ROTARY PENETRATION STEEL PIPE PILE  
(DRILL PILE) METHOD
—New Low-Noise, Low-Vibration Piling Method—

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ABSTRACT

The rotary penetration steel pipe pile method (called the drill pile method here) is a new type of low-noise/low-vibration piling method which emplaces thin-walled steel pipe piles by rotary penetration but without discharging displaced soil. The drill pile method also achieves a high skin friction resistance by minimizing soil disturbance around the pile. As penetration continues, spiral ribs located on the inner wall of the drill pile provides the inner soil cylinder with sufficient resistance to prevent further soil intrusion, resulting in pressure on the ground under the pile toe. Therefore, the drill pile has the characteristics of a displacement pile. The setting depth into the bearing stratum can be checked by measuring the penetration data (e.g. penetration torque, penetration speed) in real time at the construction site.

Scaled model tests clarified the penetration/bearing capacity mechanism of the drill pile. The compaction effect of peripheral soil and the soil plugging behavior inside the pile were confirmed by filed experiments at various sites with different bearing strata. The skin friction and point bearing capacity of the pile were separately evaluated through vertical loading tests, and an adequate bearing capacity formula was derived.

Key words: low-noise/low-vibration, no soil discharge, plugged soil, rotary penetration, soil inside pile, spiral rib, steel pipe pile (IGC: K7)

INTRODUCTION

The installation of piles into soil can be broadly categorized into displacement and non-displacement method.

The displacement pile method frequently involves driving the pile with a steady succession of blows on the top using a pile hammer. While this is a relatively fast installation method and a high skin friction and point bearing capacity is achieved, this method generates considerable noise and local vibrations which may be forbidden by local regulations or environmental agencies and, of course, may damage adjacent property.

Drilling a hole which is filled with concrete to form the pile after hardening is relatively free from noise and vibration, and causes virtually no soil displacement. However, the problems associated with this method are: a) high possibility of insufficient cleaning and removal of loose soil or slime at the hole bottom which contributes to large settlement (Yamagata, 1980), b) disturbance of the peripheral soil which reduces skin friction resistance (JSSMFE, 1992), c) uncertainty in controlling the setting depth, d) storage and disposal of displaced soil.

Our rotary penetration steel pipe pile method, called the drill pile method here, is a new type of low-noise, low-vibration method that attains the high skin friction and point bearing capacity of the displacement pile but avoids the problems associated with the non-displacement pile. The various technical aspects of the drill pile method are explained here including: —physical structure of the drill pile and a brief description of the pile installation equipment, —analysis and evaluation of the penetration and bearing capacity mechanisms using scaled-model experiments, —evaluation of the skin friction and point bearing capacity through vertical load tests, and —derivation of a proposed bearing capacity formula based on these tests and analyses.

OUTLINE OF PILING METHOD

Photo. 1 shows the structure of the drill pile used by this method. Spiral ribs of 13 mm diameter steel rod are

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welded to both the inner (vertical length of 1.5 m) and outer (vertical length of 3.0 m) circumferential surfaces of the pile toe. To facilitate penetration into the ground, cutting bits are provided. The pile head is held with a rotary device which drills the pile into the ground to penetrate the bearing stratum.

The soil around the pile toe is pushed up by the spiral ribs during pile rotation and then accumulates around the pile at the top end of the rib, increasing the radial earth pressure around the pile and generating a high skin friction resistance. Since the ribs are only present at the pile toe, the displaced soil remains underground contributing to clean execution of work with no discharged soil requiring temporary storage and disposal.

The inner spiral ribs promote compaction of the soil inside the pile at the bearing stratum at the final penetration stage. This increases the friction between the inner wall of the pipe and the soil plug, contributing to a point bearing capacity which is about that of a closed-end pile. In an open-end pile, the point bearing force is due to the friction between the inner wall of the pipe and the compacted soil along a length of 2-3 times of the pile diameter from the pile toe. The length of the inner rib portion, 1.5 m corresponds to three times the pile diameter of 500 mm.

General-purpose piling machines such as the tripod type pile driver fitted with an earth auger was used to rotate and drill the steel pipe pile into the ground in this method (Fig. 1). Since the chucking of the pile to the earth auger can be done remotely using a special rotary jig, greater safety is assured without working at heights in pile erection and splicing.

This method can confirm the existence of the bearing stratum at penetration by monitoring the penetration time per unit length and auger current value (penetration torque) (Nishizawa et al., 1990) allowing control of the setting depth.

**SCALE-MODEL EXPERIMENTS**

**Outline of Experiment**

Drill piles with a length of 1500 mm, outside diameter of 114 mm, and wall thickness of 4.5 mm were used as test piles in experiments with and without ribs, and using different installation methods (rotary penetration pile/press-in pile) as test parameters. A rod of 3.2 mm diameter was welded spirally on both the inner and outer surface of the pile toe at a pitch of 68 mm, vertical length of 128 mm for the outside and 640 mm for the inside. Cutting tools of the same thickness as the rib were installed on two points of the outer periphery of the pile tip.

Kashima quartz sand No. 6 was hot-air dried (moisture content 0.5% or less) and then used for the model ground. The characteristics of the soil used are shown in Table 1. The model ground was formed by vibration compaction of successive layers of 10 cm thickness in a round soil tank (Hashimoto et al., 1989) with an inside diameter of 968 mm and height of 1.8 m. The relative density of the model ground was between 85% and 94% in all tests.

<table>
<thead>
<tr>
<th>Soil particle Diameter</th>
<th>( D_{50} )</th>
<th>0.16 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniformity coefficient</td>
<td>( U_c )</td>
<td>1.6</td>
</tr>
<tr>
<td>Maximum void ratio</td>
<td>( e_{\text{max}} )</td>
<td>92%</td>
</tr>
<tr>
<td>Minimum void ratio</td>
<td>( e_{\text{min}} )</td>
<td>61%</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>( G_s )</td>
<td>2.679</td>
</tr>
</tbody>
</table>

Table 1. Characteristics of soil used in scaled-model experiments
A colored layer of about 1 mm thickness was placed over each 10 cm thick layer to monitor ground movements during pile penetration.

The setting depth was five times the pile diameter in all tests. For the rotary penetration pile, the push-in force (max. 25.5 kN) was gradually increased with the penetration depth by a hydraulic jack while rotating the pile at 4 rpm using a hydraulic motor. For the press-in pile, static penetration was made at 1 cm/min measured at the ground surface by a hydraulic jack. During rotary pile penetration, a surcharge of 0.098 MPa was applied by air pressure through a 32 mm thick steel plate.

Penetration/Bearing Capacity Mechanism

After rotary penetration of the model pile with inner and outer ribs was completed, the ground around the pile was carefully removed and the surface observed. Dense soil particles adhered to the entire outer surface of the pile, as shown in Fig. 2. The colored layer in the ground was distributed in the upper adhered layer. This shows that the soil particles near the pile were displaced upward along the outer ribs due to the rotary penetration of the pile.

Figure 3 shows the compacted condition of the ground around the pile after installation compared with the original ground. The compaction was evaluated by reading the load required to make a 5 mm penetration (cone index) using a pocket cone penetrometer with the end modified into a flat head (12.8 mm diameter). Before each measurement at the desired depth, the upper sand layer overlying the measurement surface was removed.

The measured value, therefore, includes the effect of stress relaxation due to the removal of the upper load soil.

![Fig. 2. Adhering of soil to outer periphery of pile due to rotary penetration](image)

*The numerical values in the figure correspond to the ratio of cone index of compacted ground to that of original ground.*

![Fig. 3. Comparison of the effect of pile installation methods on compaction of peripheral ground](image)
The rotary penetration pile achieved compaction over the entire pile length especially on the peripheral surface of pile. The peripheral ground initially disturbed due to the rotary penetration was compacted by the soil particles displaced upward from the outer rib, and the radial earth pressure around the pile became higher than the initial value. The increase in cone index of the ground at the pile toe was not so great for the rotary penetration pile as for the press-in pile. The experiments confirmed that for the rotary penetration piles, penetration occurred when applying pressure continuously on the ground under the pile toe (Hashimoto et al., 1989), and compaction based on the advanced pressure effect can be expected.

Figure 4 shows the comparative effect of inner spiral ribs on the rise of the soil inside piles installed by the rotary penetration method. In all cases, the rate of rise of the soil inside the pile decreased as the penetration proceeded. The decrease in the rate of rise was remarkable for the pile with inner ribs. As seen in previous test results (Kobayashi and Yamakawa, 1982), the inner ribs increase the shear resistance of the inner surface of the pile and constrain the upward slippage of the soil plug, promoting compaction of the soil inside the pile and facilitating the strong blockade at the pile toe.

FIELD EXPERIMENTS

Field experiments using actual piles were conducted to quantitatively evaluate the penetration/bearing characteristics particular to the drill pile.

Survey on Peripheral Soil and Soil inside Pile

Figure 5 shows the SPT (Standard Penetration Test) N-values around the settled pile at distances of 30 cm and 50 cm from the pile surface compared with the values of the original ground before pile penetration. Using a drill pile with outside diameter 406.4 mm and wall thickness 16 mm, this test evaluated the disturbance of the peripheral ground, particularly the decrease in N-value, due to the pile penetration. However, there was no decrease in the strength (N-value) of the peripheral ground and higher N-values than the values for the original ground were obtained in the upper sand layer.

To investigate the condition of the soil plug, a pile 508 mm in diameter was rotary penetrated into the gravel bearing stratum then pulled out immediately and cut in half. Photo 2 shows the soil plug inside the pile end. The formation of a very dense soil plug consisting of gravel of the bearing stratum inside the pile was confirmed.

Vertical Load Test

Table 2 shows the specifications of 7 vertical load tests conducted at different sites. The bearing strata were sand (4 sites) or gravel (3 sites). Figure 6 shows the whole $P_0$(pile head load)-$S_0$(pile head settlement) curves.

(1) Skin Friction

To evaluate the skin friction resistance quantitatively, axial strain gages were attached to the test piles T-3, T-4, T-5 and T-6. Figures 7 and 8 show the changes in skin friction measured at several positions along the pile length, against the relative displacement of the pile and ground, that is the settlement of each part of the pile in the sandy soil and cohesive soil strata, respectively. Figures 9 and 10 show the relationship between the original ground strength ($N$-value or $q_u$) and skin friction of the piles in the sandy soil and cohesive soil strata, respectively, when the pile toe settlement reaches 10% of the pile diameter ($d$) (or at maximum load for piles which did not reach 10%$d$ settlement).

Figures 7 and 8 indicate that the maximum skin fric-
tion was demonstrated at almost all locations when the relative displacement of the pile and peripheral ground reaches 10 to 30 mm. The skin friction values at 10%Δd settlement were distributed around N/0.51 kN/m² (N/5

![Fig. 7. Changes in skin friction with relative displacement of pile and ground (sandy soil)](image)

![Fig. 8. Changes in skin friction with relative displacement of pile and ground (cohesive soil)](image)

![Fig. 9. Relationship between skin friction and ground N-value at 10%Δd settlement (sandy soil)](image)

### Table 2. Specification of vertical load test piles

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Pile diameter (mm)</th>
<th>Wall thickness (mm)</th>
<th>Setting depth (mm)</th>
<th>Type of bearing stratum</th>
<th>Ratio of depth into bearing stratum to the pile diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>T-1</td>
<td>318.5</td>
<td>6.9</td>
<td>22.0</td>
<td>Sand</td>
<td>3.5</td>
</tr>
<tr>
<td>T-2</td>
<td>318.5</td>
<td>10.3</td>
<td>10.0</td>
<td>Sand</td>
<td>12.6</td>
</tr>
<tr>
<td>T-3</td>
<td>508.0</td>
<td>9.0</td>
<td>18.0</td>
<td>Sand</td>
<td>5.7</td>
</tr>
<tr>
<td>T-4</td>
<td>508.0</td>
<td>16.0</td>
<td>40.0</td>
<td>Sand</td>
<td>5.9</td>
</tr>
<tr>
<td>T-5</td>
<td>508.5</td>
<td>9.0</td>
<td>27.8</td>
<td>Gravel</td>
<td>4.5</td>
</tr>
<tr>
<td>T-6</td>
<td>400.0</td>
<td>16.0</td>
<td>17.1</td>
<td>Gravel</td>
<td>1.7</td>
</tr>
<tr>
<td>T-7</td>
<td>318.5</td>
<td>12.7</td>
<td>23.0</td>
<td>Gravel</td>
<td>8.2</td>
</tr>
</tbody>
</table>

![Fig. 6. P₀(pile head load)-S₀ (pile head settlement) curves](image)
tf/m²) or more in the sandy stratum for both ordinary and spiral rib length of the piles, and \(q_u/2\) kN/m² or more in the cohesive soil stratum for the ordinary length of the piles.

(2) Point Bearing Capacity

In an open-end pile, friction occurs between the soil inside the pile and the inner wall of the pile when loaded, this inner friction affects the vertical strains at the pile end portion and the friction on the outer surface. Therefore, it is difficult to evaluate the outer friction (skin friction) and inner friction (part of the point bearing capacity) separately by strain gauges attached to the pile. The drill pile is more complicated because the spiral ribs are attached to the inner and outer surfaces of the pile toe. Therefore, we set a design pile toe at the position of 3d upward from the real pile toe to evaluate the point bearing capacity (Fig. 12(a)). In this way, the outer frictional force occurring under the design pile toe elevation is involved in the point bearing capacity. The value of 3d corresponds to the minimum installation length of the inner spiral ribs which cause the soil plugging.

Figure 11 shows the relationship between the load \(Q_u\) and settlement \(S_p\) obtained at the design pile toe. The bearing capacity coefficient \(\alpha\) is obtained by dividing the design pile toe resistance by the average \(N\)-value and pile toe plug cross section. In this figure the design pile toe of test pile T-6, in which the setting depth into the bearing stratum did not reach 3d, was taken as corresponding to the top of the bearing stratum. Figure 11 shows that the bearing capacity coefficient \(\alpha\) was 245 or more when the settlement of the design pile toe reached 10%d, and 300 or more at the ultimate bearing capacity stage when the settlement became very large in comparison with the increase in load.

**Method of Calculating Vertical Bearing Capacity**

Formula (1) was derived from the scaled model tests and the field load tests for designing the vertical bearing capacity. Figure 12 schematically shows the design concept of the bearing capacity of the drill pile.

\[
Ru = \alpha N A_p \left( \frac{N_l}{0.51} \cdot \frac{L_d + \bar{q}_u}{2} \cdot L_c \right) \phi
\]  

(1)

where

- \(Ru\): ultimate bearing capacity, (kN)
- \(\alpha\): bearing capacity coefficient of design pile toe (end bearing capacity evaluation position)
  
  \[
  \alpha = \begin{cases} 
  245 & \text{when } L_b/d \geq 3 \\
  \frac{245}{3} \left( \frac{L_b}{d} \right) & \text{when } L_b/d < 3 
  \end{cases}
  \]
- \(L_b\): setting depth into bearing stratum, (m)
- \(d\): pile diameter, (m)
- \(\bar{N}\): average value of \(N\)-values of ground between 1d downward and 4d upward from bottom end of pile, provided that the maximum \(N\) value is 100 and \(N \leq 60\)
- \(A_p\): plug cross section of pile toe, (m²)
- \(L_s\): length in contact with sandy soil, (m)
- \(N_l\): average \(N\)-value of sandy soil layer of peripheral ground over the length \(L_s\), provided \(N_l \leq 50\).
- \(L_c\): length in contact with cohesive soil, (m)
- \(\bar{q}_u\): average uniaxial compressive strength of cohesive soil layer of peripheral ground over the length \(L_c\), (kN/m²)
- \(\phi\): circumference of pile, (m)

The design pile length is as follows:

\[
L_a = 3d, \quad \text{when } L_b/d \geq 3
\]
\[
L_a = L_b, \quad \text{when } L_b/d < 3
\]
where, $L_a$: length of pile in contact with ground, (m).

Figure 13 compares the measured value and calculated value of the bearing capacity using formula (1) for the 7 vertical load tests shown in Fig. 6. As a mean of the 7 tests, the measured value is about 1.35 times the calculated value. Formula (1), therefore, yield conservative bearing capacity values for safety. Comparing these results with the evaluation of the bearing capacity for conventional low-noise/low-vibration piles, such as caissons reinforced with cement milk or bored concrete piles, shows that the vertical bearing capacity of the drill pile is equivalent to or higher than that of conventional low-noise/low-vibration piles.

CONCLUSION

The results of the scaled-model experiments and actual field tests proved that the drill pile compacts the peripheral ground during pile rotation, especially sandy soil. The vertical bearing capacity is equivalent to or higher than that of conventional low-noise/low-vibration piles.

The drill pile method has great potential for application to installation of pile foundations particularly in urban areas where low-noise, low-vibration piling methods and a clean piling environment are desirable or necessary.

REFERENCES