher strength compared to generally soft soils it is in contact with, it is logical and realistic to consider the slippage to occur between hardened cement grout and soil interface when the load reaches yield level. Presence of nodules at regular intervals along pile length makes is unrealistic to consider slippage along pile shaft and cement grout interface. Consequently, the unit shaft resistance τ is calculated based on diameter $D_b=500$ mm (Fig. 1). For the calculation of the unit bearing resistance at pile toe q, the nodal diameter D is utilized. This is done in recognition of the fact that P_P is measured at a point 300 mm above the actual pile toe and just below the lowest node as shown in Fig. 3 by ⑩. Presence of the node close to the pile toe can be expected to result in larger effective bearing area similar to piles with expanded bulbs[6].

6 Determination of Soil Parameters

Soil types within the strain gage intervals, such as shown in Fig. 3, were broadly classified as either sand, clay or humus, depending on the major content. The corresponding average N-values, N_S, N_C or N_h, were determined from simple average of N-values within the interval. The unit shaft resistance within respective strain gage intervals were calculated from axial force distributions, such as shown in Fig. 3. Typical variation of unit shaft frictional resistance (τ_i) with the interval settlement (S_i) for tests at the site shown in Fig. 3 is given in Fig. 6.

The $\tau_i \sim S_i$ curves marked as A, B, C and D in Fig. 6 correspond to the respective strain gage intervals marked similarly in Fig. 3. Fig. 6 shows relatively large unit shaft resistance in case of the pile installed by the LES method (solid lines with solid marks) in comparison to that installed by the FES method (dotted lines with open marks). The higher values of τ_i in case of the LES method reflects the extent of improvement of the surrounding soil at this site as a result of the use of special auger (to reduce the volume of excess soil displaced). As can be noted in Fig. 3, mostly sandy soils are present in the intervals marked A, B, C and D. In general, such distinct improvement in τ_i in LES method over FES method was not observed consistently, specially in clayey soils. For this reason, no attempt was made to make a distinction between LES and FES methods for the purpose of developing relationships for the evaluation of the bearing capacity.

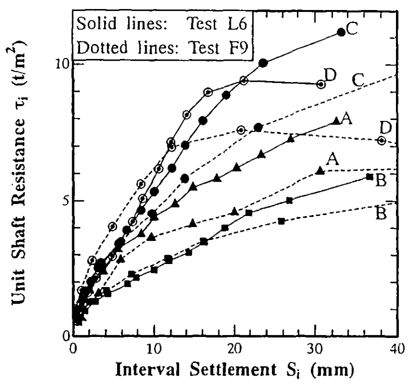


Fig. 6 Example of Unit Shaft Resistance in FES and LES Methods

The average N-value N_P of the soil bearing the pile toe was determined as the simple average of N-values in the region between $1D$ above and $1D$ below the pile toe, without regard to the soil type. However, the soils around the pile toe region are mostly sandy types as can be seen Table 1. In case of the presence of the humus type of soil around the pile toe region, the pile was considered to be completely floating with no end bearing resistance. That is, the end bearing resistance of humus was totally disregarded.

7 Effect of Low Excess Soil Disposal

FES and LES methods differ only with respect to the extent of excess soil to be disposed of. LES method can be expected to result in the increased shaft resistance P_f owing to the improvement of surrounding soil by compaction. The Extent of increase, however, depends on the soil type. Generally, the shaft resistance is expected to increase in sandy soils, whereas the increase may be insignificant in clayey soil conditions. As discussed above, the increase in τ_i at a given S_i in case of LES method (test L6) over that of FES method (test F9) in Fig. 6 illustrates the extent of improvement at a site consisting of mostly sandy soil condition. Different extent of improvement in R as a result of increased shaft resistance in LES can also be noted in Table 1. One exception is the decrease in R from F4 to L3 at a site, which is probably the result of a local variation in ground conditions because the two test piles were rather far apart.

8 Load Bearing Capacity and Soil Parameters

To develop a relationship betwwen the soil parameters and the load bearing capacity of bored PHC nodular piles, it is necessary to determine the components of R borne by different soil types along the pile shaft. For this purpose, the axial load distribution at a load equal to R at pile head, that is the axial load distribution at a pile head displacement $S_0=0.1D_a$, was determined by linear interpolation. An example of the distribution of R is shown in Fig. 3 for tests F9 (dotted line with open circle) and L6 (solid line with dark circles). From this, the unit shaft resistance within different strain gage intervals was calculated and denoted as τ_s, τ_c or τ_h depending on whether the soil type in the interval was classified as sand, clay or humus. The unit shaft resistance values are plotted against the average N-values in Fig. 7(a)~(c) for the three broadly divided soil types: sand, clay and humus. An attempt was made to carry out linear regression to fit the data to a straight line of the form $y=a+bx$. The average relationships are shown by dotted lines and the average minus one standard deviation (Avg-σ) relationships are shown by solid lines in Fig. 7(a)~(c).

No distinction is made between FES and LES methods in developing the correlations shown in Fig. 7(a)~(c). LES method considered separately showed higher average shaft resistance, but the dispersion was larger as well. The relative comparison of the correlation equations considering three different conditions is shown in Table 3. When Avg-σ equations are compared, there is no significant difference between the three conditions. Also, soil boring tests to measure N-values close to piles installed by LES method, conducted after the installation of the pile, indicated no significant changes in the N-values from those measured before pile installation. It seems N-values cannot depict adequately the increase in shaft resistance resulting from improvement of surrounding soil in LES method. It was decided to consider FES and LES methods together as shown in Fig. 7(a)~(c).

Values of unconfined compressive strength q_u were available from the soil exploration records at some of the test locations. Fig. 8 shows the unit shaft resistance of clayey soils plotted against $q_u/2$. It is clear that the variation of unit shaft resistance with $c_u=q_u/2$ shows the scatter quite similar to that with average N-values seen in Fig. 7(b).

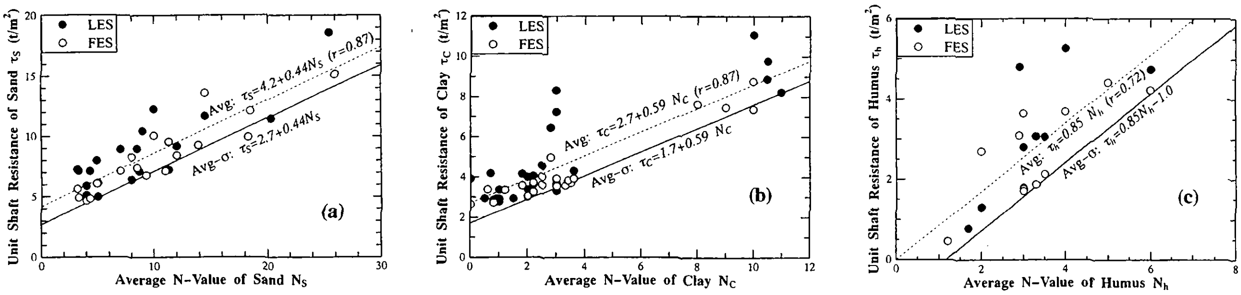


Fig. 7 Correlation of Unit Shaft Resistance with Soil Parameters

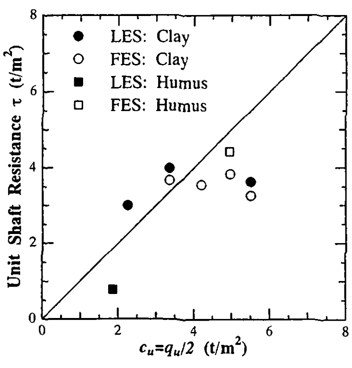


Fig. 8 Unit Shaft Resistance of Clayey Soils and $q_u/2$

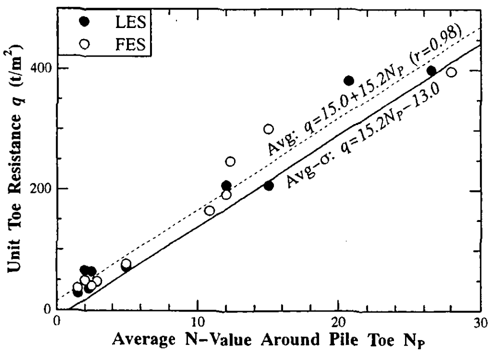


Fig. 9 Correlation of Unit Pile Toe Resistance with N_P

Fig. 9 shows the plot of unit pile toe resistance q against the average N-value N_P. Similar to the case of unit shaft resistance, the average and (Avg-σ) correlations are shown. It is noted that there is relatively smaller dispersion in this case. As explained above, the q value from test F5 is not entered here because the soil type around the pile toe is humus.

9 Measured and Calculated Bering Capacities

The correlations developed above can be utilized to evaluate the bearing capacity of PHC nodular piles. Fig. 10(a) shows the bearing capacity R measured from the 21 field tests shows in Table 1 plotted against the corresponding bearing capacities R_B calculated using the average correlation relationships. The dotted line of $R=R_B$ is also shown, which is seen to represent the average of data points from both the FES and LES methods logically. Similarly, Fig. 10(b) shows values of R plotted against R_B values calculated from Avg-σ correlations. It is seen that R_B becomes less than R in almost all the cases. Accordingly, Avg-σ relationships may be used for a rational and reliable estimate of bering capacity from soil parameters. Therefore, the set of Equations (1)～(6) is proposed for use in the design of bored PHC nodular piles.

$$R_B=\frac{\pi D^2 q}{4}+\sum\{\tau_S\times L_S+\tau_C\times L_C+\tau_h\times L_h\}\pi D_b \quad (1)$$

$$q=15.2N_P-13.0 \text{(Except that } q=0 \text{ for Humus)} \quad (2)$$

$$\tau_S=0.44N_S+2.7 \quad (3)$$

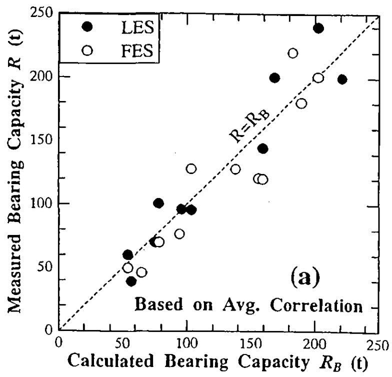
$$\tau_C=0.59N_C+1.7 \quad (4)$$

$$\tau_h=0.85N_h-1.0 \quad (5)$$

(6)

Table 3 Different Cases of Correlation with Soil Parameters

Condition	Sand	Clay
FES	Avg: $\tau_S=3.9+0.43N_S$ $\sigma=1.3$ Avg-σ: $\tau_S=2.6+0.43N_S$	Avg: $\tau_C=2.4+0.56N_C$ $\sigma=0.5$ Avg-σ: $\tau_C=1.9+0.56N_C$
LES	Avg: $\tau_S=4.5+0.46N_S$ $\sigma=1.7$ Avg-σ: $\tau_S=2.8+0.46N_S$	Avg: $\tau_C=3.0+0.62N_C$ $\sigma=1.3$ Avg-σ: $\tau_C=1.7+0.62N_C$
LES+FES	Avg: $\tau_S=4.2+0.44N_S$ $\sigma=1.5$ Avg-σ: $\tau_S=2.7+0.44N_S$	Avg: $\tau_C=2.7+0.59N_C$ $\sigma=1.0$ Avg-σ: $\tau_C=1.7+0.59N_C$



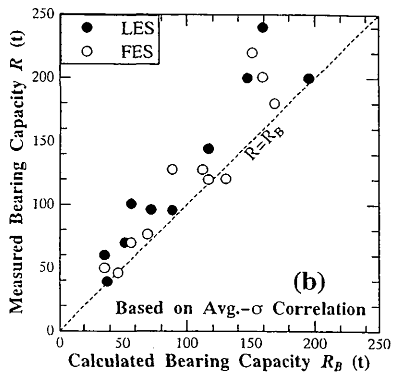


Fig. 10 Measured and Calculated Values of Bearing Capacity

10 Conclusions

A practical approach to evaluate the bearing capacity of the bored PHC nodular pile from the soil parameters readily available in practice is proposed based on the results of an extensive field load test program comprising of various soil conditions. It is seen that the increase in the pile shaft resistance, owing to the soil improvement as a result of reduced excess soil to be disposed of in LES method, cannot be adequately represented by correlations based on SPT N-values. However, the overall correlation consisting of both FES and LES methods considered together gives a set of rational and convenient relationships covering broad soil types and average N-values. The relationships developed based on the results of the field load test program are expected to be useful in practical design applications.

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