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Method and Penetration Characteristics of Low-Noise and Low-Vibration Steel Pipe Piling (The Drill Pipe Method)

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The drill pile method is a low-noise/low-vibration piling method that a steel pipe pile which has spiral ribs and bits on both the inner and outer surfaces at the toe. The pile is twisted into the ground by means of a conventional pile driving machine, using special rotary jigs. After many applications to foundation construction, a more effective procedure has been established by improving the supporting process and instruments, including the penetration monitoring/controlling system (doctor system). The rotary penetration resistance (√Tt value) obtained by the doctor system is correlated to both the ground strength (N value) at the pile toe and to the frictional resistance acting on the inner and outer pile surfaces. The correlation between √Tt and the average N value at the bearing stratum becomes clearer when the effect of frictional resistance is eliminated by using the formulae proposed in this paper. The suitability of this construction procedure is proved from field experiments involving pull-out and horizontal load tests.

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Method and Penetration Characteristics of Low-Noise and Low-Vibration Steel Pipe Piling (The Drill Pile Method)*



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The drill pile method is a low-noise/low-vibration piling method that uses a steel pipe pile which has spiral ribs and bits on both the inner and outer surfaces at the toe. The pile is twisted into the ground by means of a conventional pile driving machine, using special rotary jigs. After many applications to foundation construction, a more effective procedure has been established by improving the supporting process and instruments, including the penetration monitoring/controlling system (doctor system). The rotary penetration resistance ($\sqrt{Tt}$ value) obtained by the doctor system is correlated to both the ground strength (N value) at the pile toe and to the frictional resistance acting on the inner and outer pile surfaces. The correlation between $\sqrt{Tt}$ and the average N value at the bearing stratum becomes clearer when the effect of frictional resistance is eliminated by using the formulae proposed in this paper. The suitability of this construction procedure is proved from field experiments involving pull-out and horizontal load tests.

1 Introduction

The "drill pile method" is a newly developed piling technique characterised by its plugging effect of soil at the pile toe by applying a pre-load to the ground while minimizing the disturbance to soil around the pile.[1,2]

The steel pipe pile has spiral ribs and bits on both the inner and outer surfaces at the toe, and is installed by rotary penetration. This method was developed as a low-noise, low-vibration steel pipe piling technique without soil discharge that can be applied to low- to medium-rise buildings with foundation piles of up to 508 mm in outside diameter. The foundation structure by this method was approved by the Japan Architectural Center in June 1990 and by the Ministry of Construction in July 1990. Up to December 1991, approximately 5 000 tons of steel pipe piles (approximately 3 700 piles) have been installed by using this method.

This paper starts with an explanation of the improvements made to construction machines, tools and installation methods under various circumstances. Second, the compaction mechanism for the peripheral soil is discussed based on data from pull-out and horizontal loading tests. Third, the data obtained by the so-called "doctor system," which controls the extent of penetration into the bearing stratum, are analyzed. Finally, a quantitative determination of the correlation between the penetration resistance value at completion by this method and the N value at the toe (hereinafter referred to as the $\bar{N}$ value) is given.

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2 Installation Results and Improvements to the Installation Efficiency

2.1 Installation Results

A schematic view of the drill pile method is shown in **Fig. 1**. Normally, a pile is rotated to penetrate the bearing stratum by using the conventional pile driving machine (base machine) equipped with an earth auger shown in **Fig. 2**. The soil of the bearing stratum to which the drill pile method can be applied is sand, sandy gravel or gravel, and a pile is, as a rule, driven into the bearing stratum to a penetration depth of more than three times the pile diameter.

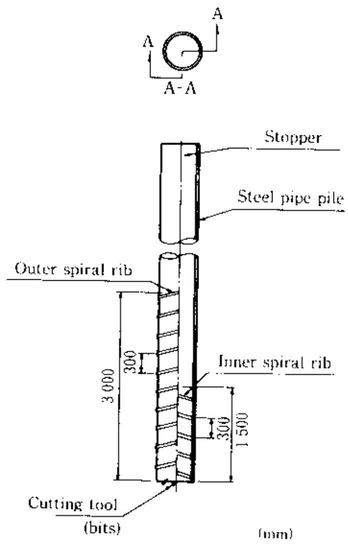

Fig. 1 Schematic view of drill pile

Table 1 Construction records of drill pile method

Construction site	Pile dia (mm)	Length (m)	Amount	Bearing stratum ($\bar{N}$ value)	Supplemental method	Execution efficiency (m/machine/d)
A	318.5	24.0	75	Sand (51)	—	105.9
B	318.5	15.5	108	Gravel (60)	Preboring (12.5 m)	98.2
C	318.5	9.5	96	Sand (45)	—	108.4
D	508.0	34.1	202	Sand (50)	Inner excavation (21.0 m)	134.0
E	508.0	12.6	799	Sand (25~50)	—	165.3
F	508.0	9.5	147	Sand (34)	—	127.0
G	508.0	18.1	829	Sand (30~40)	Preboring (15.5 m)	137.8

Table 1 shows the driving efficiency of the drill pile method obtained in recent work. The power of the equipment used was 150 hp to 240 hp for piles of 508 mm in outside diameter, and 120 hp for those of 318.5 mm. On average, the driving rate exceeded 100 m/machine/day (actual working day basis), although it varied depending on the pile diameter, length and soil conditions. Under favourable conditions, the driving rate was more than 150 m/machine/day.

2.2 Improvements to the Installation

2.2.1 Piling method and machines used

The major construction equipment used with the drill pile method is shown in **Table 2**. The earth auger and a drill pile are connected together by causing a rotary jig to engage with a stopper that is internally attached

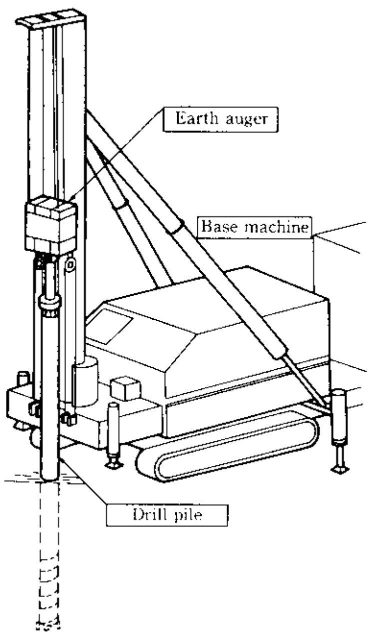

Fig. 2 Diagram of drill pile method

Table 2 Construction machine and equipment for drill pile method

Machine or equipment	Specification
Direct support type pile driver	—
Crawler crane or track crane	Crane capacity 40~60 (t)
Auger motor	90~180 (kW)
Power generator	250~500 (kVA)
Rotary jig	Pile dia ϕ 318.5~ϕ 508.0 (mm)
Follower	—
Doctor system	—

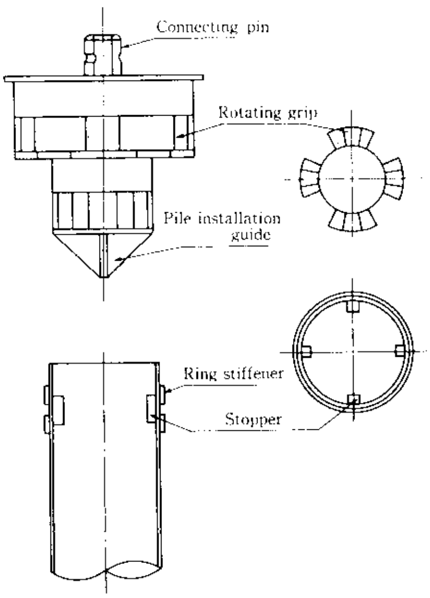

Fig. 3 Pile installation and rotation jigs

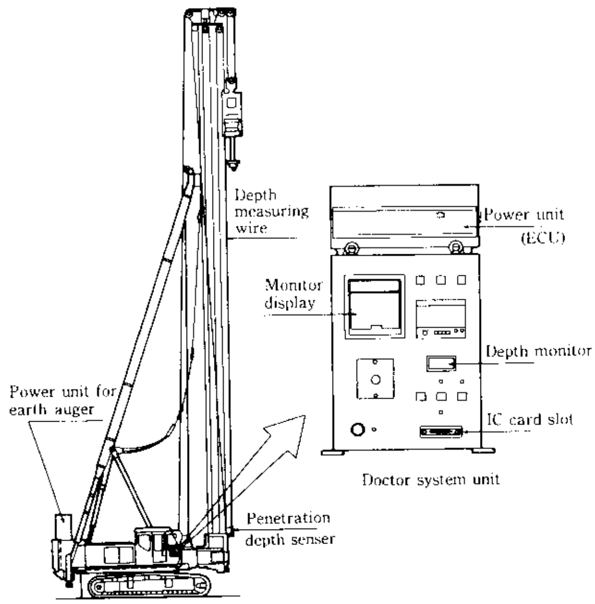

Fig. 4 Penetration control system (Doctor system)

to the top of pile as shown in **Fig. 3**. In order to apply a push-in force during rotary penetration, the earth auger is fitted with a pulley block at the bottom, and the auger is connected to the pile driver by a rope through this pulley block. When the top of a pile needs to be driven below the ground surface, rotary penetration of the pile to the desired depth can be achieved by using a follower fitted between the earth auger and the rotary jig. The maximum length of the follower ever used for actual practice is approximately 4 m, depending on the pile length and soil conditions.

Driving is done by a combination of torque, push-in force, and pull-out and push-in motions to a pile with the above-mentioned equipment and apparatus. Because of the spiral ribs attached internally at the toe of a drill pile in each section of 1.5 m, sand which tends to compact easily is taken into the pile at the surface and intermediate layers and the consequent pile plugging effect at the toe increases the penetration resistance. The pull-out and push-in motions during driving improve the penetrability at the surface and intermediate layers by moving the soil inside the pile. Using this operation, piling is continued until sand or sandy gravel in the bearing stratum is taken in. With the ordinary bored pile method, a material to harden the toe of a pile such as cement milk is poured to generate enough bearing capacity. This cement grouting operation is not necessary with the drill pile method, so that the reliability in attaining sufficient bearing capacity is higher and site conditions are cleaner.

The main variables to control when using this method are the pile material, driving equipment and associated apparatus, pile lifting, roation torque, penetration depth, accuracy and safety, the set depth being the most important of all. The doctor system shown in **Fig. 4** is used to control the penetration depth and time for driving by monitoring the load current of the auger motor and penetration time. The execution management data used is the $\sqrt{Tt}$ value (where T is the driving torque and t is the penetration time), this value being calculated by a built-in computer, and the depth of the bearing stratum and the driven length of the pile into this bearing stratum are monitored by checking the $\sqrt{Tt}$ value. This $\sqrt{Tt}$ value correlates with the N value of the soil at the pile toe,[1,3,4] it having been found that, in soil where the N value increases rapidly, for example, the $\sqrt{Tt}$ value also increases rapidly.

The feed length of chart paper used for this method, a sample of which is shown in **Fig. 5**, corresponds to the

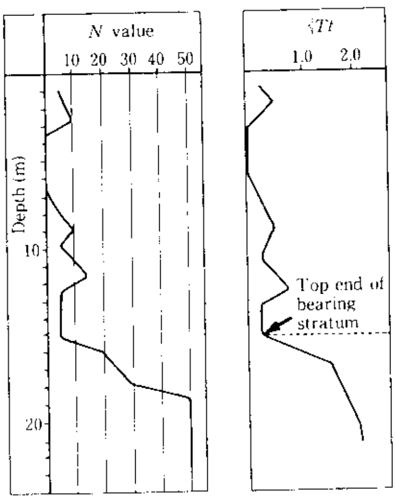

Fig. 5 Typical example of penetration control data

penetration length. The format is intended to show the correlation of the $\sqrt{Tt}$ value at each depth with a soil-boring log.

2.2.2 Employment of supplemental methods

Supplemental methods such as pre-boring and inner excavation without water are employed, using a screw auger with a diameter smaller than the pile diameter, when the surface or intermediate layers contain obstacles or hard soil. Regardless of the use of such supplemental methods, Eq. (1) is applied to calculate the vertical bearing capacity with the drill pile method.

$$R_u = \alpha \bar{N} A_p + \left(\frac{\bar{N}_s}{5} L_s + \frac{\bar{q}_u}{2} L_c \right) \varphi \quad \cdots\cdots\cdots (1)$$

where R_u: ultimate bearing capacity (tf)

α: bearing capacity coefficient at the design pile toe

$\alpha = 25$ when $L_b/d \geq 3$

$\alpha = \dfrac{25}{3}\left(\dfrac{L_b}{d}\right)$ when $L_b/d \leq 3$

L_b: set length into bearing stratum (m)

d: pile diameter (m)

$\bar{N}$: average N value between $1d$ downward and $4d$ upward from the toe bottom end of the pile, provided that N does not exceed 100 and $\bar{N} \leq 60$

A_p: cross-sectional area of the pile toe (m^2)

L_s: design length in contact with sandy soil (m)

$\bar{N}_s$: average N value for the sandy soil layer of the peripheral ground over design length L_s, provided that $\bar{N}_s \leq 50$

$\bar{q}_u$: average uni-axial compressive strength of the cohesive soil layer of the peripheral ground over design length L_s (tf/m^2), provided that $q_u \leq 10$ (tf/m^2)

φ: circumference of the pile (m)

The design pile length is defined as follows:

$L_a - 3d$ when $L_b/d \geq 3$

$L_a - L_b$ when $L_b/d \leq 3$

where L_a is the length of pile contact with the ground (m).

The penetration and bearing mechanisms for the drill pile method have already been described in detail in the previous report.[2] The desired plugging effect at the pile toe is achieved by setting the penetration into the bearing stratum to a depth of more than three times the pipe diameter. This is also effective when a supplemental method is adopted for layers above the bearing stratum or the upper soft subsoil layers. An example follows in which the design bearing capacity at the pile toe was achieved when the drill piles were driven into soil stirred by a screw auger with a diameter smaller than the pile diameter. **Table 3** shows the results of vertical loading tests. Pile T-6 in **Fig. 6** is an example of an

Table 3 Vertical loading test results

Test No.	Pile dia (mm)	Penetration depth (m)	Bearing stratum	$\frac{L_b}{d}$	Bearing capacity (tf)	
					Measured	Designed
T-1	318.5	22.0	sand	3.5	213.0	152.0
T-2	318.5	10.5	sand	12.6	230.0	175.0
T-3	508.0	18.0	sand	5.7	275.0	178.0
T-4	508.0	40.0	sand	5.9	556.0	400.0
T-5	508.0	27.8	gravel	4.5	425.0	322.0
T-6	400.0	17.1	gravel	1.7	265.0	227.0
T-7	318.5	23.0	gravel	8.2	250.0	188.0

L_b: Setting depth into bearing stratum d: Pile diameter

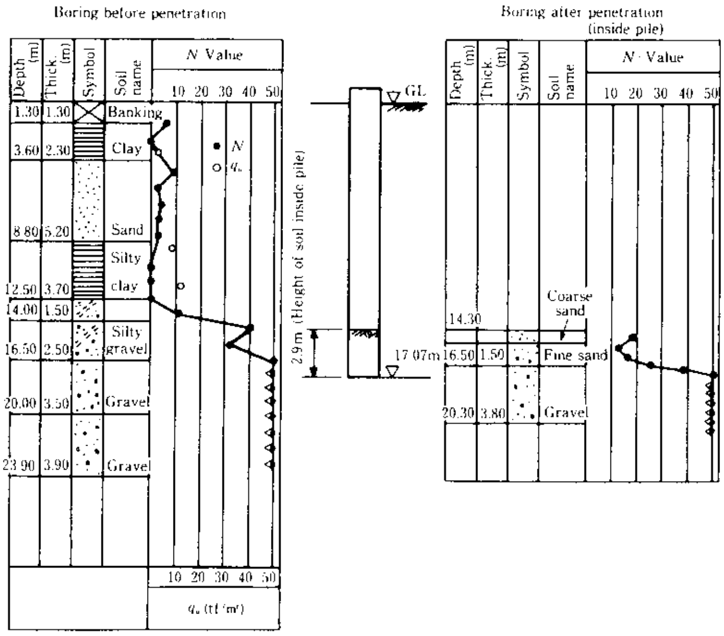

Fig. 6 Standard penetration test result of soil inside pile (ϕ400 mm, T-6)

experiment conducted to examine whether the pile toe is sufficiently plugged, even when the intermediate layer of the bearing stratum is soft subsoil and when the set depth into the bearing stratum is not more than $3d$. Although the set depth into the bearing stratum is 1.7 times the pile diameter, the point bearing capacity of pile T-6 was not less than $25\bar{N}A_p$, as shown in Table 3, and it was ascertained that the pile toe was sufficiently plugged with sandy gravel. Similarly, with pile T-4, for which supplementary inner excavation was adopted in the intermediate layer, the point bearing capacity was not less than $25\bar{N}A_p$. From these results, the prescribed equation for bearing capacity is applicable when firm penetration into a bearing stratum is ensured, even if the soil layer on the bearing stratum is poor subsoil and is stirred in a narrow band by the screw auger. The inner excavation adopted for pile T-4 (ϕ508 mm) was conducted up to a position 2 m above the pile toe by using a screw auger with an external diameter of 400 mm. The skin friction resistance with pile T-4 was more than $N/5$ in sandy soil and more than $q_u/2$ in cohesive soil.

3 Verification of the Penetration Characteristics

3.1 Penetration Characteristics

The spiral ribs and bits attached to the inner and outer surfaces at the pile toe have the function of not only improving penetrability, but also generating the bearing capacity. During penetration, the bits bite the ground, and the outer spiral ribs push up the soil near the pile toe to improve the penetrability.[2] This improvement in penetration by the outer spiral ribs might be attributed to a friction cutting effect. In fact, however, the spiral ribs on the outer surface have a screw effect. The spiral ribs on the inner surface, in contrast, provide resistance to the movement of soil inside the pile, reducing the penetrability. The soil supplied by the outer spiral ribs fills the gap between the outer wall of the pile and the peripheral soil, and is compacted. The soil inside the pile, whose movement is restricted by the inner spiral ribs, is compacted as the pile penetrates promoting firmly blockage of the pile toe.

The foregoing characteristics have been confirmed by model experiments and vertical loading tests.[2] The next section verifies that the pull-out and push-in motions during execution do not disturb the peripheral soil nor affect the plugging effect.

3.2 Verification by Loading Tests

3.2.1 Results of a pull-out test

A pull-out test was conducted on two piles, each with an outside diameter of 318.5 mm, as shown in **Fig. 7**. The test was carried out in accordance with "multi-cycle loading method A" of the vertical loading guide line of Japanese Society of Soil Mechanics and Foundation Engineering (JSSMFE).[5]

Both piles were driven into the sandy layer with pull-out and push-in motions, and without any supplemental method. The driving time was 80 min for pile TP-1 and 33 min for TP-2, and the elapsed times after driving were 24 day and 16 day, respectively. The relationship between the load and displacement measured at the pile head is shown in **Fig. 8**, and the results are shown in **Table 4**, the designated values being obtained from the skin friction resistance by using bearing capacity Eq. (1). In this test, the deadweight of both the pile and the soil inside the pile was subtracted from the pull-out load.

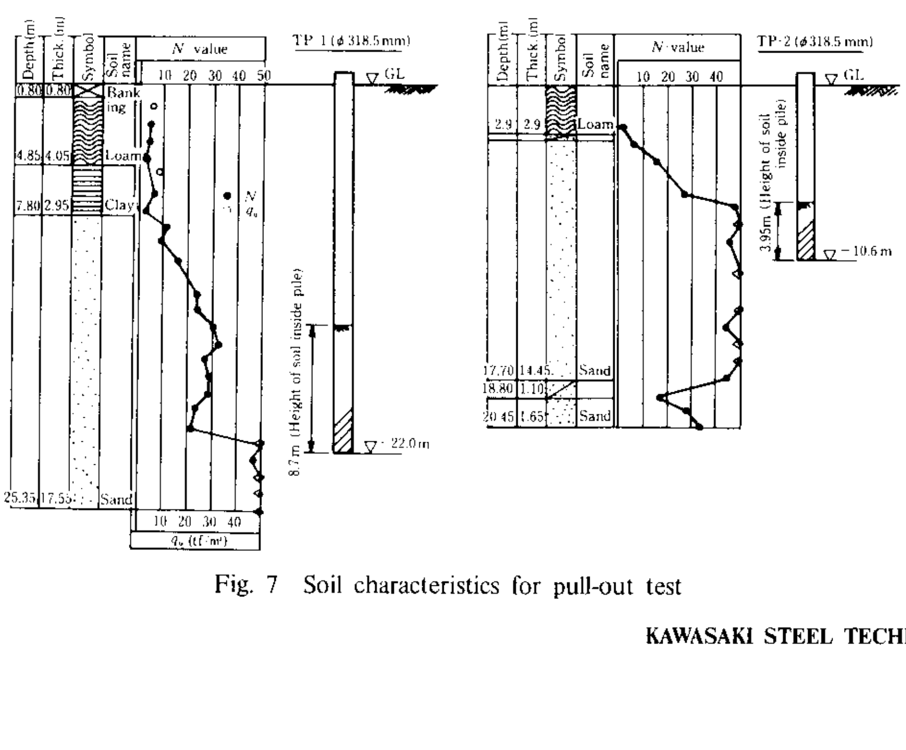

Fig. 7 Soil characteristics for pull-out test

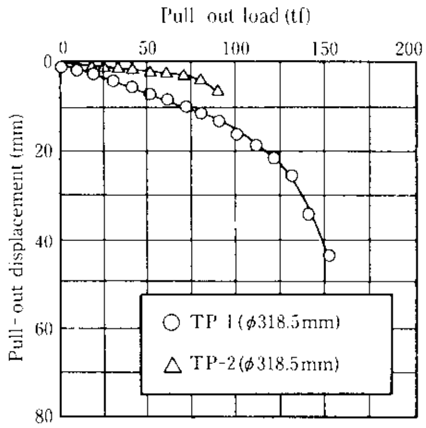

Fig. 8 Pull-out load vs displacement curve at pile head

Table 4 Pull-out test results

Test No.	Pile dia (mm)	Penetration depth (m)	Pull-out capacity (tf)	
			Measured	Designed
T-1	318.5	22.0	147.7	94.3
T-2	318.5	10.5	88.5	52.6

The measured values are about 1.6 times the design values for both piles.

3.2.2 Results of a horizontal loading test

A horizontal loading test was conducted on a pile 508 mm in outside diameter and 9 mm in wall thickness. The soil boring log and driving conditions for the test pile are shown in **Fig. 9**.

The pile penetrated through about a 20 m sandy layer with pull-out and push-in motions, and without using any supplemental method. The driving time was 35 min, and the elapsed time after driving was 26 day.

A unidirectional loading test (tension) was conducted in accordance with the horizontal loading test method of JSSMFE,[6] using a center hole jack (100 tf, capacity and 200 mm stroke). In addition to measuring the load and displacement at the test pile head (17.5 cm above the ground surface) and the displacement of the reaction pile head, the inclination angle of the pile was measured with an inclinometer for each 50 cm length of the pile under the maximum load of each loading cycle and at each zero load stage, using a square pipe (75 mm □) fitted inside the test pile beforehand.

Hysteresis curves of pile-head load H and pile-head displacement y are shown in **Fig. 10**. The relationship between horizontal pile-head displacement y and the coefficient of horizontal subgrade reaction k_h obtained by a reverse calculation based on Chang's equation is

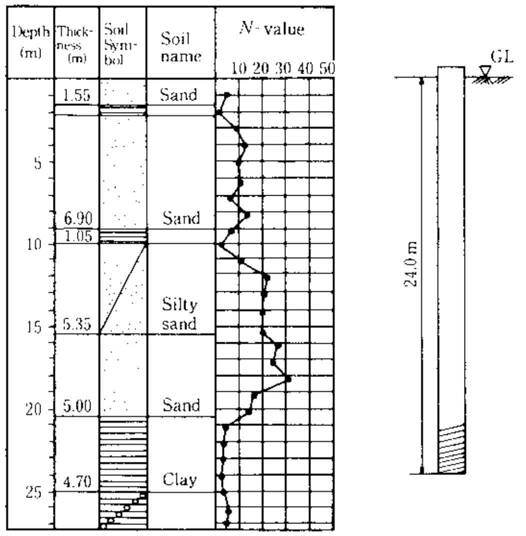

Fig. 9 Soil characteristics for horizontal loading test (ϕ508 mm)

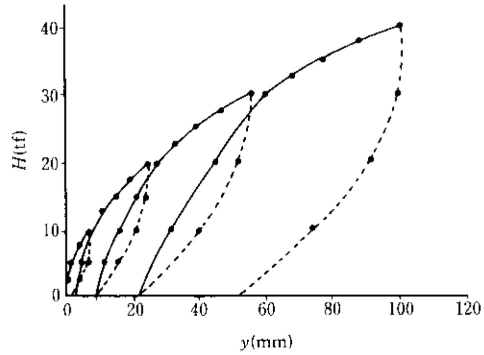

Fig. 10 Horizontal load H vs displacement y curve

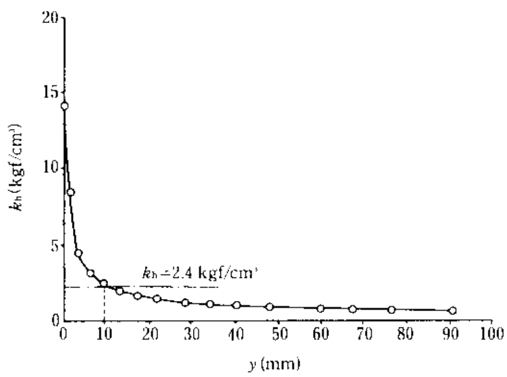

Fig. 11 Coefficient of horizontal subgrade reaction k_h vs pile head displacement y

shown in **Fig. 11**. This figure shows that the value of k_h was 2.4 kgf/cm^3 when $y = 1.0$ cm. This value is high compared with that estimated from the N value ($N = 4.7$, $k_h = 1.4$ kgf/cm^3).

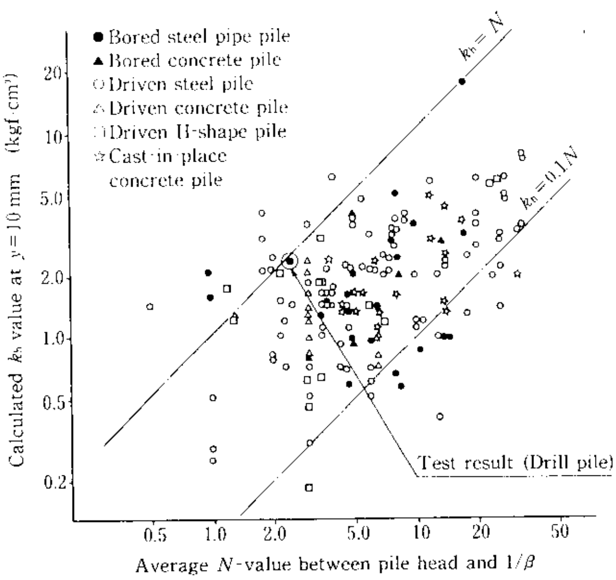

Fig. 12 Relationship between N-value and k_h

3.2.3 Analysis

It was ascertained that a pull-out resistance higher than the skin friction obtained from the design vertical bearing capacity after Eq. (1) was achieved in spite of the disturbance to the soil caused by the pull-out and push-in motions. Based on these test results, it was decided that the friction coefficient to be used with the drill pile method for sandy soil would be $N/5$, and $q_u/4$ for cohesive soil as specified for the driven piles under "Guidance for Foundations of Buildings Resistant to Earthquake Force"[7] of Japan Architectural Center.

The horizontal subgrade reaction is high in the initial stage of loading; this shows that the horizontal resistance of the soil around the pile is promoted positively. In other words, these results further support the effective compacting action to the soil around the pile, which was ascertained with the model test and other loading tests. **Figure 12** shows the relationship between the average N value (between the pile head and $1/\beta$) and the value of k_h evaluated by reverse calculation for a pile-head displacement of 1.0 cm in comparison with the results obtained by other piling methods.[8] The drill pile is comparable to other methods such as impact-driven piles and cast-in-situ concrete piles, with which a relatively large horizontal subgrade reaction can be obtained.

4 Improvements to the Driving Control Technique

With the drill pile method, the penetration into a bearing stratum is controlled by comparing the $\sqrt{Tt}$ value, which represents the penetration resistance obtained during rotary penetration, with the $\bar{N}$ value and soil conditions in the ground at the pile toe.

Analysis was conducted on the relationship between $\sqrt{Tt}$ and the penetration resistance in the intermediate layer at each pile diameter, and its correlation to the $\bar{N}$ value was evaluated.

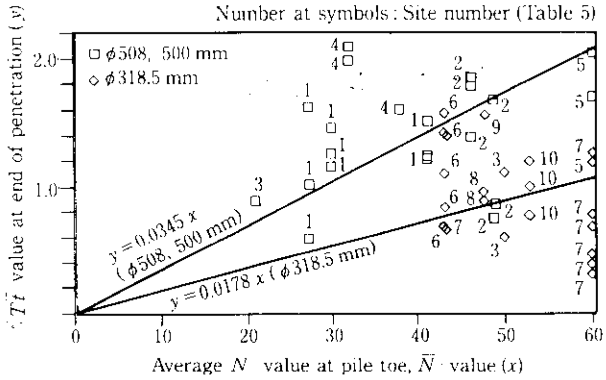

Fig. 13 Relationship between $\sqrt{Tt}$ value at end of penetration and average N-value at pile toe

Table 5 Drill piles for analysing the relationship between $\sqrt{Tt}$ and N-value

Site	Pile dia (mm)	Length (m)	Bearing stratum
1	508	13	Fine sand
2	508	9.5	Fine sand
3	508 and 318.5	18 and 44	Fine sand
4	508	26 and 27.8	Gravel
5	500 and 318.5	22 and 23	Gravel
6	318.5	9.5	Gravel
7	318.5	15 and 15.5	Silty sand
8	318.5	22	Fine sand
9	318.5	22	Fine sand
10	318.5	10.5	Fine sand

Figure 13 shows the relationship between $\sqrt{Tt}$ at penetration and $\bar{N}$ at the ten construction sites shown in **Table 5**. The data show that is impossible to link the $\sqrt{Tt}$ value directly with a characteristic value of bearing capacity such as $\bar{N}$. The reason why the correlation of these two parameters is weak is that $\sqrt{Tt}$ is influenced not only by the resistance at the penetration front, i.e., at the pile toe, but also by both the skin friction resistance along the whole length of pile outer surface and the friction resistance between the soil inside the pile and the inner wall of the pile. Therefore, correlation between $\sqrt{Tt}$ and $\bar{N}$ would be stronger if the effects of these two types of the skin friction resistance could be excluded from the $\sqrt{Tt}$ value in the bearing stratum. To achieve this, an analysis was done by classifying the penetration resistance in the bearing stratum and that in the intermediate layer (**Fig. 14**). The penetration depth of the drill pile into the bearing stratum is more than three times the pile diameter, so that a position corresponding to three times the pile diameter above the pile

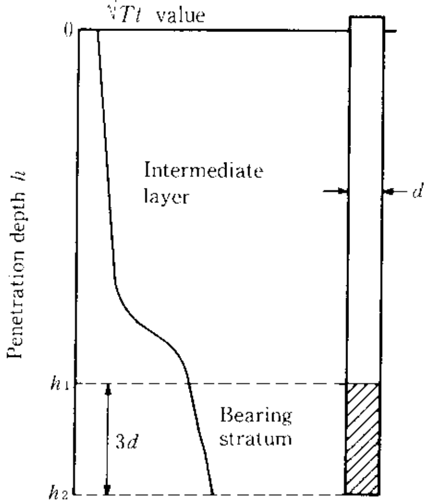

Fig. 14 Classification of bearing stratum and intermediate layer

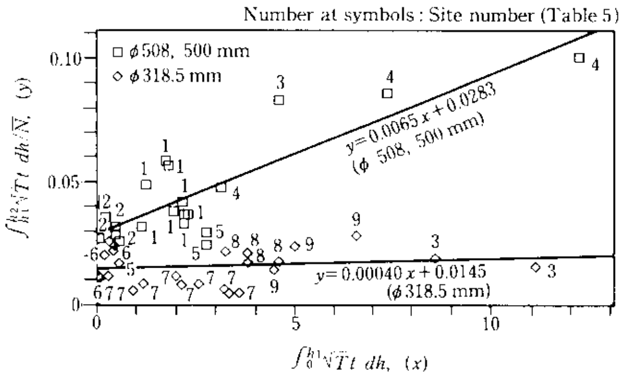

Fig. 15 Relationship between penetration energy at bearing stratum $\left(\int_{h_1}^{h_2}\sqrt{Tt}\,dh/\bar{N}\right)$ and at intermediate layer $\left(\int_0^{h_1}\sqrt{Tt}\,dh\right)$

toe was set as the top of the bearing stratum.

The energy required for penetration can be expressed by integrating the $\sqrt{Tt}$ value along the penetration length. **Figure 15** shows the relationship between the penetration energy in the bearing stratum and that in the intermediate layer, each obtained by integrating the $\sqrt{Tt}$ value. The relationship between the average $\sqrt{Tt}$ value per unit length in the bearing stratum and the $\sqrt{Tt}$ value at the end of penetration (in the 30 to 40 cm section immediately before the completion of penetration) is also shown in **Fig. 16**. The correlation found for each pile diameter by the least squares method is indicated in this figure, and can be represented by Eqs. (2) and (3) as follows:

$$\frac{\int_{h_1}^{h_2}\sqrt{Tt}\,dh}{\bar{N}} = \beta\int_0^{h_1}\sqrt{Tt}\,dh + \gamma \quad \cdots\cdots\cdots\cdots\cdots(2)$$

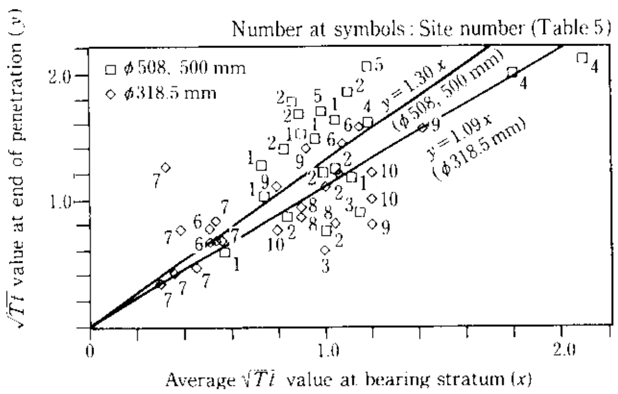

Fig. 16 Relationship between average $\sqrt{Tt}$ at bearing stratum and $\sqrt{Tt}$ at end of penetration

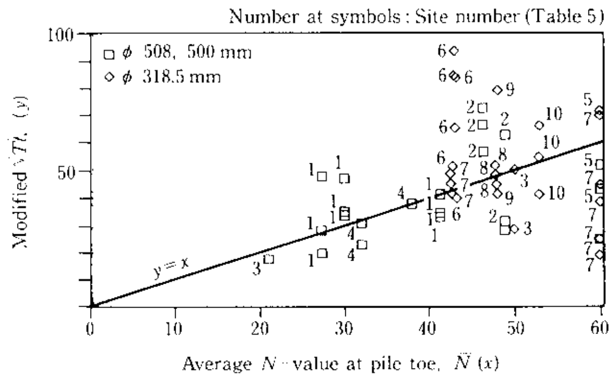

Fig. 17 Relationship between modified $\sqrt{Tt}$ and average N-value at pile toe

$$\sqrt{Tt}\ \text{(at driving completion)} = \frac{\delta}{3d}\int_{h_1}^{h_2}\sqrt{Tt}\,dh \cdots(3)$$

where T: rotational torque (tf·m)
t: penetration time per unit length (h/m)
h_1, h_2: depth at pile toe (Fig. 8)
β, γ, δ: constants that depend on the pile diameter

It is apparent from Fig. 15 that the effect of skin friction resistance inside and outside a pile in the intermediate layer increases with increasing pile diameter, resulting in increased penetration resistance of the bearing stratum. Figure 16 shows the degree of increase in the penetration resistance of the bearing stratum; the degree of this increase tends to be higher with larger pile diameter. In other words, the larger the pile diameter, the more slowly will the soil plug the pile, with the result that the longer the penetration length, the higher the degree of increase. Equation (4) is derived from Eqs. (2) and (3). The $\sqrt{Tt}$ value is modifid into the left side of Eq. (4), and corresponds to $\bar{N}$ value at the pile toe. **Figure 17** shows the validity of foregoing analysis by comparing the $\bar{N}$ value estimated from Eq. (4) with actual $\bar{N}$ of the bearing stratum.

$$\frac{3d\sqrt{Tt}\ \text{(at driving completion)}}{\delta\left(\beta\int_0^{h_1}\sqrt{Tt}\ dh+\gamma\right)}=\bar{N}\ \cdots\cdots(4)$$

This analysis was conducted to find a way of estimating the $\bar{N}$ value, which is an index of the point bearing capacity, from the $\sqrt{Tt}$ value, which is an index of the penetration resistance.

The accuracy could be increased by classifying layers more finely and making allowance for such factors such as the soil properties and capacity of the construction machine.

5 Conclusions

The results have been given of a study on improvements to the drill pile method and driving control techniques to clarify the penetration mechanism. The results obtained can be summarised as follows:

(1) With the method and the apparatus concerned both established, a driving performance of 100 m/machine/day or over is made possible, with even 150 m/machine/day or over achieved under good operating conditions.

(2) Supplemental methods such as pre-boring and inner excavation are used when the surface or intermediate layer contains obstacles or hard soil. If a pile is firmly set into the bearing stratum by applying the supplemental method with a screw auger smaller than the pile diameter, it is possible to obtain the same skin friction force and point bearing capacity as that with a pile driven by the conventional piling method.

(3) The penetration mechanism, involving spiral ribs attached to the inner and outer surfaces at the toe of a drill pile to minimize disturbance to the soil around the pile and to compact the soil, was ascertained from the results of vertical and horizontal loading tests.

(4) The data from the doctor system that is used for driving control were analyzed. It was found that the rotary penetration resistance at driving completion (modified $\sqrt{Tt}$ value) and $\bar{N}$ can be increased by excluding the friction resistance acting on both the inner and outer surface of a pile.

The authors envisage to improve execution methods and related jigs (rotary jigs, and jigs for supplementary execution, etc.) so as to reduce construction periods and improve economic performance. Also, in light of recent trend toward larger buildings and structures, increasing civil engineering works in urban areas, and application to areas of limited space and heights, development efforts will be made to attain the full-scale commercialization of larger piling diameters and smaller execution machines.

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