Technical Report

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Development of New-type Steel-pile Method for Port Facilities

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Abstract

A pile driving method with small noise and small ground vibration is needed in port area, because the noise and ground vibration caused by the hammer are problematic for neighboring industrial plants and residential area. In addition, there is a tendency that the scale of port structures is bigger, the water depth of quays is deeper and the structure system is getting rationalized. Because of these situations, Nippon Steel & Sumitomo Metal Corporation has developed on the vibratory pile driving with water and cement milk jetting. This "RS Plus™" method can reduce the ground vibration and noise, and has possibility to get large bearing capacity. This paper shows the construction process of this method, the evaluation of the bearing capacity in the range of maximum 1 600 mm diameter piles.

1. Introduction

Due to the progress of redevelopment projects in waterfront areas in recent years, there is a tendency for residential areas and plants to be located adjacent to port facilities. For this reason, impact hammer driving of steel pipe piles using hammers in port facilities as conventionally employed has been increasingly difficult in the face of issues associated with noise and vibration. In addition, with continued increases in quay water depth and in the size of port facilities mainly in subject governmental strategy aimed at strengthening the international competitiveness of port facilities, there is demand for the rationalization of structures from the viewpoint of economic efficiency. Therefore, the bearing capacity required per steel pipe pile is becoming larger, resulting in an increase in the size of piles.

Against this background, Nippon Steel & Sumitomo Metal Corporation has developed the RS Plus™ method in collaboration with the Port and Airport Research Institute and Chowa Kogyo Co., Ltd.[1, 2] This is a steel pipe pile construction method in which soil cement block using soil cement is formed using a vibratory hammer together with a water jet (hereafter referred to as "WJ") and cement milk jet (hereafter referred to as "CJ"). The use of a Vibro hammer makes it possible to construct a soil cement block that realizes high bearing capacity, while generating less vibration and noise than that generated using the impact hammer driving method. The RS Plus™ method was first introduced in the market in 2009 with its applicable maximum pile diameter up to around 1 000 mm. The maximum pile diameter to which the method is applicable was further expanded by 2014 to 1 600 mm.

A key point in this construction method is the technology to construct robust soil cement block. Partly because there is a report stating that it is difficult to gain bearing capacity by the vibratory pile driving as the ground is softened by this method when this method is applied in conjunction with WJ,[3] it is necessary to strengthen soil cement block in order to achieve high bearing capacity. For this reason, in the development, emphasis has been placed on clarifying a soil cement block formation phenomenon when using WJ and evaluating the bearing capacity derived. This paper outlines the construction method and the development results that form the technical background, which is followed by the introduction of application examples.

2. Outline of the Construction Method

2.1 Construction procedures

Figure 1 shows the processes in this method. First, a pile is put into a bearing layer up to a depth approximately three times the length of the pile diameter *D* using a Vibro hammer and a WJ together (Fig. 1 ① – ③ , and the ground in the vicinity of the pile tip is sufficiently excavated. After that, the WJ is switched to a CJ and the pile is vibrated using the Vibro hammer and is raised until the pile tip reaches the top surface of the bearing layer (Fig. 1 ④ – ⑤),

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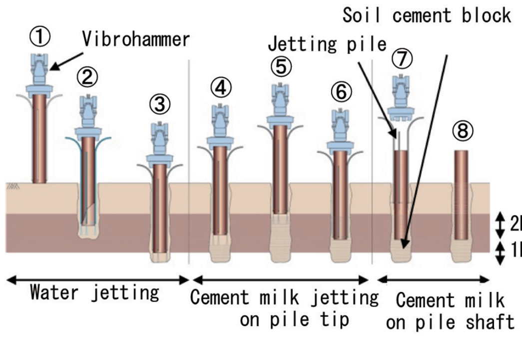


Fig. 1 Construction process

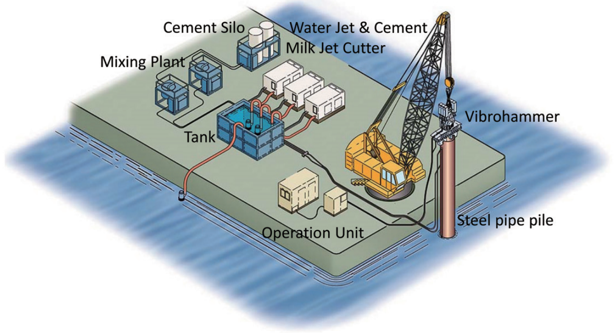


Fig. 2 Construction facilities

which is followed by sinking the pile down again to a depth of 2 D from the top surface of the bearing layer (Fig. 1 ⑥). With an up-and-down movement of the pile using CJ together in this way, the ground and cement milk are stirred and mixed, forming a soil cement block. When skin friction is needed, cement milk is applied to the steel pipe surface while pulling out the jet pipe (Fig. 1 ⑦).

As shown in **Fig. 2**, construction facilities used for this construction method consist of a water and cement milk jet cutter for the WJ and CJ, a mixing plant, and others in addition to a Vibro hammer and a crane required for pile driving. While Fig. 2 is a schematic view of construction facilities on the sea using a barge, work on the land along a quay is of course possible.

In order to produce homogeneous soil cement block, the amount of cement milk to be fed by controlling the flow rate of cement milk fed by pressure, the pressure used for the jetting, and the timing of switching from the WJ to CJ are specified in accordance with the diameter of the steel pipe pile used. Furthermore, the depth and rate of pile driving are controlled with a laser displacement meter attached to the Vibro hammer.

2.2 Pile structure

The following two types of pile tip structures can be used according to the application conditions.

(1) Outer rib plate type

As indicated in **Fig. 3**, in this system, rib plates are radially attached to the outer peripheral surface of a steel pipe, and nozzles of the WJ and CJ are placed on the pipe tip peripheral surface (hereafter referred to as the “outer rib plate type”). This system is suitable for pipes with small and medium-sized diameters of 1 000 mm maximum, and the length of the rib plates in the horizontal direction is set up such that the area of the soil cement block base becomes about two times closed pile area. Bearing capacity can be obtained

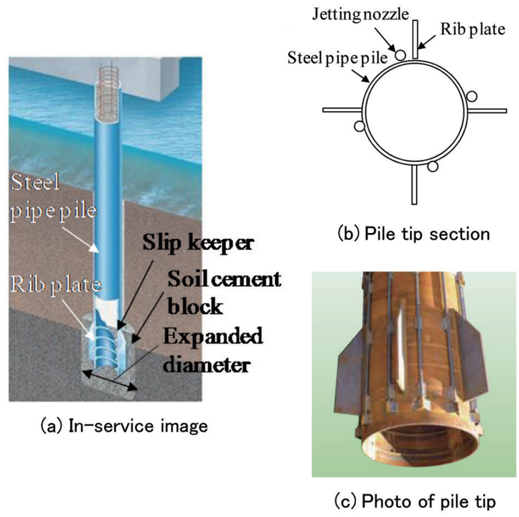


Fig. 3 Outer rib plate type

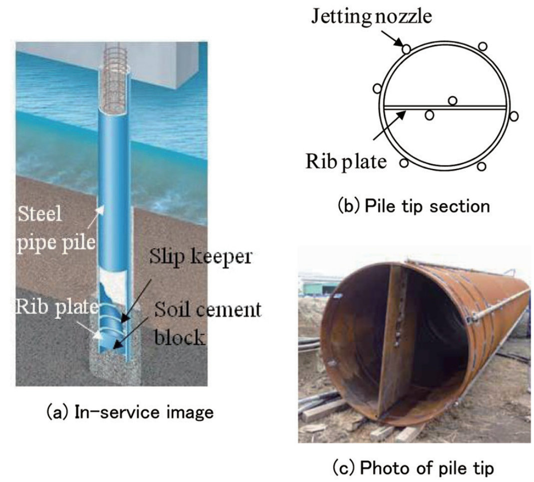


Fig. 4 Inner rib plate type

by forming a large soil cement block relative to the pile diameter.

(2) Inner rib plate type

As shown in **Fig. 4**, this system has a rib plate attached on the inner surface of the pipe such that the plate partitions the inner space of the pipe, and also such that nozzles are placed on both the pipe peripheral surface and the rib plate (hereafter referred to as the “inner rib plate type”). Since this system allows for construction of a robust soil cement block even for a large-diameter pile with a pipe diameter exceeding 1 000 mm, the use of WJ and CJ using the nozzles attached to rib plates makes it possible to excavate and stir the soil in the position of the pile center directly.

Both types are provided with multiple rows of slip keeper on the inner peripheral surface of the pipe tip to prevent the soil cement from moving, so as to secure the attachment of the pipe and soil cement.

3. Development

In order to strengthen the soil cement block, it is necessary to appropriately adjust the injection pressure and flow rate of the jets to the steel pipe diameter and soil conditions. For this purpose, we have to clarify the phenomenon whereby ground is excavated by

WJ. Therefore, we have decided to take the following steps: (1) understanding such phenomenon through experiments using models, then (2) verifying the constructability of a soil cement block through tests using mock-ups on a full scale, followed by (3) clarifying the bearing capacity performance through the static load test. A detailed explanation of steps (1) to (3) is given as follows.

(1) Clarification of excavated soil range excavate by a WJ

Two experiments are conducted using models [4]: an experiment for basic soil excavating by a WJ with a single nozzle; and an experiment simulating the RS Plus™ method construction using multiple nozzles and a steel pipe. To start, an experiment using a single nozzle is conducted. In this experiment, saturated sand ground is prepared inside a sand box that is set to a centrifugal loading equipment, and the nozzle is arranged along the side face acrylic resin wall of the sand box as shown in **Fig. 5** (a). The overburden pressure applied is varied as Cases 1–4 as shown in **Table 1** by changing the centrifugal acceleration (1–40 G) in order to observe the relationship between the overburden pressure and the excavated range. An example of the experimental results is shown in **Photo 1**. The range of excavating by the WJ is the white range in the shape of a teardrop under the nozzle hole. It is reported [5] that when large energy is applied to sandy soil by a WJ, an increase in the excess pore water pressure will cause the effective stress to decrease, which weakens the bond between soil particles, thus facilitating excavating. Moreover, based on this principle, jet penetration distance *R* can be derived using injection pressure *P* and the flow rate (nozzle diameter d_0), and the soil resistance (such as total stress σ_z and angle of shearing resist-ance ϕ') and fluid loss coefficient Λ.

Figure 6 in which Cases 1–4 are plotted shows the experimental results summarized using dimensionless quantities consisting of respective parameters, using the above-mentioned finding in a past report as a reference. Cases 1–4 are plotted on a straight line, and the applicability of the finding in the past report is high within the scope of this experiment. While the finding in the past report [5] was verified by field experiments up to 8 m in depth (overburden pressure: approx. 150 kPa), this experiment has verified in a centrifugal field that the finding is also applicable to cases with a larger overburden pressure of 213 kPa. Additionally, in all of the cases, the maximum excavation width shown in Photo 1 becomes approximately 0.6 times the excavated distance. Moreover, also in all of the cases, the excavated range takes the shape of a teardrop. Accordingly, knowing jet penetration distance *R* allows the excavated range to be obtained.

Next, by an experiment simulating construction employing the RS Plus™ method, an examination is conducted as to whether the excavated range that is sufficient to form a soil cement block area can be obtained using a WJ inside a sand box with saturated sand ground under 1G. As shown in Fig. 5 (b), nozzles are attached to both ends of the steel pipe tip of the test piles for Cases 5–8 in Table 1, simulating the outer rib plate type. As shown in **Fig. 7** (a), in Cases 5–7 using piles with relatively smaller diameters, the excavated ranges formed by the WJ from both ends of the steel pipe pile overlap each other, thus resulting in excavating the entire part of the ground just below the steel pipe pile.

However, a sufficient excavated range is not obtained in Case 8 with a large-diameter pile in which the area of the ground to be excavate is larger. Therefore, in Case 9, simulating the inner rib plate type, a triple-nozzle WJ is formed with nozzles arranged on the pile center line in addition to both ends of the steel pipe as shown in Fig. 5 (c). Furthermore, in Case 10, each WJ nozzle located at both ends of the pipe is inclined and oriented inward so that the injection directions of both nozzles focus on the center.

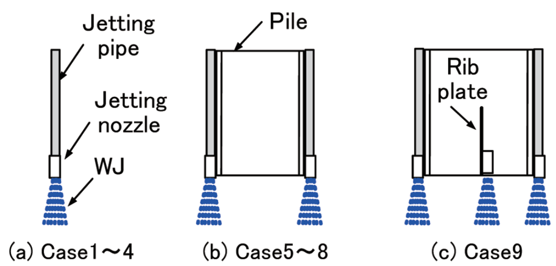


Fig. 5 WJ position of the experiment

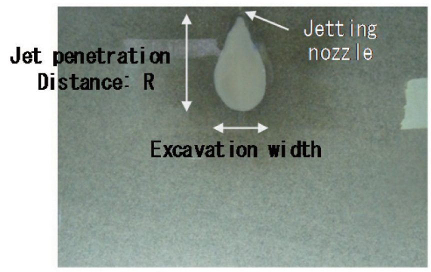


Photo 1 Sample of the centrifuge jetting experiment

Table 1 Cases of water jetting experiment

		Pile diameter (mm)	Nozzle diameter (mm)	Injection pressure (MPa)	Soil total stress (kPa)
Centrifuge (1–40 G)	Case1	-	1.0	0.69	1.2
	Case2	-	2.0	0.45	45.9
	Case3	-	2.0	0.56	103.1
	Case4	-	2.0	0.85	213.4
Sand box (1 G)	Case5	101.6	1.0	1.0–2.5	10.1
	Case6	101.6	1.3	1.0–3.0	10.1
	Case7	216.3	1.0	1.5–3.0	10.1
	Case8	318.5	2.0	0.3–1.5	10.1
	Case9	318.5	2.0	0.3–2.0	10.1
	Case10	318.5	2.5	0.3–1.8	10.1

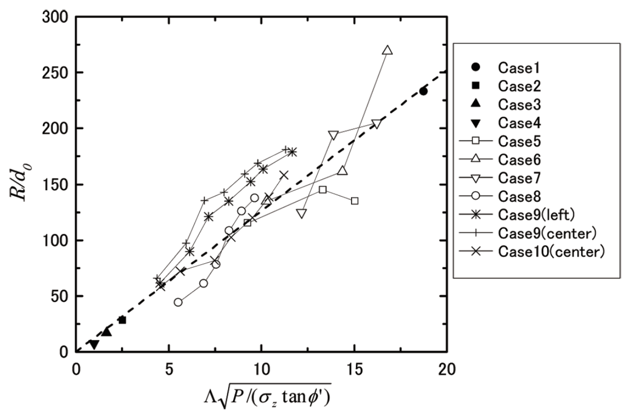
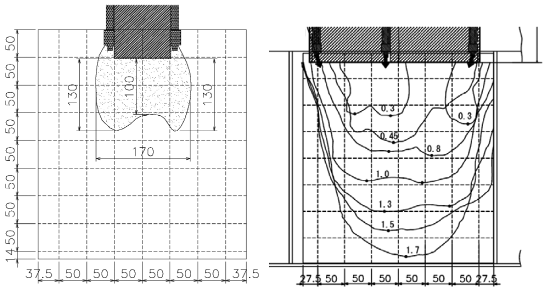


Fig. 6 Relationship amoung the WJ specification, soil resistance and jet penetration distance



(a) Case5_ 2.0MPa (b) Case10

Fig. 7 Sample of the sand box jetting experiment

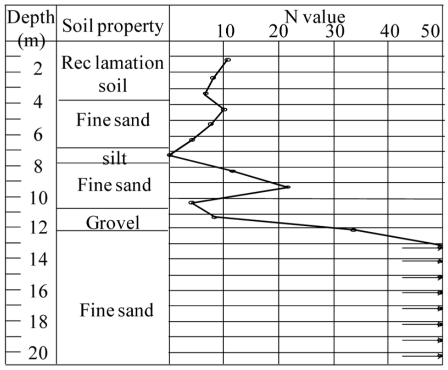


Fig. 8 Soil condition

Photo 2 Photo of turned pile (D: 1 000 mm)

As a result, the excavated ranges formed by WJ nozzles overlap each other in these cases, allowing the sufficient excavated ranges indicated in Fig. 7 (b) to be obtained. In Fig. 6, the results obtained from Cases 5–10 are also plotted. As these results show a proportional relationship with the indexes on a horizontal axis and vertical axis, the applicability of the finding in the past report to these cases is also considered to be high. In addition, a shift to a higher position than the dotted lines of Cases 1–4 is observed in most of Cases 5–10 with an increase in the injection pressure even in the same case. This suggests that the excavated distance is long relative to the ground and WJ conditions. From this, it is possible that the excavating is facilitated by a decrease in ground resistance in the middle of the adjacent excavated ranges when the distance between such ranges is reduced.

The results described above show that the applicability of the above-mentioned finding in the past report is high within the scope of these model experiments, and also that the inner rib plate type is better suited to large-diameter piles. In order to make use of these findings from our experiments for actual construction work, a full-scale soil cement block construction test is conducted as the next step.

(2) Verification of soil cement block constructability by full-scale tests

With the aim of establishing a construction method for soil cement block, (i) review of the construction process, (ii) confirmation of the shape of the finished soil cement block, and (iii) confirmation of the soil cement block in the bearing layer are implemented.

(i) Review of the construction process

A review is conducted for the construction process in a middle layer with an N value (as prescribed in JIS) of less than 50 on the premises of the Yawata Works of Nippon Steel & Sumitomo Metal. [1] The ground of the test site contains gravel and hard slag, creating relatively difficult conditions in which to work. The steel pipe pile used is a steel pipe pile (outer rib plate type) of 800 mm in diameter, and the outer diameter formed by the rib plates is 1 200 mm (1.5 *D*). The main items investigated in this test are listed below along with the test results.

- Necessity of pipe tip excavation: When the depth to which the pipe is put is set shallow in the process of Fig. 1 ③ and when moving to the process of Fig. 1 ④ without putting the pipe deeper than the final depth by a length of 1 *D* after the pipe tip reaches the final depth, a sufficient soil cement block is not formed. In this case, there is concern that the pile will not exhibit the predetermined bearing capacity. Therefore, it is important to put the pile into the ground deeper than the fixing depth by a length of 1 *D*, and also to pour cement milk directly from the bottom end of the planned soil cement block.
- Frequency of the up-and-down movement of the CJ: Just a single up-and-down movement of the CJ as shown in Fig. 1 ④ to ⑥ is sufficient, as no significant difference is observed in the soil cement block between the case of one up-and-down movement and two, while the injection amount of cement milk is kept the same.
- Nozzle arrangement and horizontal injection inside the pipe: When an outer-rib- plate-type pipe is used, arranging nozzles near a rib plate makes the cement milk spread along the rib plate, facilitating diameter expansion of the soil cement block. Furthermore, when water is injected into the inside of the pipe in the horizontal direction in addition to the WJ in the vertical direction, excavating and stirring within the pipe are facilitated, making it easy for the soil cement to form a blockage inside the pipe in the subsequent CJ process.

(ii) Confirmation of the shape of the finished soil cement block

Next, the possibility of constructing a soil cement block in conformity with an increase in the size of steel pipe diameter is examined using piles with pipe diameters of 1 000 mm and 1 600 mm. In both cases, tests are carried out in the middle layer of the ground in the premises of the R & D Laboratories of Nippon Steel & Sumitomo Metal (Futtsu City, Chiba Prefecture). The ground is mainly composed of sandy soil as shown in **Fig. 8**, and is relatively easy to work. The following have been confirmed: (a) regarding the steel pipe pile of the outer rib plate type with a pipe diameter of 1 000 mm, soil cement block is formed so as to cover the rib plates outside the pipe as shown in **Photo 2** [1]; and (b) regarding the steel pipe pile of the inner rib plate type with a pipe diameter of 1 600 mm, the space inside the steel pipe is entirely filled with soil cement, showing a favorable blockage state. [2]

(iii) Confirmation of a soil cement block in the bearing layer

Finally, the piles with their tips driven into the bearing layer are confirmed. Instead of digging up these piles, the soil cement block length, the homogeneity of the soil cement, etc., are examined through boring investigation of the inside of the steel pipes.[4] As an example, the results of a steel pipe pile with a pipe diameter of 1 300 mm are described. The test site is in the premises of the R & D Laboratories as described above. The pile is the inner rib plate type, and its nozzle arrangement and hole diameters are set as shown in **Fig. 9**. The hatched areas in the same figure indicate the supposed excavation area by the WJ considered in view of the findings obtained as described in the previous section. Through the boring investigation inside the steel pipe after the driving work, core samples as shown in **Photo 3** are obtained from a depth deeper than the pipe tip by a length of 1.5 *D*, indicating that the soil cement block length satisfies the predetermined value of 1.0 *D*. Moreover, as shown in **Photo 4**, it is confirmed using an X-ray-CT device that the core cross sections have little variation in density and that homogeneity is high as a whole.

From the above-described findings, it is confirmed that soil cement block can be formed using this method at full scale. It is also confirmed that further application of the findings and a construction procedure derived from the model experiments described in the previous sections are feasible.

(3) Evaluation of the bearing capacity performance

Regarding the bearing capacity performance, a design formula is proposed after evaluating it based on a static loading test.[6] The test cases are shown in **Table 2**. The test sites are in two locations: one in the premises of the R & D Laboratories as mentioned above, and the other in the vicinity of the R & D Laboratories. The characteristics of the bearing layer of the two sites (at least 12.3 m deep as shown in Fig. 8) are considered to be the same, and this test is deemed to be a comparison made for the same ground. In addition to the RS Plus™ method (Cases A–C), a pile driven by the impact hammer driving method[7] (Case D) is also included in the test for making a comparison.

(i) Pile end resistance force

The relationship between pile end load and pile tip displacement is shown in **Fig. 10**. In this test, the tip resistance force in Cases A–C is defined as the axial force calculated using a strain gauge attached in the position above the pipe tip by a length of 2 *D* (soil cement block top end), while the tip resistance force in Case D is defined as the axial force in the position above the pipe tip by a length of 5 *D* (bearing layer top end). As can be seen from Fig. 10, there is a gradual increase in the tip resistance force as the pile diameter increases in Cases A–C, and a comparison between Case B and Case D with the same diameter indicates that the tip resistance force of a pile driven using the RS Plus™ method is clearly greater than that of a pile driven using the conventional impact hammer driving method.

Next, whether the failure mode in limit resistance is caused by the fracture of the ground or the fracture of the soil cement block is

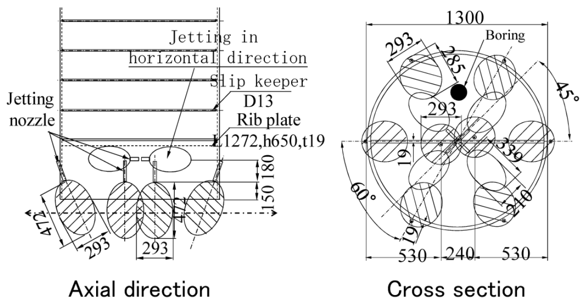


Fig. 9 Position of jetting nozzle and supposed excavation area

Photo 3 Soil-cement core bored from pile

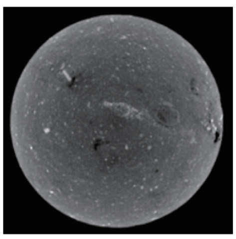


Photo 4 X-ray-CT result of the core

Table 2 Test condition and main result of the static load test

Case		Case A	Case B	Case C	Case D
D (mm)		600	800	1 300	800
Driving method		RS plus™	RS plus™	RS plus™	Impact hammer
Pile type		Outer rib	Outer rib	Inner rib	-
Pile edge depth		G.L. −18.0 m	G.L. −18.0 m	G.L. −16.0 m	G.L. −15.2 m
Pile end	Second limit resistance (kN)	6 199	8 767	10 992	3 700
	Coefficient of resistance	438	349	166	147
	Design value of coefficient of resistance	300	300	150	-

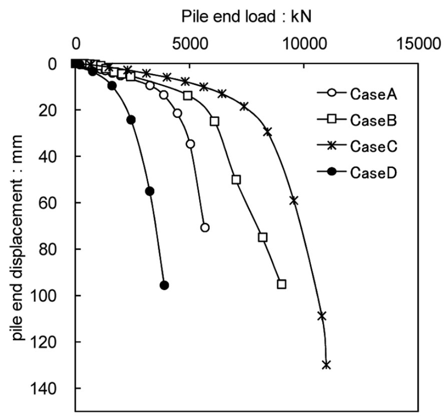


Fig. 10 Pile end load and pile end displacement

estimated. Here, limit resistance is defined as the pile tip load (secondary limit load) when the pile tip displacement reaches 0.1 D.[8] Table 2 shows the secondary limit load in each case. A value of 8.2 MPa is derived when the secondary limit load, 10992 kN, in Case C without a diameter expansion effect as an inner rib plate type is employed is divided by the steel pipe blockage area. This is lower than the uniaxial compressive strength of 13.2–30.7 MPa of soil cement test pieces sampled from the soil cement block. For this reason, it is considered that the soil cement block is not destroyed. In addition, based on structural tests for pile tip portions,[1] bearing strength R_i of slip keeper to prevent the soil cement from moving is designed using formula (1).

$$R_i = (D-2 \times t-d_i) \times \pi \times d_i \times \sigma_p \times a \times n \times d_i \qquad (1)$$

when the numerical values of this experiment (D: steel pipe diameter (=1300 mm), t: plate thickness (= 25 mm), d_i: steel bar diameter (= 13 mm), σ_p: unconfined compression strength of soil cement (= 13.2 MPa), a: coefficient of bearing pressure (= 4), and n: number of rows of slip keeper to prevent the soil cement from moving (= 7)), are substituted in formula (1) above, bearing strength R_i of the slip keeper to prevent the soil cement from moving is 18672 kN, which is significantly higher than the secondary limit load of 10992 kN. This also suggests that no destruction has occurred in the soil cement block. Instead, the secondary limit load is likely to be determined by destruction of the ground, meaning that it is possible that larger bearing capacity can be obtained in harder ground.

(ii) Skin friction

The skin friction intensity is obtained from the skin friction in each layer section with an N value of less than 50 divided by the peripheral surface area. The result shows a tendency that the skin friction intensity in sandy soil is proportional to the N value as shown in **Fig. 11**. For comparison purposes, the design lines as stipulated for cast-in-place piles (sand and gravel soil: 5 N, N ≤ 40) in the Specifications for Highway Bridges[9] are also shown in this figure, and it is evident that the skin friction intensity in each case is higher than these lines.

In addition, while only two data on skin friction intensity for cohesive soil, 41.3 kN/m^2 in Case B (82.6 N converted using the N value) and 170.7 kN/m^2 (50.2 N converted using the N value) obtained from a test[1] separately conducted, have been obtained so far, these values are significantly higher than the design value of 10 N for cast-in-place piles that is stipulated in the Specifications for Highway Bridges.

(iii) Design formula

In light of the above-described test results, formula (2) has been proposed as an equation that covers both outer-rib-plate- and inner-rib-plate-type pile tips,[2]

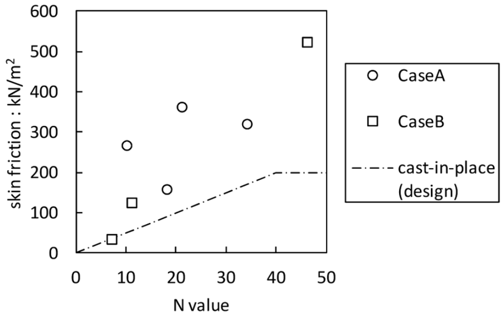


Fig. 11 Relationship between SPT-N and skin friction on sand

$$R = 300\alpha N\beta A_p + \Sigma(r_{fki} A_{si}) \qquad (2)$$

here, R: bearing capacity of the pile (kN), N: N value (≤ 50) of the pile tip ground, A_p: steel pipe tip blockage cross section, r_{fki}: pile skin friction intensity of layer i (kN/m^2) (sandy soil: 5 N [upper limit: 200 kN/m^2], cohesive soil: c or 10 N [upper limit: 150 kN/m^2]), and A_{si}: peripheral area of the steel pipe in contact with the ground in layer i (m^2). In addition, α is a coefficient that indicates variation in the resistance force in the same ground according to the construction method used, and is set to 0.5 in the case of the RS Plus™ method when this is specified as 1.0 in the case of the impact hammer driving method. This means a reduction in the ground strength in the case of the RS Plus™ method as it is difficult to take into account the displacement pile effect in this method unlike the impact hammer driving method, which is capable of excavated soil associated with plugged pile. β is also a correction coefficient for taking into account the pile tip shape; β = 2.0 for the outer rib plate type where the diameter expansion of the soil cement block bottom area to two times the steel pipe blockage area is taken into account, and β = 1.0 for the inner rib plate type for which no diameter expansion can be taken into consideration.

For the bearing capacity coefficient of the pipe tip resistance force, a comparison between the test result values and the design formulae is shown in Table 2. The bearing capacity coefficients (test result values) indicated in the second row from the bottom of Table 2 are derived by dividing the secondary limit load by the steel pipe blockage area and an N value of 50, while the design values indicated in the bottom row of Table 2 are derived from 300 $\alpha\beta$ in formula (2). This indicates that design values that are smaller than the test result values are on the safe side, and it is also clear that the design values are applicable to both the outer rib plate and the inner rib plate types. A notable point in the comparison with the impact hammer driving method is that the bearing capacity coefficients in Case C and Case D are equivalent. In the case of the impact hammer driving method, the larger the pile diameter becomes, the more difficult it is for blockage inside the pipes to occur, and thus it becomes harder to gain the necessary pipe tip resistance force.[8] Accordingly, if a pile of 1300 mm in diameter is driven using the impact hammer driving method in this ground, it is highly likely that the bearing capacity coefficient obtained becomes smaller than 166 in Case C. In addition to this, the design values of the skin friction in the RS Plus™ method as described above are equal to or larger than the design values of the impact hammer driving method[8, 9] (sandy soil: 2 N [upper limit: 100 kN/m^2], cohesive soil: c or 10 N [upper limit: 150 kN/m^2]). From the findings described above, it is shown that the bearing capacity performance of the piles driven using the RS Plus™ method exceeds that when using the impact hammer driving method.

4. Application Examples

4.1 Higashi port District quay (−9 m), Port of Karatsu

As part of intermodal transport terminal improvement projects promoted in order to improve the efficiency in transportation of increasing domestic general cargoes, and also in order to cope with the demand for large-sized passenger ships and for demand to secure transport functions in the case of a disaster, the existing old and outdated quay (constructed 40 years ago) in the port of Karatsu Higashi port District has been renovated as a quay with reinforcement against earthquakes.

Steel pipe piles of the quay are required to possess large resistance against push load, and they faced difficulties with the impact

Photo 5 Construction of RS Plus™ method on the Karatsu Port

Photo 6 Installation of submerged strut members on the Karatsu Port

Photo 7 Construction of RS Plus™ method on the Onahama Port

hammer driving method that required piles to be embedded fairly deep into the bearing layer consisting of weathered rocks (weathered granite). On the other hand, it was estimated that the RS Plus™ method would allow the driving of piles into such bearing layer using a Vibro hammer together with a WJ, and also would allow the embedment depth to be shorter than that using the impact hammer driving method as large resistance force can be gained by the soil cement block formed using a CJ. These advantages led to the adoption of the RS Plus™ method. The pile specifications are as follows: SKK490, 1300 mm in diameter, 22 mm in thickness, and 16.5–20.5 m in length. In the construction work from 2010 through 2014, 62 piles in total were driven. **Photo 5** shows the construction site. Furthermore, in this work, the RS Plus™ method was used in combination with an underwater strut beam method as shown in **Photo 6**.

4.2 Higashi port District quay (−18 m), Port of Onahama

The Port of Onahama is an international bulk strategy port supporting the supply of energy (coal) to Eastern Japan. In order to realize a more stable supply of energy at lower cost, an improvement project is in progress to allow the entry of fully loaded capesize carriers into the port. As part of this project, the construction of an 18-m-deep quay with reinforcement against earthquakes will be completed in Higashi port (a man-made island) District by 2020, and after that, a plan to deepen its water depth to 20 m has also been developed.

The structural type employed for the Higashi port quay is the pier type in which large-diameter steel pipe piles (1400 mm and 1500 mm) are arranged in three rows. Steel pipe piles are required to possess large resistance against pull-out force in addition to resistance against push-load force. The RS Plus™ method, which is effective with regard to pull-out force as it allows for a skin friction larger than that of the impact hammer driving method, in addition to pile tip resistance force, has been adopted. In the pile driving work started in FY 2014, the part for 42 piles (1500 mm in diameter) for 2 blocks out of 10 blocks in total has been completed. **Photo 7** shows the construction site. It is anticipated that part of the work for another 105 piles for 5 blocks will be completed in FY 2015.

5. Conclusion

The RS Plus™ method, which has been developed with the aim to satisfy demand for low-noise and low-vibration construction methods in port areas, as well as demand for the enlargement of pile diameters and bearing capacity to contribute to rationalizing port structures, is being increasingly employed in port projects. In the course of such development, issues requiring improvement that are instrumental in executing construction work in a more efficient and secure manner have arisen, each of which we are striving to resolve. We will continue to make efforts for further improvement, and will do our best to provide even more rational and economic construction methods.

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References

1) Kikuchi, Y. et al.: Development of Vibro Hammer Method to Drive Steel Pipe Pile Using Water and Cement Milk Jetting. Technical Note of the Port and Airport Research Institute. No. 1196, 2009
2) Mizutani, T. et al.: Application of Vibro Hammer Method Using Water and Cement Milk Jetting to Steel Pipe Piles with Large Diameter. Report of the Port and Airport Research Institute. 53 (3), (2014)
3) Uezono, A. et al.: The Bearing capacities of Steel Pipe Piles Installed by Water Jet Vibratory Technique and New Improving Methods for the Capacities. Journal of the Japan Society of Civil Engineers. 54 (700), 15–29 (2002)
4) Morikawa, Y. et al.: Performance Evaluation of Ground Excavation and Mixing by Plural High Pressure Injection Nozzles. Technical Note of the Port and Airport Research Institute. No. 1293, 2014
5) Modoni et al.: Theoretical Modelling of Jet Grouting. Géotechnique. 56, 335–347 (2006)
6) Moriyasu, S. et al.: Evaluation on Vertical Bearing Resistance of a Large Diameter Steel Pipe Pile Driven by the Vibratory Hammer Method with Water and Cement Milk Jetting. 58th Symposium on Geotechnical Engineering. 2013, p. 13–18
7) Takahashi, K. et al.: A Bearing Capacity of Vibratory Driven Steel Pipe Piles. Proceedings of the 34th Japan National Conference on Geotechnical Engineering. 1999, p. 1437–1438
8) Ports & Harbours Association of Japan: Technical Standard for Harbor Facilities and Its Commentary. 2007, p. 584–605
9) Japan Road Association: Specifications for Highway Bridges and Its Commentary. 2007, p. 377–402

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