ESTIMATION OF *N*-VALUE USING PRESSURIZED SAND TANK AND VERIFICATION OF VERTICAL LOADING TEST OF MODEL PILE

加圧土槽を用いた*N*値の評価および模型杭の鉛直載荷試験結果の検証

Kazuma ISHIKAWA——＊1　Atsushi ITO ———＊2
Hitoshi OGURA ———＊3　Yuji SHIMOHIRA ——＊4

石川一真 ———＊1　伊藤淳志 ———＊2
小椋仁志 ———＊3　下平祐司 ———＊4

Keywords:
Standard penetration test, *N*-value, Pressurized sand tank, Relative density, Vertical loading pressure, Bearing capacity

キーワード：
標準貫入試験，*N* 値，加圧土槽，相対密度，上載圧，支持力

The *N*-value of soil in the pressurized sand tank was necessary for comparing the results of loading test on model pile and actual pile. The relationships between relative density after pressurizing the soil, vertical loading pressure and the *N*-value were obtained from the standard penetration test in the tank. Using these relationships, the relationship between the bearing capacity of the model pile and the *N*-value was determined. By comparing this relationship with similar relationships obtained from the loading tests of actual pile, the results of model loading test using the tank and those of the standard penetration test were validated.

1. Introduction

Some of the authors of this report have been conducting model loading tests related to pile toe grouted base in the bored piling method using pressurized sand tank [1),2)]. The tank was used because it is important to reproduce the stress condition of the actual soil in the model soil[3)]. The *N*-value of soil in the tank was necessary for comparing the results of loading test on model pile and that of actual pile. However, the findings of research[4)-8)] already carried out to study the relationship between the relative density of soil in the tank, the vertical loading pressure and the *N*-value were not used because the test conditions were different; that is, the vertical loading pressure was smaller than that of the soil in the tank used in the model loading test, and so on. The standard penetration test was carried out in the tank and the relationships between relative density after pressurizing the soil *Drc*, vertical loading pressure *pv* and *N*-value were studied. The relationships obtained were used to assess the soil of the model loading test, and the bearing capacity of the model pile was validated from the results of the loading test on the actual pile.

2. Standard penetration test

2.1 Overview of test

Fig. 1 shows the test apparatus. The pressurized sand tank is a steel tank with an inside diameter of 584 mm, height of 700 mm and thickness of 15 mm. To reduce the frictional resistance between the soil and the inside wall surface of the tank, two layers of teflon sheets (with silicone grease applied between the sheets) were affixed on the inside wall surface (refer to Reference 9). The pressurizing plate for applying vertical loading pressure on the soil was a rigid steel plate with an outside diameter of 568 mm and thickness of 30 mm. A hole (diameter 100 mm) was made at the centre of the plate for penetration of the sampler. Vertical loading pressure was applied on the soil using four hydraulic jacks (capacity 50 kN each) through the pressurizing plate.

The sand used for the model soil was Yodo River sand. After washing with water, the sand was passed through a 1.2 mm sieve and dried naturally. Table 1 shows the physical properties of sand, while Fig. 2 shows the results of the grain size analysis. The sandy soil was prepared by multiple sieving pluviation method using hose-type sand rainer, as shown in Fig. 3 (refer to Reference 10). The hose-type sand rainer can be used to control the relative density of soil by varying the diameter *d* of the hole in the flow regulating plate and the height through which sand falls.

Standard penetration test apparatus complying with JIS A 1219 was used. Automatic drop system (semi-automatic type) was used so that the weight could be dropped from a specific height (76±1 cm). When the catcher in contact with the weight passes the protrusion of the rod, the weight dissociates from the catcher.

A part of this report was published in the Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan (2011) [note 1)].

*1 Graduate School of Science and Engineering, Kansai Univ., JAPAN PILE Corporation, M. Sci.
*2 Assoc. Prof., Dept. of Architecture, Faculty of Environmental and Urban Engineering, Kansai Univ., Dr. Eng.
*3 JAPAN PILE Corporation, Dr. Eng.
*4 General Building Research Corporation of Japan, Dr. Eng.

*1 関西大学大学院，ジャパンパイル㈱　修士（理学）
（〒103-0007　東京都中央区日本橋浜町 2-1-1）
*2 関西大学環境都市工学部建築学科　准教授・博士（工学）
*3 ジャパンパイル㈱　工博
*4 （一財）日本建築総合試験所　博士（工学）

The test parameters were: four vertical loading pressures *pv* at 150, 300, 450, 600 kN/m², and four values of relative density before pressurizing the soil *Dri* at approximately 60, 70, 80, 90 %. The test was performed by applying vertical loading pressure after preparing the soil. After pre-driving to 15 cm from the soil surface, main driving was performed through 30 cm, and the penetration of the sampler for each blow and the displacement of the pressurizing plate were measured. Moreover, the relative density before and after pressurization was calculated from the mass of the sand in the pressurized sand tank and from the tank volume before and after pressurizing the soil. It was confirmed from the earth pressure gauge installed on the bottom of the tank that the vertical loading pressure had reached the bottom without any reduction in its value.

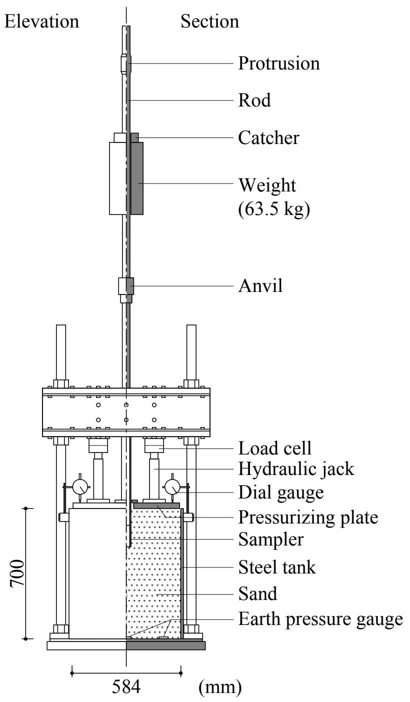

Fig.1. Test apparatus

Table 1. Properties of sand

Soil particle density	ρ_s (g/cm^3)	2.62
Mean grain size	D_{50} (mm)	0.54
Uniformity coefficient	U_c	2.17
Dry unit weight in densest condition	ρ_{dmax} (g/cm^3)	1.631
Dry unit weight in loosest condition	ρ_{dmin} (g/cm^3)	1.314

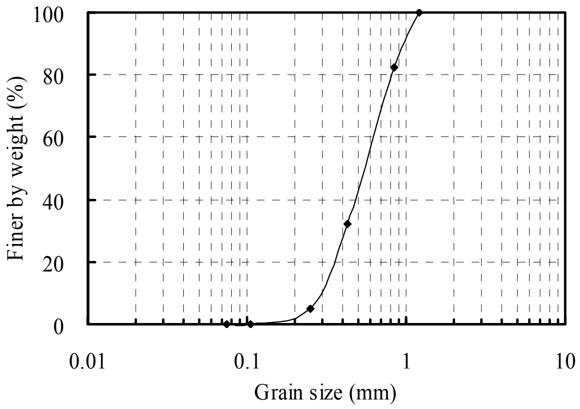

Fig.2. Grain size distribution

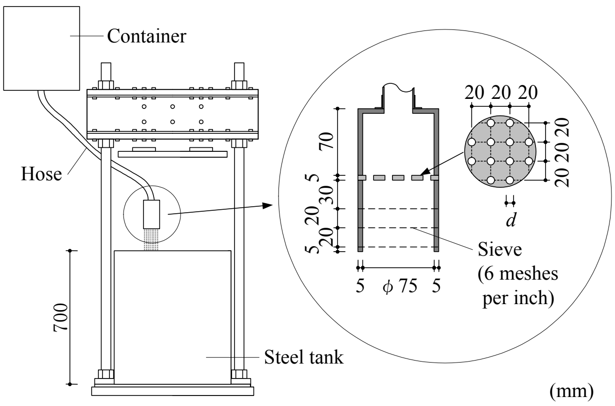

Fig.3. Hose-type sand rainer

2.2 Test results

Fig. 4 and Fig. 5 show various relationships for vertical loading pressure *pv* = 600 kN/m² as examples of the test results. Fig. 4 shows the relationship between sampler cumulative penetration and blow count during the main driving, while Fig. 5 shows the relationship between the axial strain of the soil and the cumulative penetration. The strain in the axial direction is the displacement of the pressurizing plate during main driving divided by the height of the test soil after pre-driving. From Fig. 4, it is observed that the larger the relative density, the greater is the blow count required for sampler penetration of 30 cm; it is also seen that with the increase in the cumulative penetration, the penetration per blow reduces.

From Fig. 5, it is observed that when the relative density before pressurization *Dri* is approximately 60 or 70 %, the pressurizing plate settles with the penetration of the sampler; however, when *Dri* is approximately 80 or 90 %, the soil expands during the penetration. Practically no change was observed in the value of the earth pressure gauge due to penetration of the sampler.

Table 2 shows the results of the test. It shows the *N*-value determined from the blow count when the penetration at main driving was 30 cm, and the relative density after pressurization *Drc*. Fig. 6 shows the relationship between *N*-value and *Drc*. The magnitude of the vertical loading pressure is indicated by different symbols in the figure. The figure shows that as *Drc* increases, or as pv increases, the N-value also increases. Multiple regression analysis was carried out taking Drc and pv as independent variables, and the N-value as dependent variable, then the regression equation below was obtained. The reason for taking the regression equation as a exponential function is to select a function such that the correlation coefficient becomes large.

$N = \exp(2.210 \ln Drc + 0.646 \ln pv - 10.437)$ (1)

Multiple correlation coefficient : R = 0.988

[Applicable range : pv = 150 to 600 kN/m², Drc = 65 to 100 %]

The curves in Fig. 6 are regression curves obtained by the equation above.

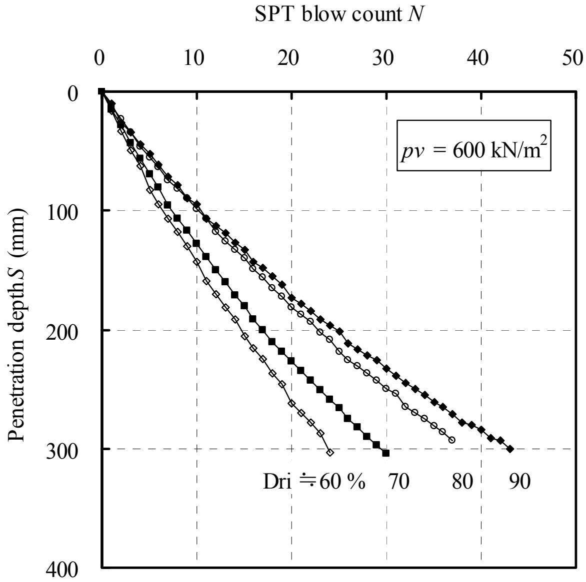

Fig.4. Relationship between sampler cumulative penetration and blow count

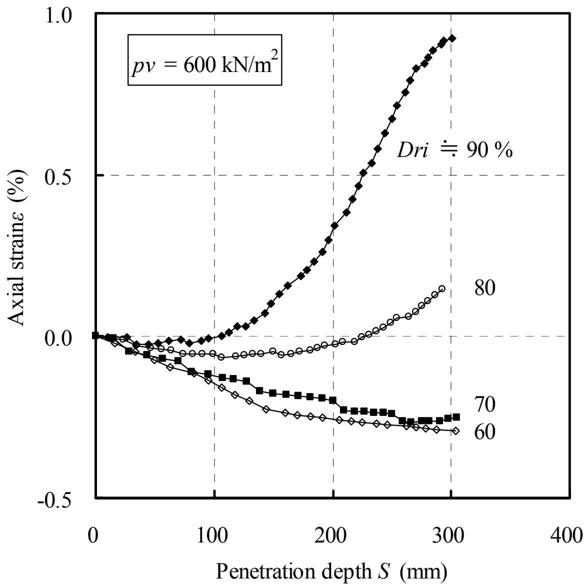

Fig.5. Relationship between axial strain of soil and cumulative penetration

Table 2. Results of Standard penetration tests

Vertical loading pressure pv (kN/m²)	Relative density (initial) Dri (%)	Relative density (after pressurization) Drc (%)	Penetration resisitance N-value
150	61.5	68.2	8.2
	69.8	74.6	10.4
	76.6	80.5	10.4
	90.1	92.0	18.8
300	60.6	69.6	15.7
	70.8	76.5	16.9
	76.2	82.1	19.5
	91.9	95.5	27.6
450	62.9	71.7	18.8
	73.5	80.5	23.1
	79.0	84.8	25.2
	91.4	95.9	38.7
600	60.8	70.9	23.7
	72.7	80.0	29.6
	87.9	91.5	37.9
	89.1	94.6	42.9

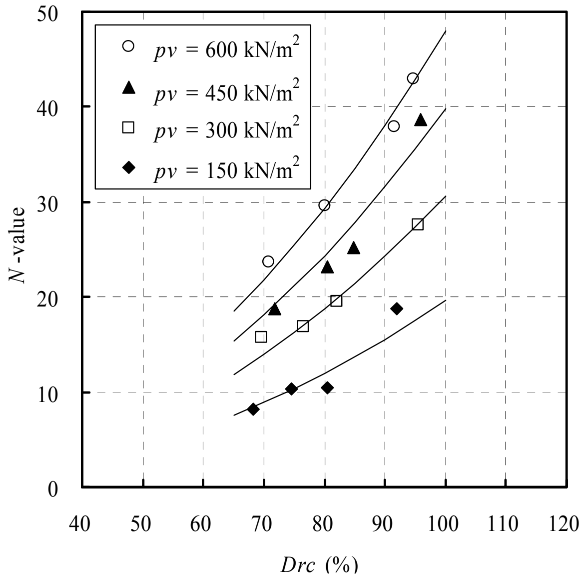

Fig.6. Relationship between N-value and Drc

2.3 Discussion

This research is compared with research conducted earlier. Table 3 shows the equation proposed in the different studies and the test conditions. Fig. 7 shows the N-value versus relative density relationship when the graph is plotted according to the proposed equation at pv=300 kN/m². TEST-NC [8] and TEST-NT are tests conducted by some of the authors in the past.

Fig. 7 shows that the results of TEST-NC and this study are quite similar. Although the size of the pressurized sand tank in these two cases is a little different, the sand grain-size distribution is practically the same, and other test conditions are also similar. For this reason, the N-value of the soil is governed by the relative density after pressurization and the vertical loading pressure, and results that resembled each other closely were obtained.

However, the results of this study and those of the research by Gibbs, H.J. and Holtz, W.G., and those by Marcuson, W.F. and Bieganousky, W.A. and Yamada et al. are very different. The reasons are considered to be as follows: i) From the research already carried out, Gibbs, H.J. and Holtz, W.G. and Marcuson, W.F. and Bieganousky, W.A., have conducted the standard penetration test sequentially at several locations in the same test soil. For this reason, the density increases gradually in loose soil, and the confining effect appears in the tank in dense soil, probably leading to an increase in the confining pressure. ii) In research already carried out, vibrator or tamper has sometimes been used to obtain the test soil of the specified relative density. For this reason, sedimentary structure different from that of the test soil prepared using sand rainer occurs, with denser soil occurring at larger depths, leading to an increase in the lateral pressure coefficient. iii) From the already conducted research, Yamada et al. have conducted tests using test soil in the triaxial pressurization condition (coefficient of lateral pressure K = 1). This is probably why confining pressure greater than in the steel tank confinement research was obtained.

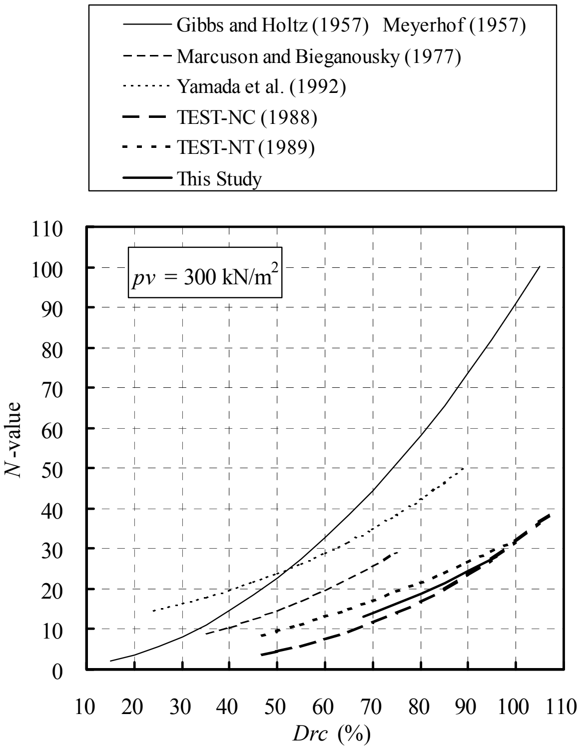

Fig.7. Relationship between N-value and Drc

3. Validation of model loading test

3.1 Assessment of soil in the model loading test

In the model loading tests [1),2)] related to pile toe grouted base of the bored piling method conducted by some of the authors, the pressurized sand tank mentioned in Section 2 was used, and vertical loading pressure pv = 600 N/m^2 was applied on the soil. For details of the model loading test, please refer to the references mentioned above. Table 4 shows the test cases used for validation. The first notation of the test case here indicates the shape of the pile body with S indicating straight pile and N indicating nodular pile. The next number indicates the diameter (unit: mm) of the grouted base. The next number indicates the length (mm, L in Fig. 8) between the pile tip and tip of the base, while the last number indicates the strength (N/mm^2) of the grout. From the strength test on grout using test pieces sampled when preparing the grouted base, it was confirmed that the grout strength was approximately close to the required strength. The Young's modulus of the grout was 5.2 to 13.9 x 10^3 N/mm^2. The relative density after pressurization of the soil Drc is 92.6 to 98.5%. The N-value of soil calculated after using Drc and equation (1) was in the range of 40.5 to 46.5.

Table 3. Comparison with previous studies

Researcher or test case	Proposed equation	Range of the test	Size of the sand tank	Confining state	Number of tests in a test soil	Mean grain size D_{50}	Compacting way of sand
Gibbs and Holtz (1957) Meyerhof (1957)	$N = 1.7 \cdot (Dr/100)^2 \cdot (0.145\,pv + 10)$ pv : kN/m^2, Dr : %	Dr = 15～105 % pv = 0～276 kN/m^2	ϕ = 90 cm H = 120 cm	Confining of steel tank	6 times	1.58 mm	Surface vibrator
Marcuson and Bieganousky (1977)	$N = -5.5 + 2.8 \cdot 0.145\,pv + 0.0046\,Dr^2$ pv : kN/m^2, Dr : %	Dr = 35～75 % pv = 69～552 kN/m^2	ϕ = 122 cm H = 183 cm	Confining of Steel ring and rubber spacer ring	3 times × 3 points	0.23 mm	Sand rainer Tamper
Yamada et al. (1992)	$N = 3.0 \cdot \exp(0.023\,Dr) \cdot (\sigma m/98.1)^{(1-0.0035\,Dr)}$ σm : kN/m^2, Dr : %	Dr = 24～89 % σm = 49～294 kN/m^2	ϕ = 50 cm H = 70 cm	Triaxial compression (K=1.0)	1 time	0.2 mm	Compaction rod Sand rainer
TEST-NC (1988)	$N = \exp(2.853 \ln Drc + 0.673 \ln pv - 13.518)$ pv : kN/m^2, Drc : %	Drc = 47～107 % pv = 49～294 kN/m^2	ϕ = 52 cm H = 100 cm	Confining of steel tank	1 time	0.45 mm	Sand rainer
TEST-NT (1989)	$N = \exp(1.759 \ln Drc + 0.699 \ln pv - 8.624)$ pv : kN/m^2, Drc : %	Drc = 47～102 % pv = 49～294 kN/m^2	ϕ = 50 cm H = 100 cm	Triaxial compression (K=0.5)	1 time	0.45 mm	Sand rainer
This Study	$N = \exp(2.210 \ln Drc + 0.646 \ln pv - 10.437)$ pv : kN/m^2, Drc : %	Drc = 68～96 % pv = 150～600 kN/m^2	ϕ = 58.4 cm H = 70 cm	Confining of steel tank	1 time	0.54 mm	Sand rainer

3.2 Bearing capacity

Fig. 9 shows the pile head load versus settlement curve obtained from the model loading tests. As shown in Fig. 8, the pile diameter *D* of the straight pile is 30 mm, while the diameter at the node of the nodular pile *Dn* is 40 mm. However, the diameter of the grouted base *De* is 64 mm regardless of the kind of pile. Therefore, the pile head load *Pou* was determined when the settlement *S* was 6.4 mm, which is 10% of *De*. This is shown in Table 4. *Pou* was divided by the cross section area *Ae* of the base to obtain the bearing capacity *Pou*/*Ae*, and is shown in Table 4. Cases where the pile head load reduced steeply, and the gradient of the load-settlement curve showed a declining trend (S-64-0-5, S-64-0-15, S-64-25-5, S-64-25-15, N-64-0-5), are cases in which the effects of damage to base have appeared[1), 2)]. Comparing Pou in S-64-0-15 and N-64-0-15, the former is small and the latter is large. This is attributed to the load transfer method between the pile side surface and the base which in the case of straight pile is adhesion and friction force, while in the case of the nodular pile, it is the resisting force from the base on the under surface of the node.[1)]

3.3 Discussion

Fig. 10 shows the relationship between bearing capacity and *N*-value from the model loading test. Fig. 10 shows also the bearing capacity of pile toe obtained from the loading test of the actual pile. This is the ultimate bearing capacity when the settlement is 10% of the pile diameter (diameter at the node for a

Table 4. Results of model loading tests

test No.	*Drc* (%)	*N*-value	*Pou* (kN)	*Pou*/*Ae* (kN/m^2)
S-64-0-5	92.6	40.5	12.1	3758
S-64-0-15	97.8	45.8	12.9	4016
S-64-25-5	97.7	45.6	14.3	4442
S-64-25-15	93.8	41.8	21.3	6627
S-64-40-15	98.3	46.3	25.6	7952
N-64-0-5	96.1	44.0	22.8	7094
N-64-0-10	96.6	44.5	26.2	8138
N-64-0-15	98.5	46.5	27.4	8502
N-64-0-20	96.9	44.9	27.7	8614
N-64-25-5	94.6	42.5	27.1	8408
N-64-25-15	94.8	42.8	27.9	8657

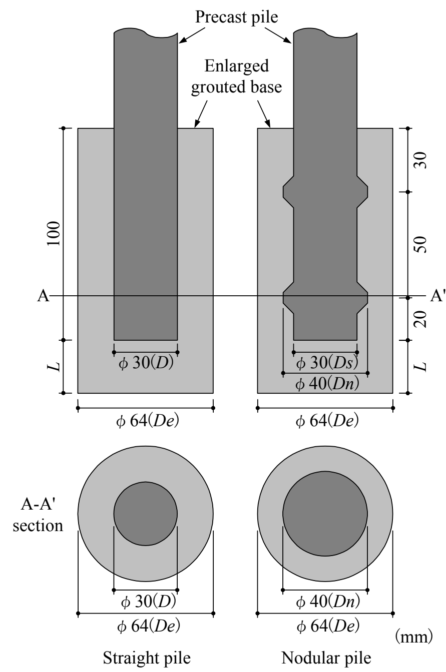

Fig.8. Section of model pile

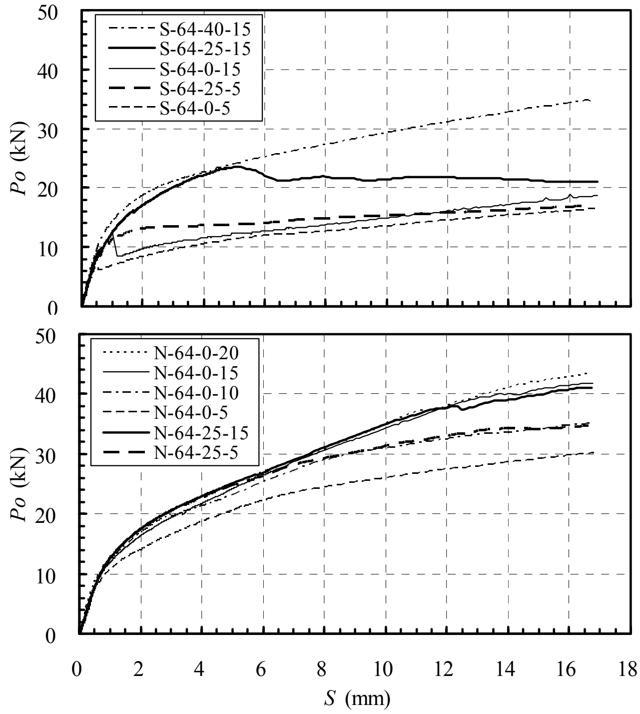

Fig.9. Pile head load versus settlement curve obtained from the model loading tests

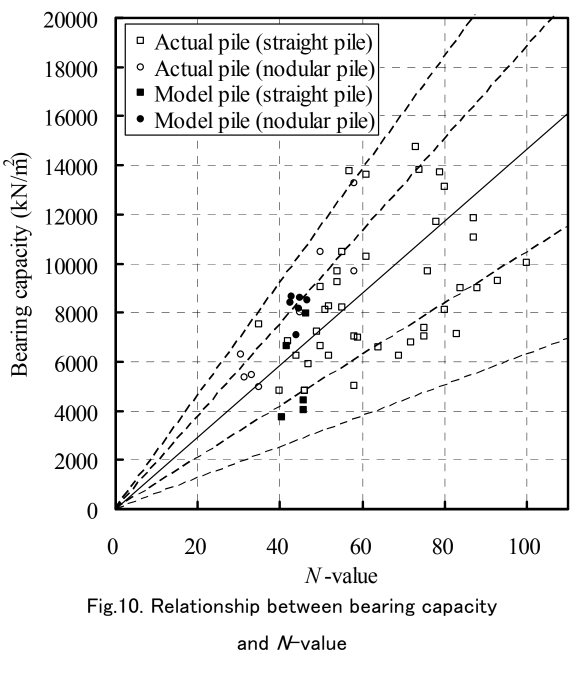

Fig.10. Relationship between bearing capacity and *N*-value

nodular pile) of the pile made by root enlarged and solidified prebored piling method (straight pile [11] and nodular pile [12]) divided by the cross section area of the grouted base. Strictly speaking, the assessment should be made by the bearing capacity when the settlement is 10% of the diameter of the grouted base, but data for making such a judgment was not available[11), 12)], so the bearing capacity at 10% of the pile diameter was used. The bold line in the figure indicates the regression line, while the broken lines indicate $\pm 1\sigma$ and $\pm 2\sigma$ (σ : standard deviation) curves.

Referring to Fig. 10, the bearing capacity from the model loading tests for cases in which the effect of damage to the base was particularly large, as in cases S-64-0-5, S-64-0-15, S-64-25-5, can be observed to approach the lower limiting value of the data of the actual pile. However, it was confirmed that the bearing capacity in other model loading tests was within the range of variation of bearing capacity obtained by the loading test on the actual pile. Compared to the real values, the results of the model loading test and the results of the standard penetration test using pressurized sand tank can be judged as valid results. If the ultimate bearing capacity is to be made the same as the bearing capacity of pile when the settlement is 10% of the base diameter, the bearing capacity of the actual pile shown in the figure may probably become slightly higher.

4. Conclusions

1) Standard penetration tests were performed using pressurized sand tank. The trend confirmed was that the greater the relative density of soil, or the greater the vertical loading pressure, the greater was the N-value. For relative density of soil in the range of 50 to 100 % and vertical loading pressure in the range of 50 to 600 kN/m^2, correlation equation with these two parameters and the *N*-value was proposed.

2) It was evident by comparing with the results of research already carried out that the results of the standard penetration test in the pressurized sand tank were affected to a large extent by the test conditions.

3) The soil used in the model loading test of bored precast pile was assessed by the relationship between *N*-value and the parameters *Drc* and *pv*, and the bearing capacity of the model pile was determined. By comparing this bearing capacity with that of actual piles, the bearing capacity from the results of model loading test and the *N*-value obtained from the results of standard penetration test using pressurized sand tank were confirmed to be valid.

Acknowledgements

The authors would like to thank Mr. Teruyuki Hamada and Mr. Masaru Nagai for their great supports in the experiments.

References

1) Ishikawa, K., Ito, A., Ogura, H. and Nagai, M. : Effect of strength and tip length of enlarged grouted base on bearing capacity of nodular pile (Model tests on enlarged grouted base of bored precast pile Part 1), Journal of Structural and Construction Engineering, Architectural Institute of Japan, Vol.76, No.670, pp.2107-2113, 2011.12 (in Japanese)

2) Ishikawa, K., Ito, A. and Ogura, H. : Effect of tip length and strength of enlarged grouted base on bearing capacity of straight pile (Model tests on enlarged grouted base of bored precast pile Part 2), Journal of Structural and Construction Engineering, Architectural Institute of Japan, Vol.77, No.676, pp.883-889, 2012.6 (in Japanese)

3) Emori, I. and Schuring, D.J. : Theory and application of model experiments, Gihodo, 1973.5 (in Japanese)

4) Gibbs, H.J. and Holtz, W.G. : Research on determining the density of sands by spoon penetration testing, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, Vol.1, pp.35-39, 1957.8

5) Meyerhof, G.G. : Discussion, Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering, Vol.3, p.110, 1957.8

6) Marcuson, W.F. and Bieganousky, W.A. : Laboratory standard penetration tests on fine sands, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol.103, GT6, pp.565-587, 1977.6

7) Yamada, K., Kamao, S., Yoshino, H. and Masuda, M. : On the influence factors for SPT *N*-Value, Tsuchi-to-Kiso, The Japanese Society of Soil Mechanics and Foundation Engineering, Vol.40, No.4, Ser.No.411, pp.23-28, 1992.4 (in Japanese)

8) Yamagata, K., Ito, A., Shimohira, Y. and Hamada, T. : Some properties of sand plugs in split spoon sampler on penetration test, The 23th Japan National Conference on Soil Mechanics and Foundation Engineering, C-3, pp.105-106, 1988.6 (in Japanese)

9) Tatsuoka, F. and Haibara, O. : Shear resistance between sand and smooth or lubricated surfaces, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol.25, No.1, pp.89-98, 1985.3

10) Miura, S. and Toki, S. : A sample preparation method and its effect on static and cyclic deformation-strength properties of sand, Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol.22, No.1, pp.61-77, 1982.3

11) Yamagata, K., Ito, A., Tanaka, T. and Kuramoto, Y. : Statistical study on ultimate point load and load-settlement characteristics of bored precast pile, Journal of Structural and Construction Engineering, Architectural Institute of Japan, No.436, pp.81-89, 1992.6 (in Japanese)

12) Ogura, H., Komatsu, G., Manabe, M., Oshima, A., Chikusa, N., Hosoda, Y., Sumi, M. and Mimura, T. : Enlarged boring diameter and vertical bearing capacity installed by root enlarged and solidified prebored pilling method of precast pile, GBRC, Vol.32, No.1, pp.10-21, 2007.1 (in Japanese)

Note

Note 1) Ishikawa, K., Nagai, M., Ito, A., and Ogura, H.: Standard penetration tests in sand confined by steel tank, Summaries of Technical Papers of Annual Meeting, Architectural Institute of Japan, B-1, pp.417-418, 2011.8 (in Japanese)

[2012 年 6 月 20 日原稿受理　2012 年 8 月 17 日採用決定]