STUDY ON THE RETROFITTING OF BRIDGE FOUNDATIONS WITH MICROPILES

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ABSTRACT

Micropiles are anticipated that they will be used to retrofit bridge foundations under restricted conditions thanks to their superior execution characteristics. This report describes a study on the applicability of micropiles as foundation retrofitting conducted by performing a trial micropile execution followed by vertical and horizontal alternating loading tests to clarify their bearing capacity and deformation properties.

1. INTRODUCTION

The Hyogo-ken Nanbu Earthquake of January 17, 1995 generated extremely large seismic motion (magnitude 7.2) hitherto not observed in Japan. This earthquake caused extensive severe damage on highway bridges including the destruction of bridge piers and collapse of bridge girders. The Ministry of Construction established the new specification for highway bridges in 1996. Under this specification, in addition to elastic design as in the past, highway bridges design account for the non-linear properties of both foundation members and the ground around a foundation (ductility design method). While the seismic retrofitting of highway bridges is now in progress throughout Japan, this retrofitting work is focussed on bridge piers so that work on the retrofitting of foundations under ground has been somewhat delayed. Reasons for this are the generally small damage the earthquake inflicted on foundations compared with the damage to above ground structures and the relatively high cost of retrofitting underground structures. But it is assumed that the foundations of many existing highway bridges should be retrofitting, either because they do not satisfy the new specification or because the retrofitting of bridge piers above foundations has resulted in their strength becoming relatively small.

The methods used to retrofit existing bridge foundations in Japan have been the addition of piles, relatively large-diameter piles, and the expansion of a footing. The following is the present state of pile construction methods in Japan. Figure-1 shows the trends in the frequency of the use of various pile construction methods [1)]. Pile foundations account for about 60 % of all foundations constructed under highway bridges, and most of these are cast-in-place piles. Figure-2 shows the piles used in Japan broken into by their diameter. As this figure indicates, most piles used as bearing piles in Japan have a diameter of about 1 m. But because large machinery is necessary to execute piles of this size, foundation retrofitting work is extremely difficult to execute and is very costly because of restrictions imposed by the space under girders, executions close to existing structures, and right-of-way boundaries.

In America, the use of micropiles for seismic retrofitting of existing bridge foundations has been increasing in recent years (Figure-3). Micropiles permit the reduction of the expansion of a footing and can be executed where the headroom and work yard space are restricted thanks to the small size of micropile execution equipments. For these reasons, micropiles are expected to be used as foundation retrofitting methods in Japan.

Although in Japan, micropiles are used as underpinning for structures and as reinforcement for the bottom of tunnels, they have not been used particularly often and there are no design standards for small diameter piles with a diameter less than 300 mm. Consequently, the authors studied the applicability of micropiles as bridge foundations retrofitting by performing a trial micropile construction followed by vertical and horizontal alternating loading tests to clarify their bearing capacity and deformation properties.

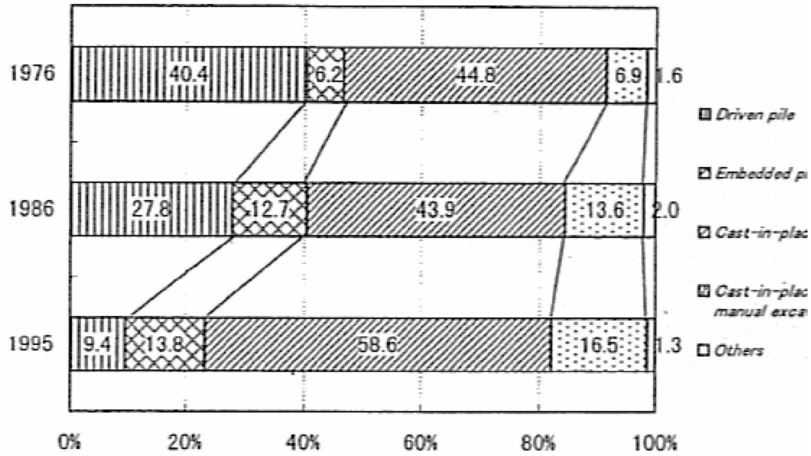

Figure-1 Trend of the Use of Pile Methods

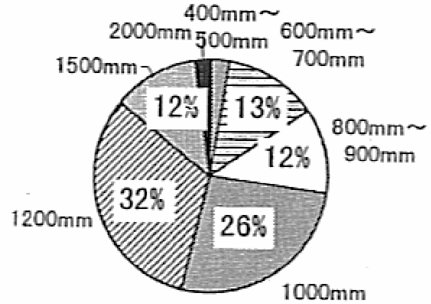

Figure-2 Number of the Use in Pile Diameter

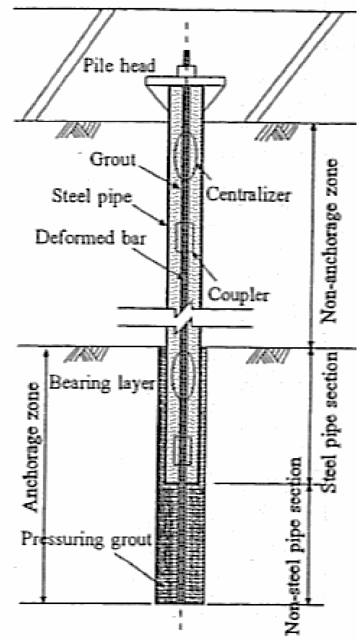

Figure-3 Example of the Structure of Micropile

2. EXECUTION METHODS AND MATERIAL DIMENSIONS OF MICROPILES

Figure-4 shows the execution procedure of micropiles.

[1] Using a steel pipe as a casing, a hole is bored to the bearing layer by successively connecting the steel pipes and inner rods (drill bit at the end of the steel pipe: diameter of 200 mm).

[2] After slime is removed by pure water discharged from the end of the inner rod, the inner

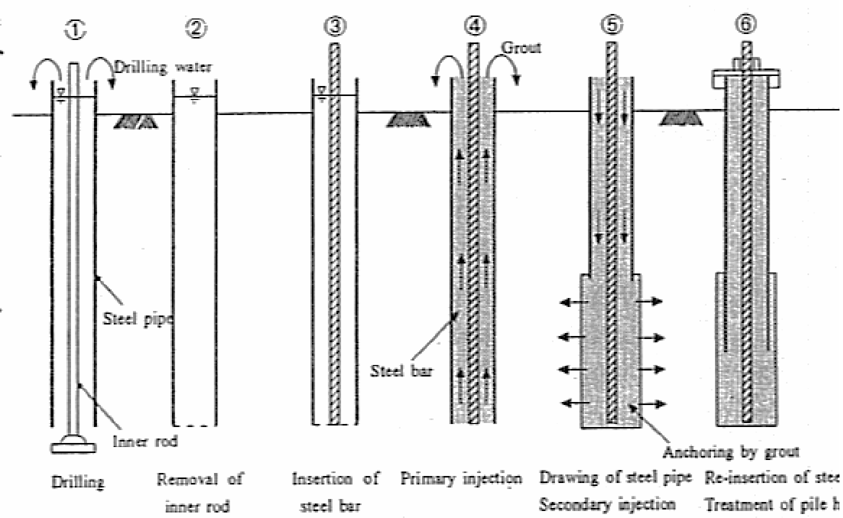

Figure-4 Execution Procedure of Micropile rod is removed.

[3] Deformed bar with centralizers and a grout injection pipe is inserted inside the steel pipe.

[4] Primary injection of grout is performed from the pipe mouth attached to the end of the pile replacing the water inside the hole with grout. After the primary injection is completed, the grout injection pipe is removed.

[5] While the steel pipe is pulled up to the top edge of an anchor part and removed, a packer installed at the top of the pile is used to perform secondary injection (pressurized injection: about 0.5 MPa).

[6] After the secondary injection up to the top edge of the anchor part has been completed, the steel pipe is reconnected as reinserted to the anchor. With reference to execution cases in the United States, the steel pipe is reinserted to the anchor part to minimize the stress generated at the part where the steel pipe is not inserted: structurally the weakest part of the bottom of the pile.

Table-1 Dimensions of Micropile

Material	Standard
Steel pipe	Unjointed steel pipe for oil well (API-N80),external diameter =177.8mm, thickness=12.7mm, length =1,500mm, yield point strength =550N/mm^2 elastic modulus =2.0×10^{5}N/mm^2, threaded coupling joint at both ends
Deformed bar	Thread jointed deformed bars SD490 D51, elastic modulus =2.0×10^{5}N/mm^2 coupler joint with epoxy resin filled
Grout	Cement milk (water-cement ratio W/C=45%), high early strength cement with admixtures design strength =30N/mm^2, elastic modulus = 1.35×10^{4}N/mm^2(experiment results)
Anchorage zone	Effective anchoring diameter = 200mm(drilling diameter), anchoring rength =5.0m

Table-1 presents the dimensions of the micropiles used for this study. The steel pipe was high strength oil well pipe (API-N80), and to confirm the execution properties in low space, they were linked with screw type couplings at intervals of 1.5 m.

3. VERTICAL ALTERNATING LOADING TEST OF MICROPILE

3-1. Ground Conditions at Test Location

Figure-5 shows the soil borehole log of the test location ground. The soil of the test location ground consisted of alternating layers of loam, fine sand, clay etc., and the anchor layer selected was a relatively compacted fine sand layer with the N value of 30 or more at the depth of GL -10.5 m and deeper.

3-2. Method of Vertical Alternating Loading Test

Figure-6 shows the measurement locations during

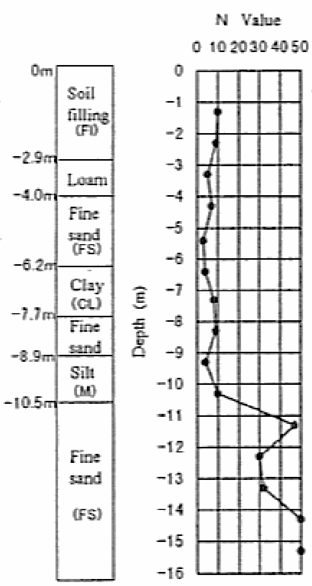

Figure-5 Soil Boring Log

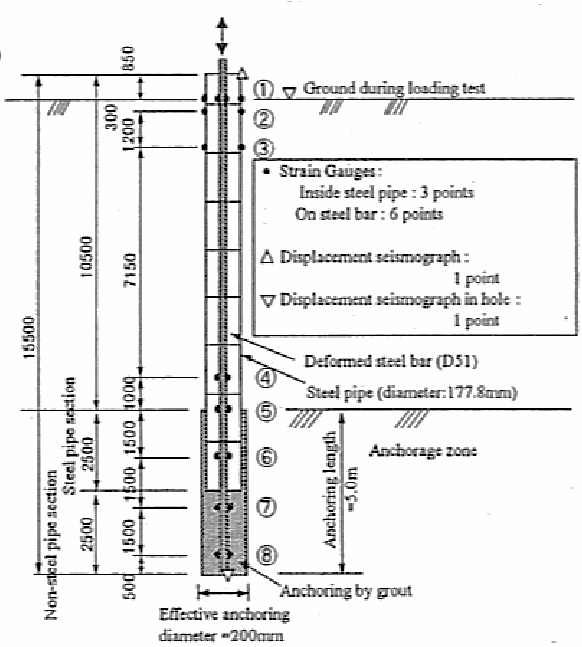

Figure-6 Locations of Measurement

the loading test. Strain gauges were placed at three sections on the inside of the steel pipe and at 6 sections on the steel bar. The loading was done by alternating push-in - pull-out loading to the non-linear range in the load - displacement relationship in order to clarify the deformation properties in both ranges, continuously, by monotonic pull-out loading to the ultimate state.

3-3. Results of Vertical Alternating Loading Test

Figure-7 shows the load - pile head displacement hysterisis curve. The results indicate the yield pull-out bearing capacity of 900 kN and ultimate pull-out bearing capacity of 1,050 kN, revealing that the ultimate state was reached at the quantity of deformation equal to approximately 10 % of the pile diameter. The gradient of the push-in and pull-out up to near the yield was uniform and the axial direction spring constants in both directions were almost identical.

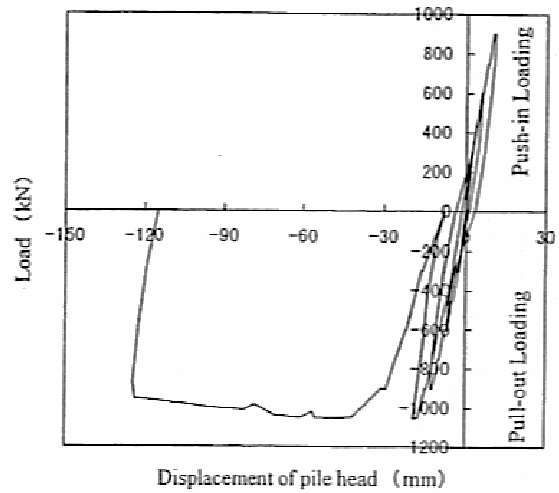

Figure-7 Hysteresis Curve of Load-Pile Head Displacement

Figure-8 shows the axial force distribution in the depth direction. In the United States, the effective diameter of the anchor part (sections ⑤ to ⑧) is widened beyond the drilled hole diameter by means of pressurized injection, but because the relationship between ground conditions and an effective diameter has not been fully clarified in Japan, the drilled hole diameter was treated as the effective diameter for this study. This figure reveals that the axial force fluctuates relatively little from section ① to section ⑤ at each loading stage and that although a little axial force is retained in the parts above 0.5 m from the tip of the pile (section ⑧), its values during push-in and during pull-out are almost identical. This fact confirms that the friction of the non-anchor part and the tip bearing capacity of a micropile are low and that the bearing mechanism of a micropile is supported mainly by the friction of the anchor part. Consequently, it has been considered important for the design to incorporate the evaluation of the surface friction strength of the anchor part. The specification for highway bridges [2] stipulate that the maximum surface friction strength of a cast-in-place pile shall be 200 kN/m^2, but the results of this loading test, it is between 1.5 and 2.4 times that value, revealing that the anchor part provides the friction resistance greater than that of conventional construction methods.

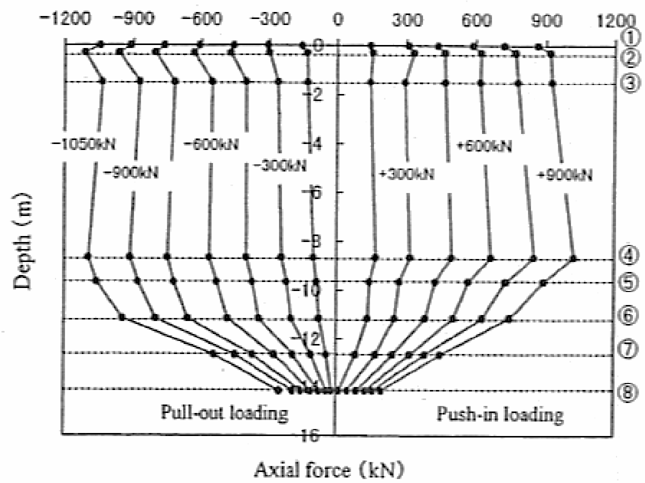

Figure-8 Axial Force Diagram

4. HORIZONTAL ALTERNATING LOADING TEST OF MICROPILE

4-1 Ground Conditions at Test Location

Figure-9 shows the soil borehole log of the ground at the test location, and the soil constants obtained from the triaxial compression test and lateral loading test in the borehole.

4-2. Method of Horizontal Alternating Loading Test

Figure-10 shows the measurement locations during the loading test. Strain gauges were placed at five sections on the inside and outside of the steel pipe and at 6 sections on the steel bar. The loading, which was alternating loading, was performed by two center hole jacks with a jack stroke of 200 mm in order to perform large deformation loading.

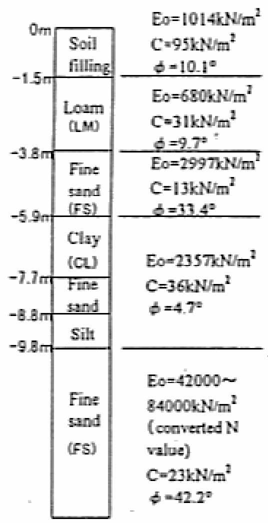

Figure-9 Soil Boring Log

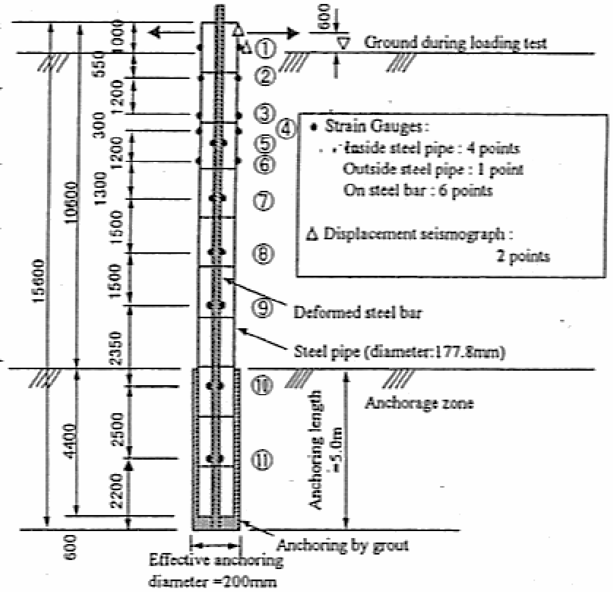

Figure-10 Locations of Measurement

4-3. Results of Horizontal Alternating Loading Test

Figure-11 shows the horizontal load - pile head displacement hysterisis curve. In the final cycle, the maximum horizontal load of 160 kN was loaded in the positive direction causing displacement of 355 mm in the pile head, but it does not reveal any clear point of the change nor any points where the residual displacement increased abruptly in the load - displacement relationship.

In order to confirm the compatibility of micropiles with a small diameter compound structure consisting of a steel pipe, steel bar and grout with the current specification for highway bridges, the static non-linear analysis of a single micropile were conducted. For this analysis, the ground was assumed to be completely elasto-plastic and the M- ϕ relationship of the pile body was set with reference to the results of the bending test in atmosphere of a micropile with identical dimensions [3]. The horizontal load - pile head displacement curve shown in Figure-11, along with the loading test results, shows the analytical results. For the analytical result-1, the results of triaxial compressive test performed in various soil layers and the lateral loading test inside the borehole were used to set the soil constants (Figure-9). And for the analytical result-2, the soil constants were set based on the

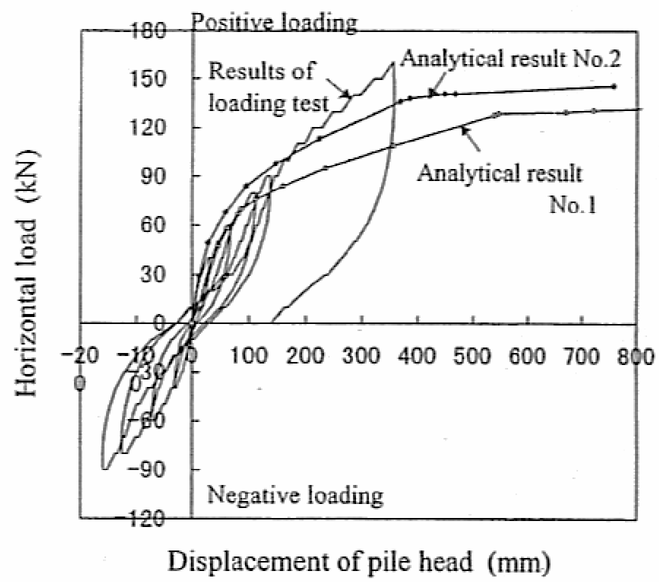

Figure-11 Hysteresis Curve of Load-Pile Head Displacement

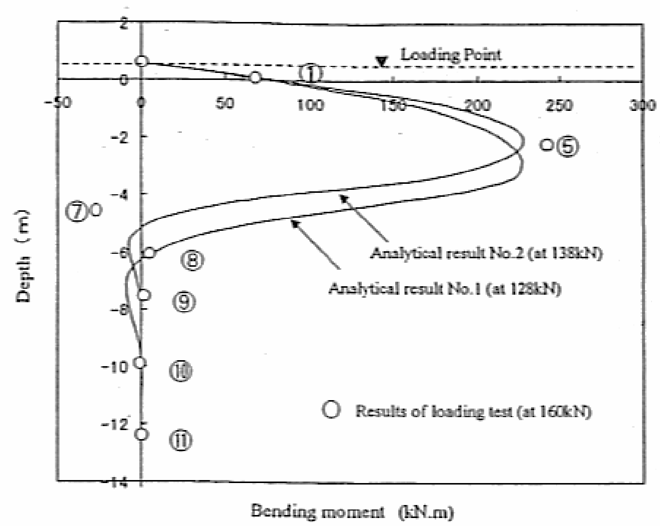

Figure-12 Bending Moment Diagram

results of these soil tests plus the results of static cone penetrating test (CPT) that was performed to measure soil data continuously. Although there were the differences caused by the evaluation method of ground properties, these analytical results are more on the safe side than the actual measured values. Figure-12 plots the bending moment values during maximum loading, indicating the distribution of the bending moments at the time the pile body achieved the total plastic moment in both analysis cases. This figure indicates that the measurement values and analytical values of the depth and the state of the maximum bending moment distribution conform relatively closely. Because there were a few measurement points on the pile body and the continuous depth direction data was unobtainable, it is difficult to say that these are adequate verification results, but it is possible to design micropiles using a beam - spring model based on the existing design method.

5. APPLICABILITY TO RETROFITTING OF EXISTING FOUNDATIONS

In order to study the applicability of micropiles to the seismic retrofitting of existing foundations, it is essential to confirm the extent to which the lateral strength of an existing foundation can be raised by installing micropiles. So a trial calculation of the retrofitting of an existing foundation was performed based on the ductility design method accounting for the bearing capacity properties of micropiles obtained from these loading tests.

5-1. Method of Trial Calculation

The trial calculation was performed by hypothesizing that the lateral strength of an existing bridge foundation was inadequate in only the longitudinal direction and determining the values of all pile dimensions and the numbers of piles required in order to increase the lateral strength of 1.5 times using a conventional method and micropiles. The model of an existing foundation was an ordinary bridge foundation consisting of cast-in-place piles (diameter; 1,200 mm) in the reference [4]. And because this trial calculation was focussed on increasing the lateral strength of a foundation by increasing piles, the ground resistance of the front surface of a footing was ignored and it was assumed that existing piles were undamaged.

5-2. Cases of Trial Calculation

Table-2 Cases of Calculation

CASE	Contents	method	Share of Horizontal Resistance by Pile groupe				
			Row M1	Row C1	Row C2	Row C3	Row M2
A	Existing piles only (before retrofitting)	—	—	1.0	0.5	0.5	—
B	Retrofitting by conventional method	Cast-in-place piles (diameter 1,200mm)	1.0	0.5	0.5	0.5	0.5
C	Retrofitting by micropiles	Micropiles(diameter 177.8mm)	1.0	0.5	0.5	0.5	0.5
D			0.5	1.0	0.5	0.5	0.5
E			0.2	0.8	0.5	0.5	0.5

(But 1.0 regardless of pile row in clay)

As shown in Table-2, the trial calculation was performed for five cases. Case-B that was performed using the conventional retrofitting method which is, as stated in the introduction, the method in commonest use in Japan, involves adding new piles with the same dimensions as the existing piles (cast-in-place piles with a diameter of 1,200 mm). The structure of the micropiles was identical to the dimensions of the piles used the loading test, but with regard to the sharing factor of ground lateral resistance, three patterns (Cases-C to -E) were set. Under the current specification, the share of the load borne by piles in the back row is smaller than that of the front row as the consequence of pile group effect, and the lateral resistance factor of the piles at the rear was set as 1/2 of that of the front. But this value were set based on the pile group experiments[5)] for the case where all piles are same dimensions, and in the case of pile groups consisting of small diameter piles such as micropiles and of relatively large diameter existing piles, the lateral resistance factor will differ from the values stipulated by the current specification. Consequently, the effects of pile groups consisting of differing types of piles were studied by setting these three lateral resistance factor: Case-C where the lateral resistance of the micropiles in the front row (row M1) was forecast to be 100 %, Case- D that considered the lateral resistance of the existing piles (row C1) to be 100 %, and Case-E that considered the micropiles to be 20 % and the existing piles to be 80 %. But the lateral resistance factor in clay was, in conformity with the current specification, set at 1 regardless of the row.

5-3. Conditions of Trial Calculation

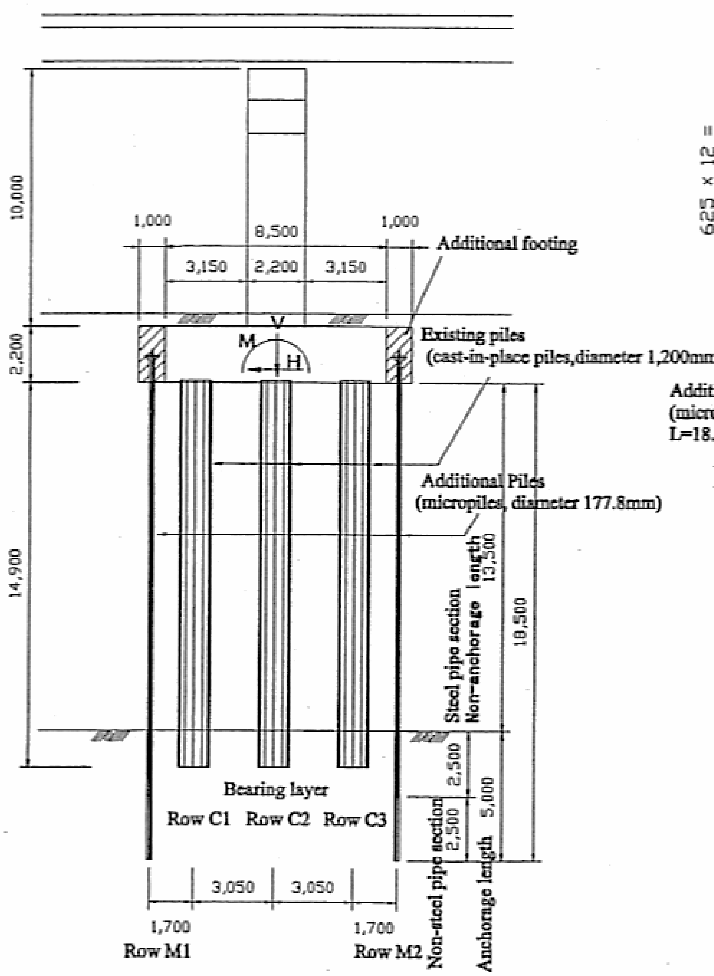

Figure-13 Reinforcing Section by Micropile

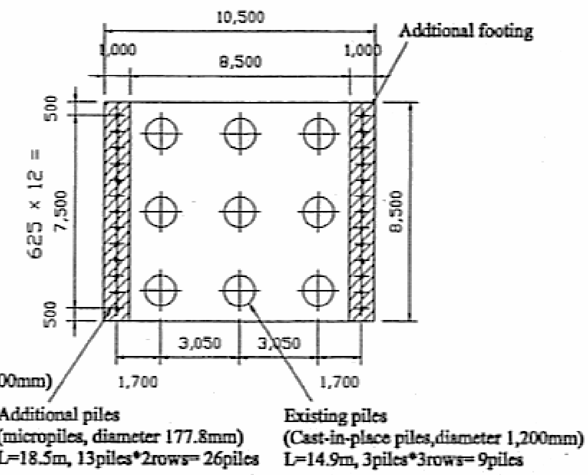

Figure-14 Reinforcing Plan by Micropile

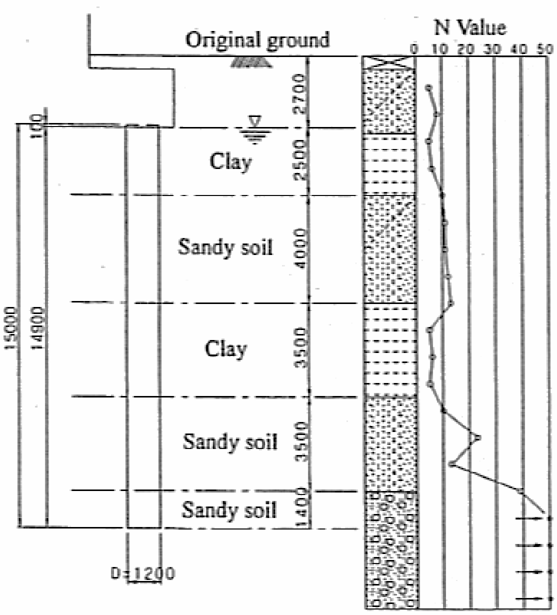

Figure-15 Soil Boring Log

The following are the conditions set for the retrofitting with micropiles. The conditions set for retrofitting using the conventional method (cast-in-place piles) are omitted.

[1] Micropiles dimensions

Table-3 shows the micropiles dimensions and conditions. The pile diameter and materials are

Table-3 Setting Conditions of Micropiles

Item	Unit	Quantity	Remarks
Pile diameter (Non-anchorage zone)	mm	177.8	Diameter of steel pipe
Pile diameter (anchorage zone)	mm	200.0	Diameter of bore hole
Pile length	m	18.5	
Non-anchorage length	m	13.5	
Anchorage length	m	5.0	Same specifications as loading test
Ultimate push-in bearing capacity	kN	1,050	Loading test value
Ultimate pull-out bearing capacity	kN	1,050	Loading test value
Axial direction spring constant	kN/m	70,367	Loading test value
Horizontal coefficient of ground reaction			Calculated based on reference[4]
Layer No.1	kN/m^3	68,610	
Layer No.2	kN/m^3	137,220	
Layer No.3	kN/m^3	68,610	
Layer No.4	kN/m^3	205,830	
Layer No.5	kN/m^3	686,110	
Ultimate moment of pile body	kN.m	218.52	Acting axial force N = 42kN/pile
Ultimate curvature of pile body	1/m	0.040969	Accounting for steel pipe,grout and bar

identical to those used for the loading tests and the anchoring length is set at 5 m in the bearing layer with the N value in excess of 30.

[2] Micropiles arrangement

Figure-13 is the cross section of the foundation retrofitting with micropiles, Figure-14 is the plane diagram of micropiles, and Figure-15 is the soil borehole log. The retrofitting work was done by placing micropiles on both sides of the existing footing, with increasing the width of the footing by 1.0 m. The intervals between the existing piles and the micropiles was set at the average of 2.5 times the diameter of the existing piles and the micropiles.

[3] Axial direction spring constant and upper limit of bearing capacity

Because, for this trial calculation case, the anchoring was identical to that in the loading test and it was placed in sandy ground identical to that in the loading test, the axial direction spring constant and the upper limit of the bearing capacity were both identical to those obtained from the results of the loading test. And because the bearing capacity of the tip of micropiles was small, the upper limits of push-in and pull-out bearing capacity were assumed to be identical.

[4] Bending moment M - curvature ϕ relationship of a micropile

The resistance properties of a micropile are governed by a steel pipe, but according to the reference 3), the effects of grout and a steel bar do increase resistance. Therefore, the M-ϕ relationship of a micropile that represents the resistance properties of a pile body is assumed to be bilinear to account for all constituent members of a pile body.

[5] Upper limit of lateral ground reaction

Table-4 shows the upper limit of lateral ground reaction. The upper limits of the lateral ground reaction actually used for each pile row are set as follows accounting for the lateral resistance factor referred to above.

$$P_{HU} = \eta_P \alpha_P \times P_u \times \lambda$$

P_{HU}: the upper limit of lateral ground reaction (kN/m^2)

$\eta_P \alpha_P$: the correction factor accounting for both single pile and pile group effect at right angles to the loading direction of each pile

P_u: Passive earth pressure strength (kN/m^2)

λ : the lateral resistance factor in the loading direction of each pile (see Table-2: but in clay layers, it is unrelated to the pile row 1)

Table-4 Ground Constants

	Ground Type	Layer Thickness (m)	Average N Value	Cohesiveness C(kN/m²)	Angle of Internal Friction ϕ (°)	Unit Weight γ' (kN/m³)	Passive Earth Pressure Coefficient K_{EP}	Passive Earth Pressure Strength P_p(kN/m²)	Upper Limit of Horizontal ground reaction coefficient	
									$\eta_p \alpha_p$	P_{HU}(kN/m²)
Layer No.1	Clay	2.5	5	30.0	0	8.0	1.000	105.90 125.90	1.500	158.85 188.85
Layer No.2	Sandy Soil	4.0	10	0.0	27	8.0	3.035	200.03 297.16	3.000	600.09 891.48
Layer No.3	Clay	3.5	5	30.0	0	8.0	1.000	157.90 185.90	1.500	236.85 278.85
Layer No.4	Sandy Soil	3.5	15	0.0	30	10.0	3.505	441.30 563.98	3.000	1323.90 1691.94
Layer No.5	Sandy Soil	5.0	50	0.0	40	10.0	5.996	964.76 1244.56	3.000	2894.28 3733.68

5-4. Results of Trial Calculation

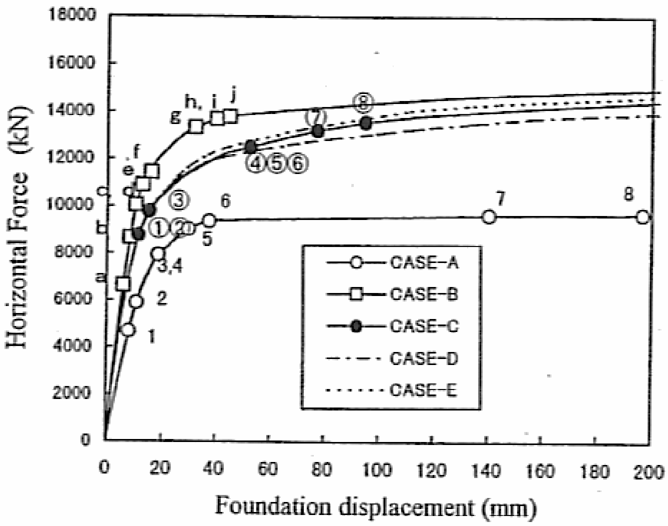

Figure-16 Trial Calculation Results

(Relationship of Horizontal Force - Displacement)

Table-5 Trial Calculation Results

State of Foundation (Case-A) : Before Retrofitting

Marks	Horizontal Force (kN)	Horizontal Displacement of Foundation (mm)	State of Each pile
1	466	7.98	Row C3 - Pile body yield
2	580	10.51	Row C3 - Pile body ultimate
3	780	17.33	Row C3 - Pull-out ultimate
4	790	18.03	Row C2 - Pile body yield
5	904	29.59	Row C1 - Pile body yield
6	937	36.43	Row C3 - Push-in ultimate
7	964	140.12	Row C1 - Pile body ultimate
8	968	196.74	Row C2 - Pile body ultimate

State of Foundation (Case-B) : Retrofitting by conventional method

Marks	Horizontal Force (kN)	Horizontal Displacement of Foundation (mm)	State of Each pile
a	632	5.57	Row M2 - Pile body yield
b	866	8.29	Row M2 - Pile body ultimate
c	1,005	10.35	Row C3 - Pile body yield
d	1,005	10.35	Row C2 - Pile body yield
e	1,089	12.62	Row M1 - Pile body yield
f	1,142	14.67	Row C1 - Pile body yield
g	1,338	32.99	Row M1 - Pile body ultimate
h	1,370	40.01	Row C1 - Pile body ultimate
i	1,373	40.91	Row C3 - Pile body ultimate
j	1,373	40.91	Row C2 - Pile body ultimate

State of Foundation (Case-C) : Retrofitting by micropiles

Marks	Horizontal Force (kN)	Horizontal Displacement of Foundation (mm)	State of Each pile
①	878	11.29	Row C3 - Pile body yield
②	878	11.29	Row C2 - Pile body yield
③	994	15.96	Row C1 - Pile body yield
④	1,218	41.74	Row C1 - Pile body ultimate
⑤	1,221	42.42	Row C3 - Pile body ultimate
⑥	1,221	42.42	Row C2 - Pile body ultimate
⑦	1,321	77.05	Row M1 - Pile body ultimate
⑧	1,356	94.58	Row M2 - Pile body ultimate

As the results of the trial calculation, Figure-16 shows the horizontal force - displacement relationship of the foundations, and Table-5 (Case-A, -B, and -C) shows the state of the pile at each loading stage. Regardless of slight differences in the increase in the lateral strength in each case, almost identical retrofitting effects are obtained by the 2 piles on each side by retrofitting based on the conventional method (Case-B) and with the 13 piles on each side based on the micropile method (Case-C).

The comparison of Case-C, Case-D, and Case-E which are distinguished by varying lateral resistance factor reveals that the increase in the lateral strength in Case-D that considered the lateral resistance of the existing piles to be 100 % is greater and the increase in the lateral strength in Case-E is slightly lower than that of Case-C which considered the lateral resistance of the front row of the micropiles to be 100 %. It is assumed that the results are influenced by the fact that the first ground layer that determines the lateral resistance is a clay layer, but there is little difference in the degrees of the increase of the lateral strength according to the lateral resistance factor. We will clarify the lateral resistance factor based on some experiments of various kinds of piles for a future study.

Next, the comparison of the failure states of the foundations (Table-5) reveals that the failure in Case-A (the existing foundation model) occurred as follows: the yield, ultimate state, and pull-out bearing capacity of the tension side pile (C3), yield of all piles, followed by the push-in bearing capacity of the compression side pile (C1), resulting in the limit state of the foundation. In Case-C (the retrofitting model by micropiles), on the other hand, after all the existing piles yielded and reached the ultimate state, the micropiles on the compression side then on the tension side reached the ultimate state so the foundation lost its resistance function. The increase in the number of micropiles raised the rocking resistance, so that the micropiles did not reached in the pull-out or push-in bearing capacity, but because the ultimate state of the foundation was reached based on the ultimate state of the micropiles, the seismic design method must confirm the ultimate state of a foundation accounting not only for the rocking resistance of a foundation, but also for the lateral and bending resistance of a pile body.

6. Conclusions

This report introduces micropiles that are expected to be applied for the seismic retrofitting of existing foundations and describes the trial execution and loading tests of micropiles which was conducted to study the applicability of micropiles as a foundation retrofitting method. The results of the trial calculation showed that although a relatively large number of piles are needed, even micropiles with small diameter provide a certain degree of the effects of foundation retrofitting. But, we will study the effects of micripiles retrofitting, confirming the share of lateral resistance, as the fact above mentioned, concerning with the difference of the diameter and stiffness between existing piles and micropiles.

The comparison of the conventional retrofitting method and micropiles method reveals that, because the pile diameter and lateral strength of a micropile are both low, in cases where it is necessary to obtain the identical quantity of the increase in the lateral strength, enough piles to provide the pile diameter ratio identical to that of the conventional method are necessary, regardless of the sectional area ratio. But, because the increase in the width of a footing is small using micropiles method, aside from additional pile execution work, less structural excavation work and less footing construction work is required than when using conventional retrofitting method. In future, we must study that whether micropiles can apply to retrofitting method, in respect to economicability for all of retrofitting work. We showed the calculation results accounted fully for the loading test results, hereafter, we will study for micropiles with slightly larger steel pipe diameters, anchoring methods, and inclining micropiles and so on.

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