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Performance of Seismic Retrofits with High Capacity Micropiles

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The civil engineering structure of Japan received big damage by Hyogo-ken Nanbu Earthquake (Kobe Earthquake). It was recognized that the seismic retrofit method of the existing structure was important after that. It is extremely difficult to obtain adequate execution space because of restrictions imposed by the space available under bridge girders and by nearby structures. The development of the industrial method which was able to be applied under such a condition was expected. The Public Works Research Institute conducted joint research with the Advanced Construction Technology Center and 12 private sector companies for three years beginning in 1999 in order to develop a seismic retrofitting method and a liquefaction countermeasure method that are not restricted by site conditions, even directly under an existing structure. As a result, High Capacity Micropile method was developed as one of "Joint Research on the Development of Seismic Retrofit Method for the Existing Bridge Foundation"

This report describes the performance of seismic retrofits with High Capacity Micropiles clarified by a model experiment and an analytical result when researching. Moreover, after the research had been completed, some seismic retrofit constructions were executed. This report introduces it.

Key Words: *High-Capacity Micro-pile, Performance of Seismic Retrofitting, Model-Test, and Batter-Pile*

1. Introduction

The technology was introduced by Italy in 1979, and the history of the micropile in Japan was applied as an underpinning of the observatory (Figure-1.1) in 1980. Micropile was applied in a wide field since then. When micropile was introduced, the case used by the ground reinforcement and the tunnel leg reinforcement was most. The type of these micropiles does not expect bearing capacity too much. Most the types were Figure-1.2(a) or Figure-1.2(b).

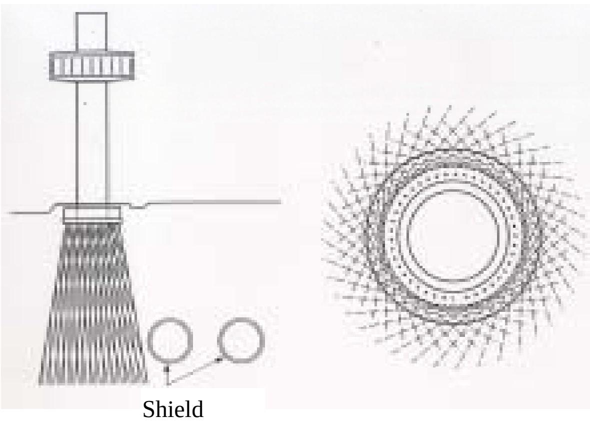

Fig-1.1 The observatory

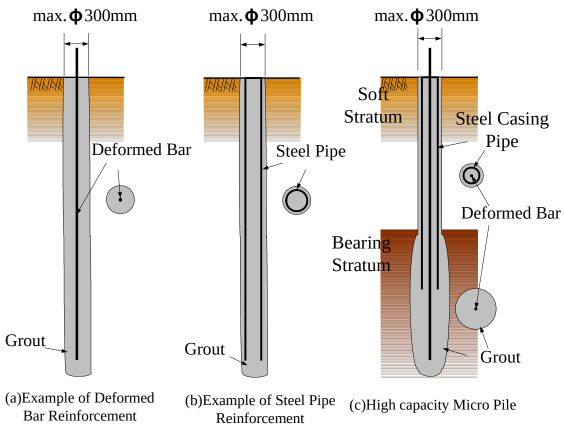

Fig-1.2 Structure of the Micropiles

Hyogo-ken Nanbu Earthquake (Kobe Earthquake) occurred on January 17, 1995. The position of seismic center is an Awaji Island northern part at 34°36" north, 135°02" east. The hypocenter was 16km in depth, and the scale of the earthquake was 7.3-magnitudes. This earthquake was generated in the inland, and was so-called an epicentral earthquake. A very big quake was caused in the destroyed vicinity of the fault. Big damage was received in Hanshin region which centers on Kobe City and the Awaji island northern part (photograph-1.1).

Photograph-1.1 Hyogo-ken Nanbu Earthquake

Since the Hyogo-ken Nanbu Earthquake (Kobe Earthquake) of 1995, the damage investigation and the cause were examined, and various design specifications etc. were revised. And the seismic design standards for highway bridges have been revised. It is, therefore, now necessary to perform seismic retrofitting of existing structures that have not been damaged. It is extremely difficult to obtain adequate execution space because of restrictions imposed by the space available under bridge girders and by nearby structures. The development of the industrial method which was able to be applied under such a condition was expected. The Public Works Research Institute conducted joint research with the Advanced Construction Technology Center and 12 private sector companies for three years beginning in 1999 in order to develop a seismic retrofitting method and a liquefaction countermeasure method that are not restricted by site conditions, even directly under an existing structure. As a result, High Capacity Micropile method was developed as one of "Joint Research on the Development of Seismic Retrofit Method for the Existing Bridge Foundation"

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2. Model Test 1(Horizontal Loading Tests of Different kinds of Group Piles)

(1) Purpose

When the existing foundations are retrofitted by micropiles, many micropiles will be arranged around the existing piles and connecting them to the footing. And, they shows behavior as group of piles. Each piles interfere mutually through the stress in the ground when horizontal force acts on the group of piles, and the piles are transformed greatly. It is called the effect of the group of piles. Up to now, most of the experiment concerning the group of piles of Japan had been executed with the pile of the same diameter in the same kind. The performance of the group of piles composed of existing piles of big diameter and micropiles of small diameter has not been elucidated yet. Then because we want to understand the performance of a group of piles with different diameters, we did a static load testing which used them. Moreover, we have aimed to understand the performance of seismic retrofits with HMP appropriately, and to obtain the material of the design method.

(2) Outline of the test

We did a static load testing which used the group of piles with different diameters. The model actually used was an approximately 1/5 scale model of an actual pile foundation. The test was performed for the seven cases shown in Table 2.1.

Table-2.1　Cases of Horizontal Loading Tests

Case	Number of Piles	Spacing between Existing Piles Center and Micropiles Center (mm)	Inclination Angle of Micropiles (°)
1	Single Existing Pile	–	–
2	Single Micropile	–	–
3	4 Existing Piles	–	–
4	4 Existing Piles and 6 Micropiles	200	0
5	4 Existing Piles and 6 Micropiles	400	0
6	4 Existing Piles and 6 Micropiles	200	10
7	4 Existing Piles and 6 Micropiles	200	20

Table-2.2　Specifications of Model Piles

	Diameter (mm)	Thickness (mm)	Sectional Area (cm^2)	Moment of Inertia (cm^4)
Existing Pile	114.3	3.5	12.18	187.0
Micropile	34.0	2.3	2.291	2.89

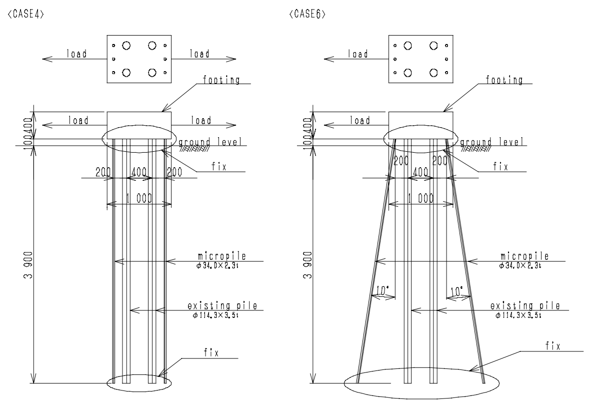
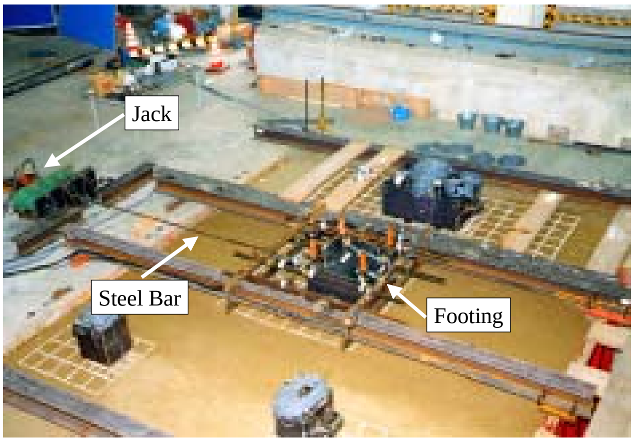


Fig-2.1　Case4 and Case6



Photograph-2.1 Loading Test for Case 3

Case 1 is single existing pile. Case 2 is single micropile. Case 3 is a footing with four existing piles. The effect of group of piles is confirmed with case 3. Case 4 was retrofitted case 3 with six micropiles. Three micropiles were set up respectively on both load sides. The effect of group of piles of the different kind piles is confirmed with case 4. In case 5 the interval between the existing piles and micropiles is twice case 4. Micropiles of case 4 and case 5 are vertical. Micropiles of case 6 and case 7 are batter piles. Micropiles of case 6 are ten degrees, and those of case 7 are 20 degrees. The parameter of the existing piles and micropiles used for each case is shown in Table-2.2. Moreover, the outline of case 4 and case 6 is shown in Figure-2.1.

The footing was made steel. And the piles and the footing united with the adhesive of the epoxy.

The horizontal displacement, vertical displacement of the footing, load, and the warp of the pile were measured. The examination situation of case 3 is shown in photograph-2.1.

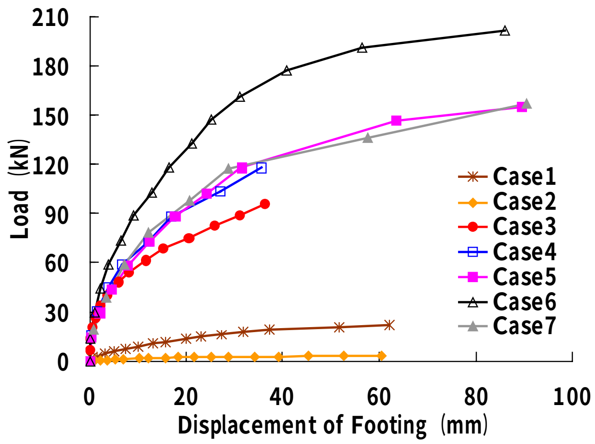


Fig.-2.2 Curve of Load and Displacement

(3) Result of Load and Displacement

Figure-2.2 shows the load – displacement curves for each case. Figure confirms that in case 4 and case 6, the retrofitting effects of the micropiles are greater than in case 3 that represents conditions before retrofitting. A comparison of the results for case 4 and case 5 with different spacing between the existing piles and micropiles shows that there is no conspicuous difference between the retrofitting effects according to the interval between piles. In case 6 where the micropiles were inclined, retrofitting effects of the micropiles were conspicuously greater than in case 4 and case 5. Case 7 was omitted from the analysis because the model was defective.

(4) Consideration

Here, CASE3 and 4 are described. The bending moment of the existing pile is shown in Figure-2.3. Figure shows the one in load stage before the yield load, and it is 88.8kN in CASE3, and it is 88.0kN in CASE4. In CASE3, the value of the maximum bending moment of

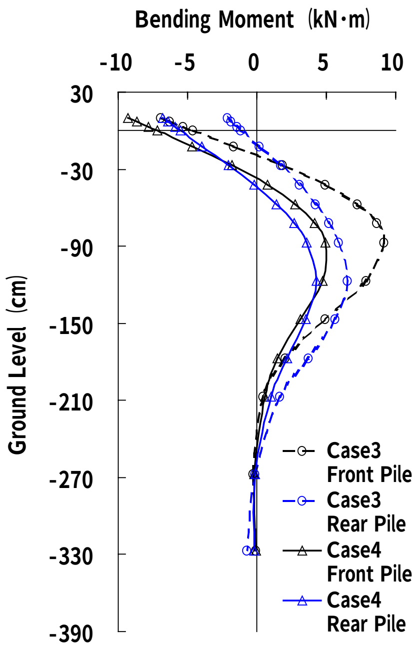


Fig-2.3 Bending Moment on Existing Piles in Case 3

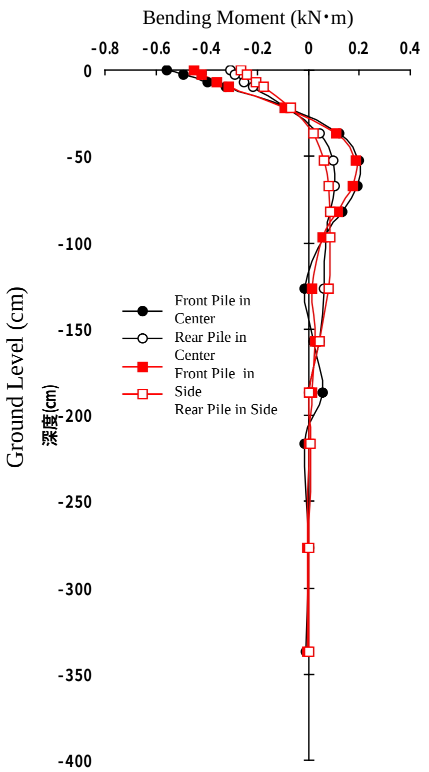


Fig-2.4 Bending Moment on Micropiles in Case 4

a front pile is larger than that of the rear pile. A front pile of the maximum bending moment was almost the same the rear pile in CASE4.

The bending moment of micropiles are shown in Figure-2.4. The load stage is the same as Figure-2.3. An almost equal value was indicated in a central pile and the side pile. On the other hand, the maximum bending moment and the pile head moment of a front pile was larger than that of the rear pile. Moreover, the distribution form of the bending moment of the rear pile was greatly different from the front pile. The bending moment value of the rear pile indicates an almost constant value to bottom -50cm to -150cm of the footing.

These properties originate in the difference of horizontal ground reaction.

And, it is thought that the deep position of bending moment is influenced by the difference of pile diameter (difference of the flexural rigidity). The cause of the difference of horizontal ground reaction is due to the effect of pile group. We should examine the method of value the different kind piles in group.

Load and shearing force of pile head of case4 is shown in Figure-2.5. The analysis of frame result in figure is what calculated by the condition that the effect of pile group is not considered. And the pile of nonlinear character and ground of nonlinear character are not considered. It is understood that the ratio of the front existing pile of the allotment of the shearing force of pile head is larger than rear existing pile from the experiment result. When the load grows, this tendency is clear. Moreover, it is a similar tendency in HMP. In the sum total of the shearing force of all piles, the allotment rate of shearing force of pile head of all HMP was about 10 percent. The allotment rate of HMP was about 20 percent in analysis of frame result, and it was larger than the experiment result. It will be necessary to examine a quantitative evaluation of the allotment rate of HMP to us in the future.

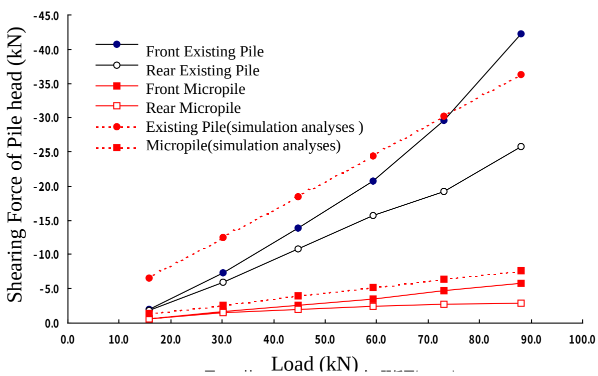
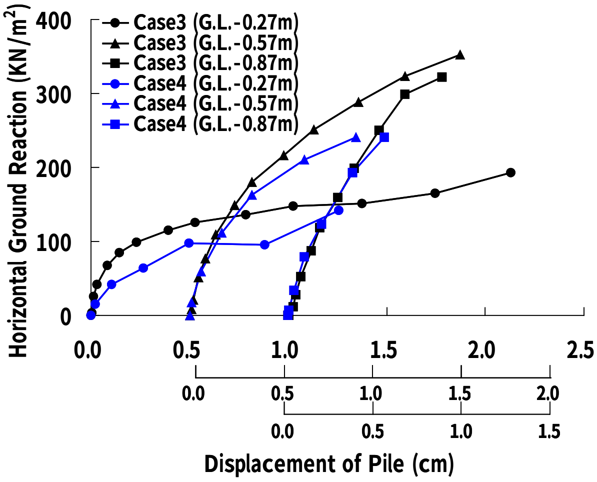


Fig-2.5 Load and Shearing Force of Pile head (CASE4)

The curve of horizontal ground reaction and displacement of existing pile is shown in Figure-2.6. In CASE3, 4, the horizontal ground reaction of each case is almost equal when displacement is small. However, in a shallow range in the ground, when displacement



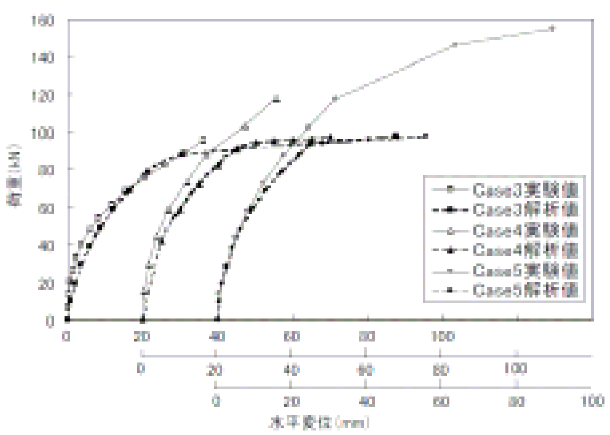
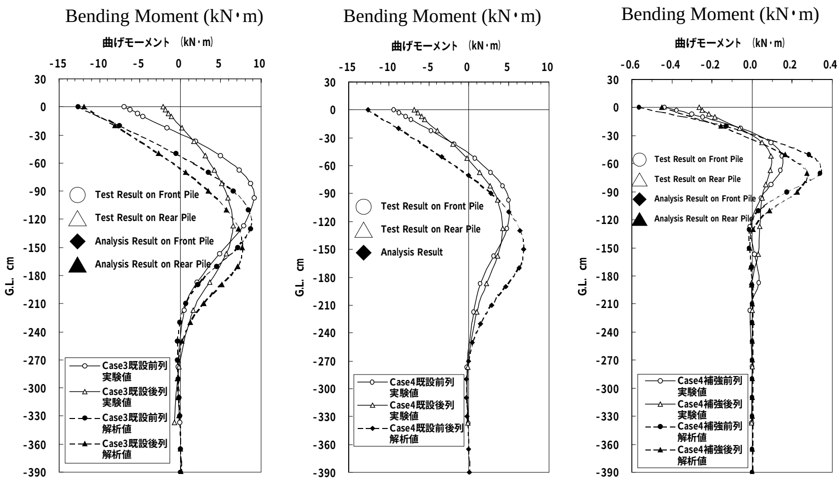


Fig-3.1 Curve of Load and Displacement

Fig-2.6 Curve of Horizontal Ground Reaction and Displacement

progresses, horizontal ground reaction of CASE4 is smaller than that of CASE3. This reason is horizontal ground reaction decreased by the influence of the pile in front HMP of CASE4. However, the difference becomes small while becoming deep. In GL-0.87m, the curve was almost corresponding. This means the influence of horizontal ground reaction appears at a shallow position contrary to the finding concerning the bending moment of the rear pile of HMP above-mentioned.



CASE3(Existing Piles) CASE4(Existing Piles)

CASE4(Micropiles)

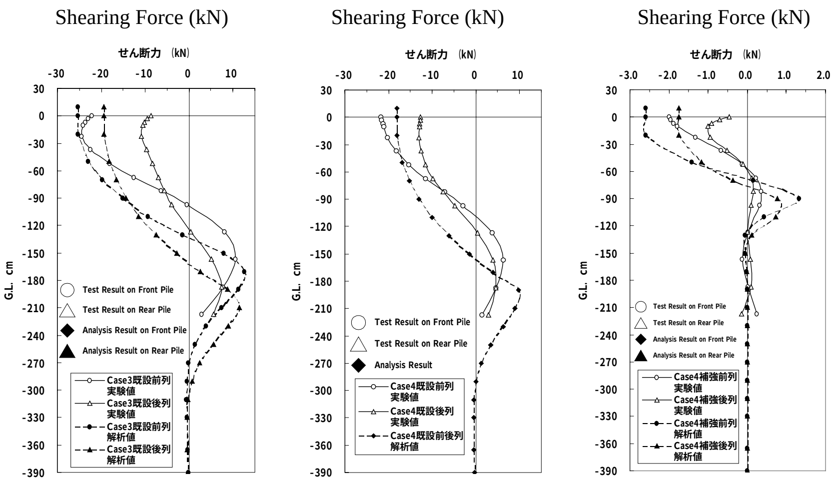
Fig-3.2 Bending Moment in Case 3 and Case 4

3. Static analysis for Group of piles with different diameter (Simulation Analyses for the Horizontal Loading Tests)

(1) Purpose

The purpose of a static horizontal load test of the above-mentioned group of piles with different diameter model was the undermentioned two points.

1. Examination of effect of group of piles with different diameter when existing foundation is retrofited by HMP.
2. Examination of design technique that effect of retrofit is



CASE3(Existing Piles) CASE4(Existing Piles)

CASE4(Micropiles)

Fig-3.3 Shearing Force in Case 3 and Case 4

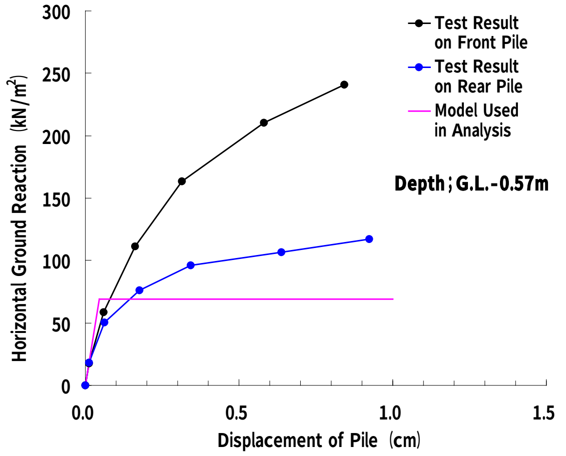


Fig-3.4 Curve of Horizontal Ground Reaction and Displacement in Case4

appropriately considered.

Here, the application was examined by analyzing simulating the above-mentioned horizontal load test based on the standard design in Japan. We describe the part of it.

(2) Comparison with the simulation analysis

The simulation analysis was performed based on the ductility design method, that is stipulated in the Design Specifications of Highway Bridges. For the analysis, a correction factor that accounts for the pile group effect was set so that the test results could be reproduced.

Figure-3.1 compares the load – displacement curves obtained by the test results and by the analysis results for cases 3 to 6. As shown in Figure -3.1, the load – displacement relationships for all cases are reproduced with relatively high accuracy by appropriately setting the correction factors. Similarly to the test results, there are almost no differences in the analysis results for case 4 and case 5, and it is assumed that increasing the spacing between the existing piles and micropiles has little effect on the retrofitting effects.

Figure-3.2 and Figure-3.3 show the bending moment distribution and the shear force distribution of the existing piles in case 3 and the existing piles and the retrofitted piles in case 4 obtained by the testing and by the analysis, In both cases, the distribution of the bending moment and the location of its maximum value obtained by the analysis closely resemble those obtained by the testing. The shear force in the analysis results also closely resembles that from the testing results. The testing confirmed that retrofitting a pile foundation with micropiles obtains retrofitting effects. It also confirmed that installing the micropiles at an angle increases the retrofitting effects.

The ductility design method confirmed that it is possible to perform design that appropriately reflects the retrofitting effects of micropiles.

Curve of horizontal ground reaction and displacement of case4 is shown in Figure-3.4. The elastoplastic model of ground in analysis is shown in figure. The elastoplastic model in the ground considered the correction factor to adjust to load-displacement relation. These are compared. The elastoplastic model in the ground and existing rear pile show almost the same tendency. But it and existing front pile are away.

4. Model Test2 (Shaking Table Test of an Existing Foundation Reinforced with Micropiles)

(1) Purpose

The purpose of this shaking table test is to understand the dynamic behavior of the seismic retrofits foundation by the vertical HMP or the batter HMP.

(2) Soil container and Soil model

The test was conducted in the gravitational field using a shear soil container on a large shaking table. An outline of the test apparatus and the shear soil container are shown in Figure-4.1.

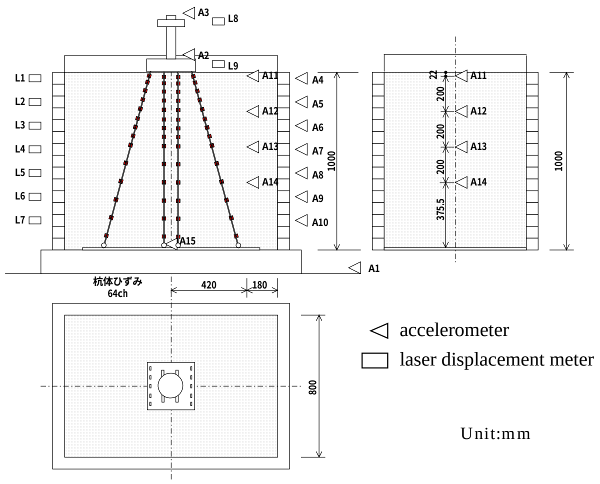


Fig-4.1 Model of Shaking Table Test

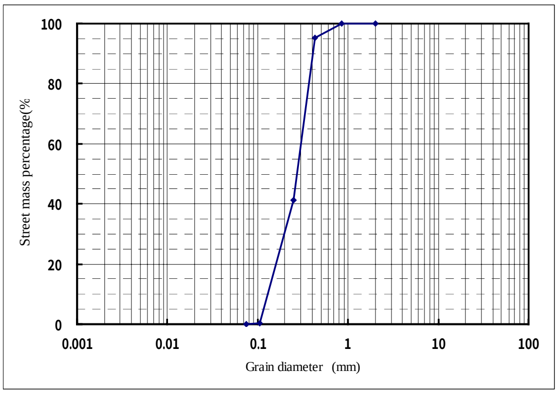


Fig-4.2 gradation curve of the Hamaoka sand

Table-4.1

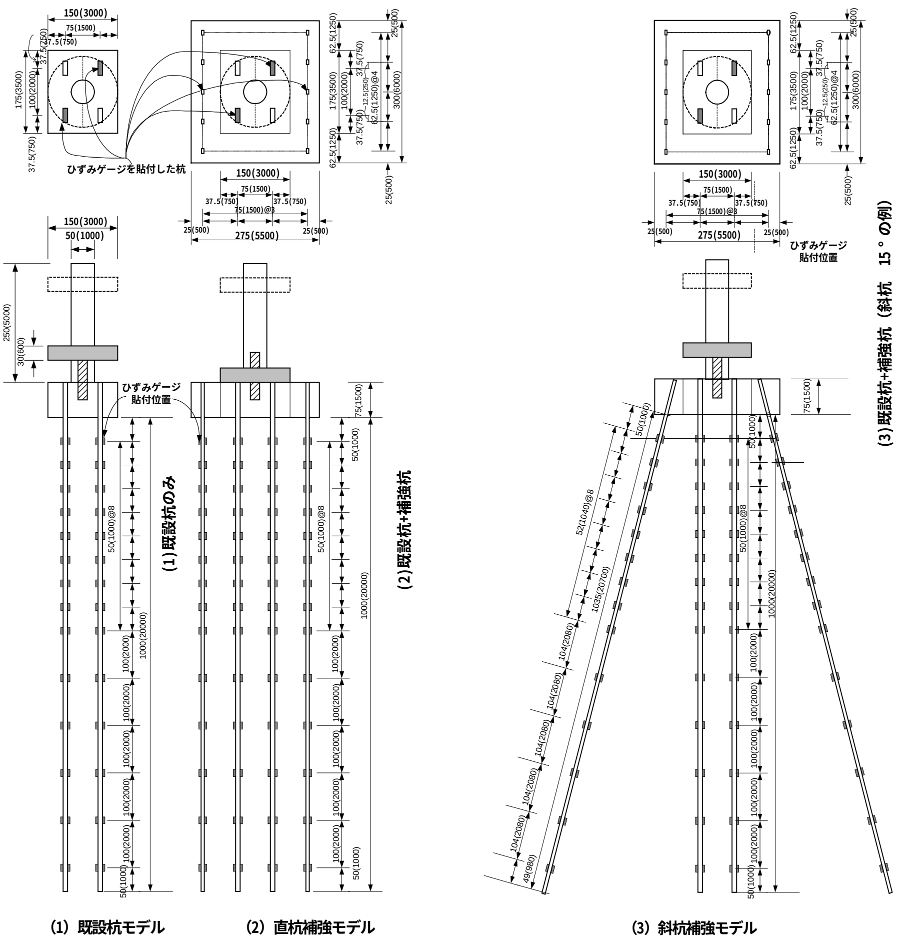
Density of soil particle	ρs	2.699 g/cm^3
Grin size distribution	Gravel content	0%
	Sand content	100.00%
	Silt content	0.00%
	Clay content	0.00%
	Uniformity coefficient	2.31
	Curvature coefficient	1.03
Maximum dry density	ρd max	1.694 g/cm^3
Minimum dry density	ρd min	1.396 g/cm^3

The Enshu-Hamaoka sand was used for the soil model. Table-4.1 and Figure-4.2 show the physical properties and gradation curve of the Hamaoka sand, respectively. As shown in the figure, the Hamaoka sand is well-graded. The soil model was created by spraying the sand in the air over the shear soil container in which pile models had been installed until the sand reached the bottom surface of the footing. The sand was dropped for a height of 1.5 m so that the initial relative density might become 60%.

(3) Soil container and Soil model

The test model of foundation is shown in Figure-4.3. The model1 is the model of existing piles. The model2 is retrofitted by the vertical HMP. The model3 is retrofitted by the batter HMP of 15 degree. The test was executed to each model in consideration of "There was a structure in the part" and "There is no structure in the part".

Models with a 1:20 scale were made of an existing 600-diameter prestressed concrete pile, and a 177.8-diameter steel pipe pile used for reinforcement. Table-4.2 lists the dimensions of the existing and reinforcing piles. The



(1)model 1 (2)model 2 (3)model3

Fig-4.3 Pile Model

bending stiffness of the pile model was set at one-tenth of the value obtained by the law of similarity because the test aimed to grasp the reinforcing effect qualitatively. Stainless steel plates 30 mm wide and 5 mm thick and those 10 mm wide and 2 mm thick were used for the models of the existing and small-diameter piles, respectively. Table-4.3 shows the dimensions of the pile models. A group of four piles were rigidly connected to the footing to make a model with the existing piles, and the pile heads were pin-connected. A total of ten piles were used in two rows of five on either side to represent models reinforced with small-diameter piles.

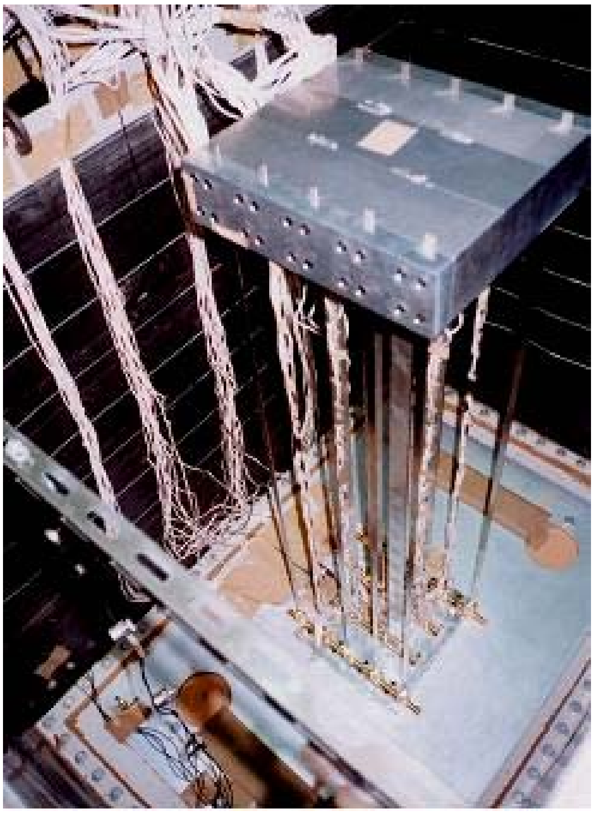
Sine wave with a frequency of 10 Hz and a maximum acceleration of 100 to 300 gal was input as a ground motion because the natural period of the soil model was 0.104 seconds.

Table-4.2 Dimensions of the piles

	Pile Type	Diameter φ(mm)	Thickness (mm)	EI (kN·m^2)
Existing Pile	Prestressed Concrete Pile	600	90	2.00E+05
Reinforcing Pile	Steel Pipe Pile	177.8	12.7	2.47E+03

Table-4.3 Dimensions of the pile models

Pile model	Material	Width (mm)	Thickness (mm)	EI (N·cm^2)
Existing	Stainless steel	30	5	5.52E+05
Reinforcing	Stainless steel	10	2	1.18E+04

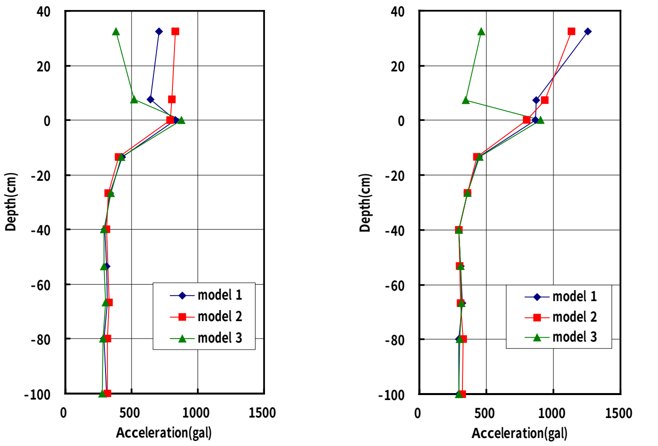


Photograph-4.1 The conditions that a model installed

(4) Results and Consideration

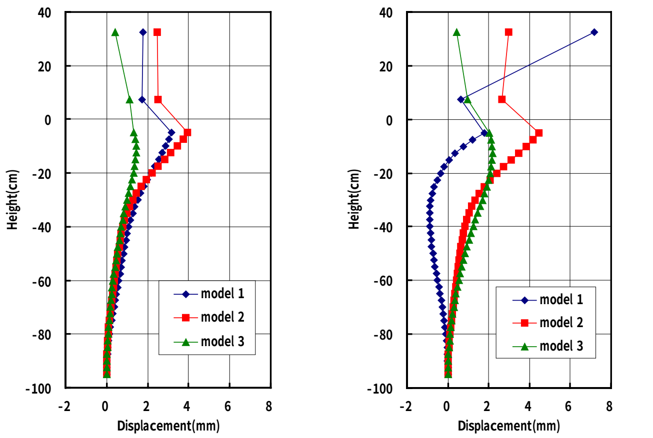
a) Maximum acceleration response

Figure-4.4 shows the maximum acceleration response. The values without and with the weight of the superstructure are shown on the left and right sides of the figure, respectively. The acceleration response of the shear soil container was the same for all models. Thus it was confirmed that testing was conducted under the same soil conditions. The acceleration response of the bridge pier was almost the same for the model with existing piles and that reinforced with vertical piles. The model reinforced with raking piles had a smaller acceleration response than the other two types of models. An increase in lateral displacement of the soil caused the raking piles in front of the existing piles to become straighter and those behind to tilt further, creating a rotating force that led to the displacement of the bridge pier in the opposite



a) the values without the weight b) the values with the weight

Fig-4.4 the maximum acceleration response



a) the values without the weight b) the values with the weight

Fig-4.5 the displacements of the existing pile and bridge pier direction to that of soil displacement. As a result, the response acceleration of the model reinforced with raking piles may have decreased.

b) Distribution of displacements

Figure-4.5 shows the displacements of the existing pile and bridge pier when the bending moment of the existing pile was largest. The displacement of the existing pile was obtained from its bending moment. The displacement of the bridge pier in the model reinforced with vertical piles was larger than in the model with existing piles probably because the footing became larger after reinforcement and had greater inertia force. Vertical piles are, however, considered to have reinforced the foundation because while the total weight of the reinforced model was 2.8 times larger than the pre-reinforcement weight (without the weight of the superstructure), the displacement of pile head was 1.4 times larger. It was confirmed that the model reinforced with raking piles was effective in controlling the response of the pile head and bridge pier as described earlier. When there was a weight of the superstructure, displacement was very large at the top of the bridge pier because the existing foundation could no longer bear the angular moment that was created at the base of the pier by the inertia force of the superstructure, and tilted.

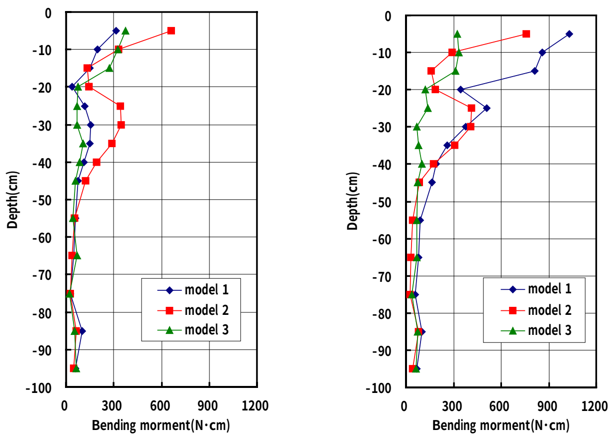
c) Distribution of maximum bending moments

Figure-4.6 shows the distribution of maximum bending moments. The reinforced models behaved similarly regardless of whether there was a weight of the superstructure or not. With the model with existing piles, the bending moment at pile head was 3.2 times larger when there was a weight of the superstructure than when there was none. The bending moment at pile head was 1.1 times larger with the model reinforced with vertical piles. This means that vertical piles effectively reinforced the foundation. The maximum bending moment was larger for the model reinforced with vertical piles with no weight of the superstructure than for the model with existing piles because the footing gained weight as described earlier.

(4) Conclusion

The shaking table test produced the following qualitative results.

- The dynamic response characteristics of the foundation reinforced with small-diameter steel pipe piles proved free from problems.
- Adding small-diameter steel pipe piles were found effective in reinforcing the foundation.
- It was verified that the deformability of raking piles installed to reinforce the foundation could help control the response of the structure.



a) the values without the weight b) the values with the weight

Fig-4.6 the distribution of maximum bending moments

5. Result of a joint research

PWRI collaborated with private companies in researching “The Development of Seismic Retrofit Method for the Existing Bridge Foundation".

It was researched from 1999 to 2001.

In a joint research, a more detailed examination was done based on the above-mentioned model experiment and the numerical analysis result. As a result, the design method and the construction method of HMP were established.

Mr. Ishida from PWRI distributed the English version of the design and the construction manual of HMP in IWM2003. Please refer to that for details.

6. Application Example with HMP

After completing the design and the construction manual, HMP has increased the application example as a seismic retrofit method in Japan.

In the following, some of the application example with HMP are introduced.

(1) Seismic Retrofitting of RYUSENJI Viaduct

a) Background

Seismic performance of the existing foundation was checked, so it was judged that the seismic retrofitting of the existing foundation was necessary in the RYUSENJI viaduct located in the Miyazaki Pref. Miyakonojou City. The height limitation of this site was about 4m, and the construction condition was hard. Therefore, HMP was applied as seismic retrofitting method of the existing PC pile.

b) Specifications of RYUSENJI Viaduct

The length of the RYUSENJI viaduct is 119.3m, width is 12m, and it is 9 span. The length of one span is 12.2m or 14.4m. This viaduct was designed according to

Photograph-6.1 Appearance of construction

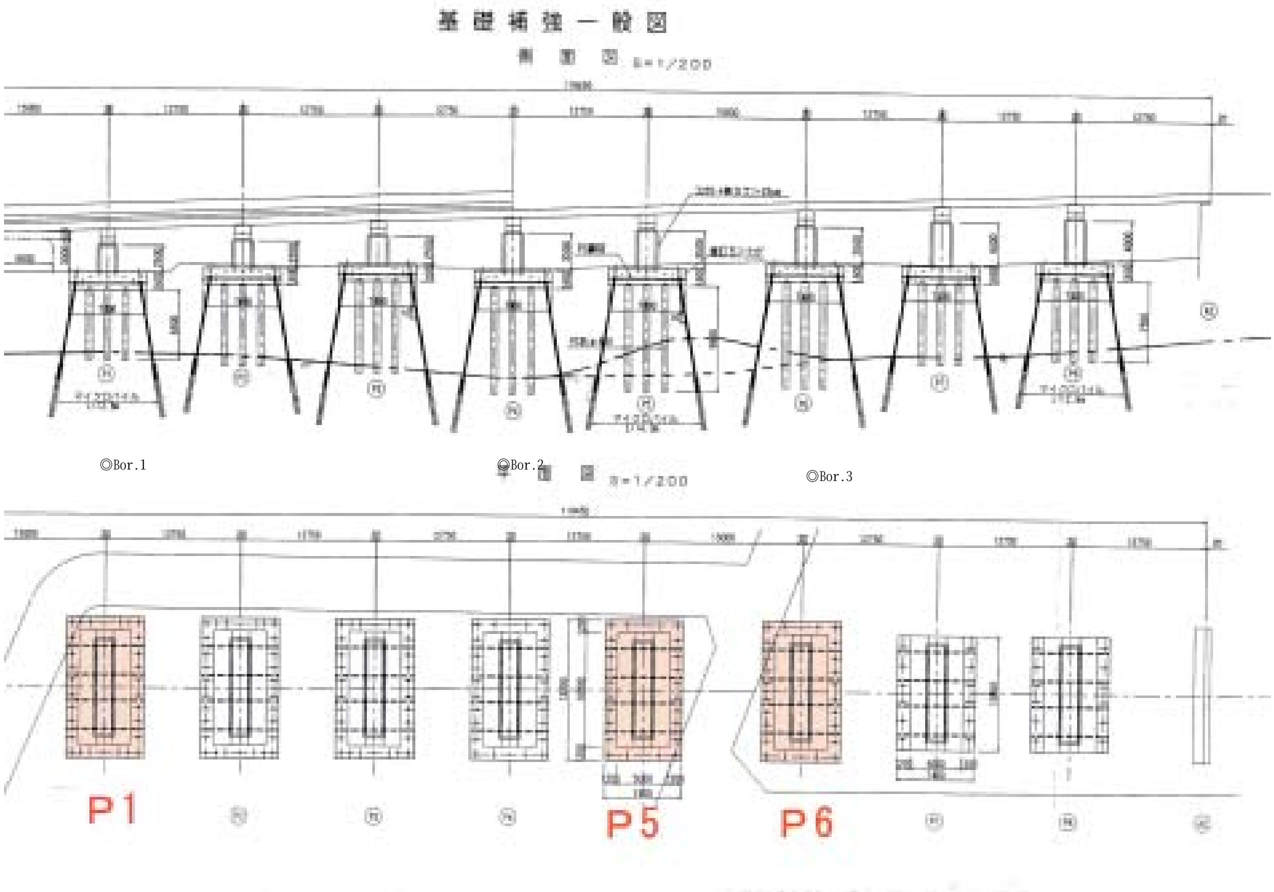


Fig-6.1 A side view and a ground plan

Design Specifications for Highway Bridge in 1980, and completed in 1983. It is necessary to retrofit for eight piers. First of all, three piers(P1,P5,P6) were retrofitted on this site(Refer to Figure-6.1 and Photograph-6.1).

c)Outline of Construction

Both the vertical HMP and batter HMP were necessary for P1 because of the difference of the design condition. P5 and P6 were the construction only of batter HMP. P1 was retrofitted by 8 pieces of vertical HMP and 20 pieces of batter HMP. And the total was 28 (L=12.8m). Each of P5 and P6 were retrofitted by 24 pieces of batter HMP of 14.3m in length. The angle of both batter HMP was 10 degrees.

(2) Seismic Retrofit Construction of a water service station at KAMEIDO in TOKYO

a) Background

KAMEIDO Water Service Station (hereafter, Water Service Station) was constructed in 1970. It is RC structure that the half is in the underground. It has the facilities for supplying drinking water to people in that area and the wide refuge area. Seismic performance of this facilities was checked, so it was judged that the seismic retrofitting of the existing foundation was necessary. When the retrofitting of the foundation was examined, it was difficult to retrofit from the outside of the water service station because of the standpipe and so on. Therefore, it was necessary to retrofit for existing foundation from the inside on the water service station. However, there were the problems in the construction condition that the construction place was narrow, and there was a limitation in height. Because HMP was able to solve these problems, HMP was applied as a seismic retrofitting of structural foundation method of this facilities.

b)Outline of KAMEIDO Water Service Station

It is a structure of about 7m in height and width is 104m×104m. It can store up the drinking water of 60,000m^3. And it consists of two ponds.

Existing pile is PC pile of 35m-48m in length, and the diameter is ϕ 600. They were constructed by a houndstooth check at intervals of 2m. The number of them are about 1700 pieces. This facilities is delimited to 16 rooms with an inside wall, and it has divided into nine blocks by expanded joint(Figure-6.2 references). The stratum is very soft and N value is 0-3 in the range up to 35m in depth. Almost the existing pile is bearing to the gravel layer of 35m in depth. However, the 61 of existing piles were 48m length in existing 1700 piles according to the construction record. The 61 of existing piles went through the first bearing layer, and reached TOKYO gravel layer of second bearing layer.

c)Outline of Construction

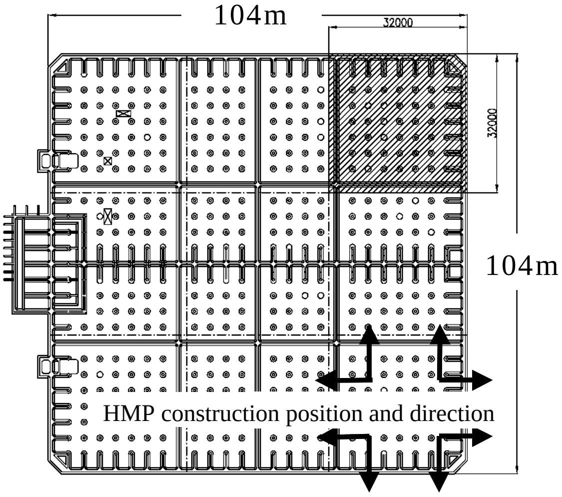


Fig-6.2 a ground plan of Water Service Station

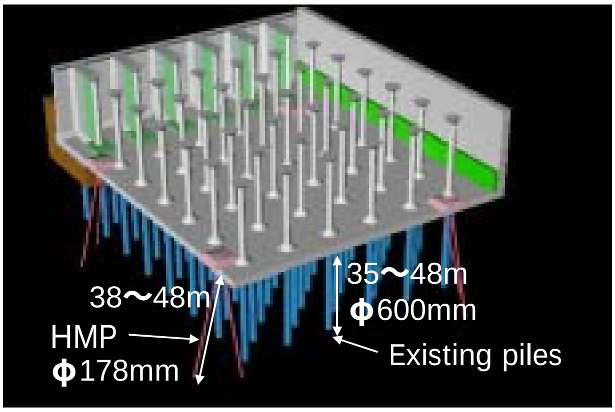


Fig-6.3 Retrofitting image (Expansion of Fig-6.2 hatching part)

Photo-6.2 Appearance of construction

KAMEIDO Water Service Station was divided into nine blocks. Two pieces of HMP were constructed from four corners downward between the existing piles at every block. HMP was batter pile of all 15 degrees, and length was from 38 to 48m. The 72 pieces of HMP in all were constructed(Figure-6.3 and Photograph-6.2 references).

(3) Seismic Retrofit Construction of Electric Pylon

a) Background

HMP was applied to the retrofitting of the iron tower foundation of the power-transmission line on the mountains ground. The applied main reason is as follows.

①The construction machine and the material were able to be transported by the helicopter because it was light and small.

②HMP can be constructed in a narrow place in the existing power-transmission line iron tower neighborhood.

③HMP can be constructed by penetrating through the existing footing.

④It is construction in slope ground of the average diagonal degree 40 degrees.

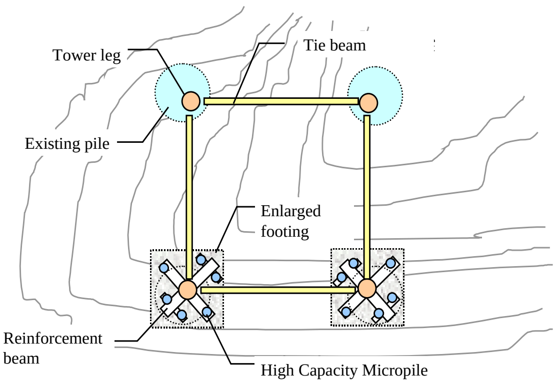


Fig-6.4 Outline of Construction

Photo-6.3 Helicopter for transportation

b) Outline of Construction

HMP penetrated through the footing and was constructed in surroundings of the iron tower leg judged that retrofitting was necessary. And the steel pipe of the head of HMP supported the iron tower leg directly (Figure-6.4 references).

It was necessary to do the transportation to the construction area of the materials and machine parts with a large-scale helicopter (photograph-6.3), and to become 2t or less in the maximum weight, all the materials and machine parts was divided. Moreover, the trestle was constructed in the tube pipes with the wooden work stands on the slope (photograph-6.4), and the drilling of HMP was with a drilling machine of skid type (photograph-6.5).

Photo -6.4 Trestle

Photo-6.5 Appearance of construction

7. Development in the future

(1) Application to New Foundation

Recently, HMP is often used for seismic retrofitting of the existing foundation. On the other hand, the application of HMP is examined on the site of the new foundation. Their construction place is narrow in the mountains ground or the city part, or site where installation of construction machine large-scale in consideration of influence on surroundings is difficult.

Japan Highway Public Corporation laboratory paid attention to HMP as an alternative method of the caisson type pile method that the construction scale increased on the mountain ground. JH researched the application of HMP about the durability performance of the bridge foundation of structure on the mountains ground and, to reduce the improvement and the cost.

An economic effect when HMP is applied in place of the caisson type pile method is calculated on trial. That result is shown in Table-7.1. A great reduction of the labor expense can be expected of HMP, because the pile construction of HMP is possible by simpler construction machine than the caisson type pile method. また、施工 Moreover, a further cost reduction can be expected because it is thought that cost of incidental facilities from the process shortening can be reduced.

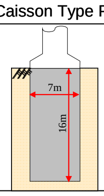
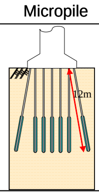
HMP can be improved by the economy and construction compared with the caisson type pile method, and can be expected enough as an alternative method of the caisson type pile method.

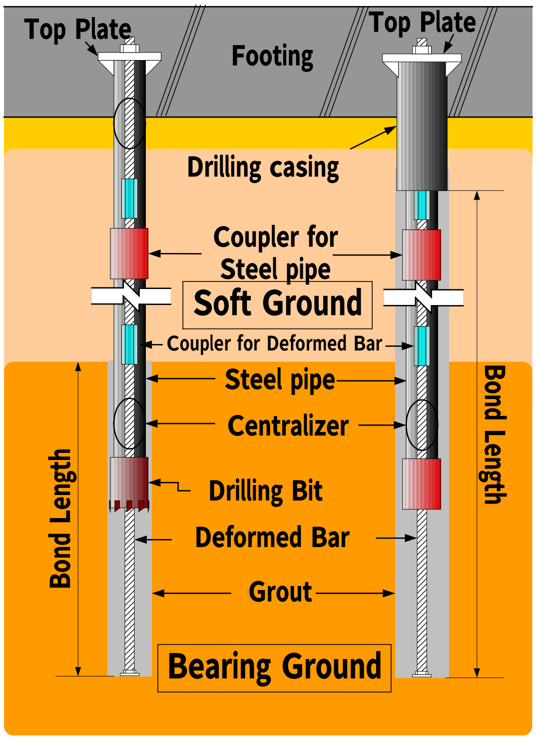
(2) Improvement of HMP method (NEW-HMP method)

The traffic jam occurs chronic in a lot of main intersections in the city part, and an economic loss due to traffic congestion and the influence on the environment, and so on are social problems in our country. We have to need quickly the overpass of the intersection to solve such a problem.

However, the application of the HMP method was examined as a new foundation in the city part because there was a site that an enough construction yard was not able to be secured. Then, the improvement was added to the HMP method, and the NEW-HMP method to aim at the confirmation of application to the new foundation was developed for the new foundation of the overpass part in the city part. NEW-HMP is a pile which expands the drilling diameter more than HMP, friction in the soft stratum on the surroundings side is secured, and the increase of the bearing capacity becomes possible. The outline is shown in Figure-7.1.

Table-7.1 Effect when HMP is applied to new foundation

		Caisson Type Pile	Micropile
Structure		 Φ7m@16m @1 piece	 Φ0.17m@12m @49 pieces
Amount of digging(m³)		615	115
Economy	materials	39%	43%(+4%)
	labor	43%	21%(-22%)
	machine	6%	22%(+16%)
	incidental facilities	12%	4%(-8%)
	Total	100%	90%(-10%)



(a)HMP (b)NEW-HMP

Fig-7.1 Outline of HMP and NEW-HMP

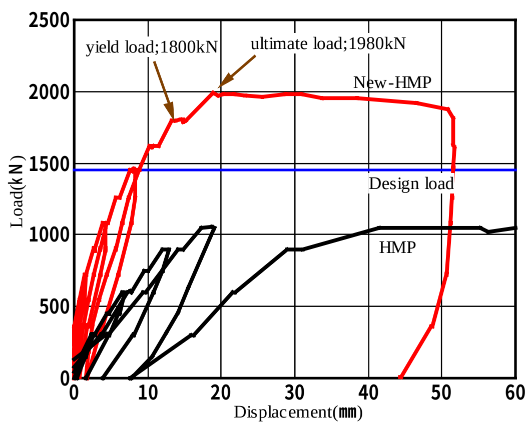


Fig-7.2 Pull-out test of NEW-HMP

Moreover, a main characteristic is recorded as follows.

① Construction in a narrow place is possible, and easing the second congestion is expected while constructing NEW-HMP.
② Diameter of NEW-HMP is larger than that of HMP, and the bearing capacity of the NEW-HMP increases more than HMP in the soft stratum as the skin friction can be expected (Friction in the soft stratum on the surroundings side is not considered in HMP).
③ Because NEW-HMP decreases the number of the pile compared with HMP, the construction period can be shortened and the cost can be made down.
④ Because only fixed length leaves the surface of the ground the drilling casing used with NEW-HMP, the horizontal resistance of the NEW-HMP is larger than that of HMP.
⑤ The process of re-insertion of the steel pipe in HMP becomes unnecessary, and the simplification of the construction process becomes possible.

Moreover, the pulling out load test of NEW-HMP was executed in PWRI, and it was confirmed for the skin friction in the soft stratum. And the NEW-HMP bearing capacity which was 20 percent bigger than the HMP design load.

8. Conclusion

After completing the design and the construction manual, HMP has increased the application example as a seismic retrofit method in Japan.

The construction results of HMP were evaluated, and HMP method received “Award of excellent new civil engineering technology" in 2005. Moreover, I want to expect the expansion and the development of the application field of the micropiles in the future as the down of the cost and the performance improvement are achieved by researching NEW-HMP.

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Pohto-7.1 Appearance of NEW-HMP construction