STATIC ALTERNATING CYCLIC HORIZONTAL LOAD TESTS ON DRIVEN STEEL PIPE PILES OF FOUNDATIONS FOR HIGHWAY BRIDGES

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Two bridges were constructed for extension work of Central Hokkaido Connection Road in 2005 at two different sites. Pile group foundations were employed for the foundations of abutments of the two bridges. In order to assess the performance of the constructed piles in the sites and to obtain design parameters for design in future, static alternating cyclic horizontal load test was carried out on one of the open-ended steel pipe piles constructed at each site. The ground at one site was characterised as a thick deposit of volcanic soils, and the ground at the other site was characterised as a very soft peat layer near the surface. In this paper the results of static alternating cyclic horizontal load tests on the piles at the two sites are presented and discussed in detail.

INTRODUCTION

A highway route of 80 km called Central Hokkaido Connection Road that connects Chitose City and Otaru City is under construction. Several bridges are planned to be constructed for the highway. Pile groups were selected for foundations of the bridges, since volcanic ash and peat soils distribute over the area of the highway route. Foundations of highway bridges in Japan are designed according to the *Specifications for Highway Bridges* (Japan Road Association, 2002). In the *Specifications*, although design of piles in clayey and sandy soils is prescribed, the design of piles in volcanic ash and peat soils is not definitely prescribed. Therefore, design of piles in volcanic ash soils is made with an assumption that volcanic ash soil is categorised as sandy soil at present.

Mechanical properties of volcanic ash are different from those of sand due to high crushability and corresponding high compressibility of volcanic ash (for examples, Kitamura 1999, Miura and Yagi 2002). Hence, attention will be paid in the application of the *Specifications* to design piles in volcanic ash soils.

Another problem in pile design for this highway is the existence of peat soil. It has very high water content and high compressibility that results in very low horizontal pile resistance within allowable horizontal displacement of the pile. Therefore, soil improvement is often needed for peat soils in order to gain appropriate pile performance until the design horizontal load.

During construction of Central Hokkaido Connection Road, horizontal load tests of steel pipe piles were carried out at two sites, Kiusu site and Shinotsu site, in 2005, to confirm performance of the constructed piles and to get appropriate design parameters for future construction of foundation piles for the highway bridges. Volcanic ash soil distributes at Kiusu site, while peat exists at Shinotsu site. The peat soil layer at Shinotsu site was improved to increase its strength and stiffness.

In this paper, results of the static horizontal load tests at the two sites are presented and discussion will be made on design of piles in such particular soils.

DESIGN METHODS FOR PILES FOR BRIDGE FOUNDATION

As mentioned earlier, pile foundations of highway brides in Japan are designed according to the *Specifications for Highway Bridges*. It may be appropriate here to introduce the *Specifications* briefly for later discussions.

Vertical Bearing Capacity

The vertical bearing capacity, R_u, of a pile is calculated as

$$R_u = q_d A + U \sum L_i f_i \qquad (1)$$

The first and the second terms of of Equation (1) are the toe capacity and the shaft friction capacity, respectively. In Equation (1), q_d is the toe resistance per unit area, A and U are the cross-sectional area and the circumference the pile, and L_i and f_i are the length and the shaft resistance per unit area of soil layer i. The shaft resistance f is calculated from SPT N-value or cohesion c of the soil as follows:

$$f_i = 2N \text{ kPa} \quad (\text{max.} = 100 \text{ kPa}) \text{ for sand} \qquad (2)$$

$$f_i = c \text{ or } 10N \text{ kPa} \quad (\text{max.} = 150 \text{ kPa}) \text{ for clay} \qquad (3)$$

The toe resistance per unit area q_d of driven open-ended steel pipe pile is given by

$$q_d = 60N \times (L_d / D) \text{ kPa} \quad (\text{max.} = 1500 \text{ kPa}) \qquad (4)$$

where L_d is embedment length of the pile toe into a bearing stratum having N greater 30.

Two types of external loads, usual load and earthquake load, are considered in design. Design pile capacity is estimated with safety factors of 3 and 2 for usual load and earthquake load, respectively. If a pile load test is carried out at the site, the safety factors can be reduced to 2.5 and 1.7, respectively, for usual and earthquake loads.

Horizontal Resistance

The horizontal resistance of a pile is usually determined from allowable horizontal displacement of the pile head. The use of Equation (5) is prescribed in the *Specifications*.

$$u = \{\exp(\beta z) \cdot (C_1 \cos \beta z + C_2 \sin \beta z) + \exp(-\beta z) \cdot (C_3 \cos \beta z + C_4 \sin \beta z)\} / (2EI\beta^3) \qquad (5)$$

In Equation (5), z is the distance from the pile head and the characteristic length β is given by

$$\beta = \left(\frac{k_H D}{4EI} \right)^{-1/4} \qquad (6)$$

where k_H is the coefficient of horizontal subgrade reaction, and D, E and I are the diameter, Young's modulus and the geometric moment of inertia of the pile. Coefficients C_1 to C_4 are determined from boundary conditions such as rotation of the pile head and horizontal load and moment given at the pile head as well as boundary conditions at depth β.

The value of k_H is estimated by

$$k_H = k_{H0} \left(\frac{B_H}{0.3} \right)^{-3/4} \text{ kPa/m} \qquad (7)$$

where k_{H0} and B_H are given by the following equations:

$$k_{H0} = \frac{\alpha}{0.3} E_0 \text{ kPa/m} \qquad (8)$$

$$B_H = \sqrt{D / \beta} \qquad (9)$$

In the above equations, E_0 is Young's modulus of soil and α is correction factor dependent on method used in estimation of E_0. For examples, α = 4 if E_0 is estimated from borehole lateral pressure test, and α = 1 if E_0 is estimated from an empirical equation (10).

$$E_0 = 2800N \text{ kPa} \qquad (10)$$

KIUSU SITE

Test Site and Test Pile

Figure 1 shows the profiles of soil layers and SPT N-values at Kiusu site. The soil profile at this site is characterised by thick deposits of volcanic soils. SPT N-values are typically less than 7, except for gravel at depths from 17.5 m to 21.2 m and volcanic soils at depths from 27.5 m to 31 m. No definite bearing stratum having N greater than 30 can be found to a depth of 32 m.

Two abutments were constructed here. Each abutment was supported by 40 driven open-ended steel pipe piles. Pile design for the vertical load and the horizon load are summarised in Tables 1 and 2, respectively.

The maximum values of design vertical loads on piles were 1198 kN and 2267 kN for usual and earthquake loads, respectively. The design pile capacities considering safety factors were 3594 kN and 4534 kN without pile load test, and 2876 kN and 3627 kN with pile load test, for usual and earthquake loads. Finally, piles were designed to have the pile capacity of 4534 kN at the first stage of pile design.

The horizontal loads on a pile in the pile group were 187 kN and 555 kN, respectively, for usual and earthquake loads. The design requirement was that the horizontal pile head displacement should not exceed 15 mm for the horizontal load of 555 kN.

The specifications of the pile were determined as shown in Table 3 in order to satisfy the above design requirements. The expected pile capacity for vertical loading was 4880 kN and the expected pile head displacement was 11.4 mm for the design horizontal load of 555 kN.

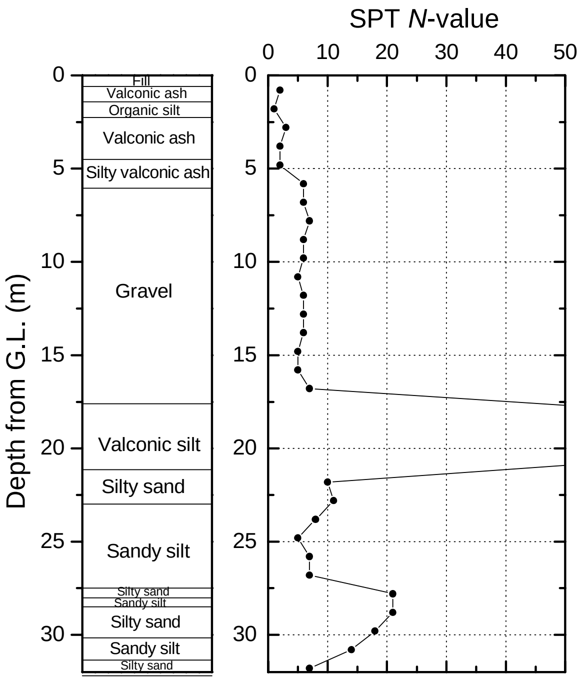

Figure 1 Profiles of soil layers and SPT *N*-values at Kiusu site

Table 1 Design of pile for vertical load at Kiusu

	Usual	Earthquake
Design vertical load on abutment (kN)	43295	43558
Number of piles	40	
Design vertical load on pile (kN)	978 to 1198	-89 to 2267
Safety factor	3	2
Design pile capacity (kN)	3594	**4534**
Safety factor with pile load test	2.5	1.7
Design pile capacity with pile load test (kN)	2995	**3854**

Table 2 Design of pile for horizontal load at Kiusu

	Usual	Earthquake
Design total horizontal load on the abutment, *H* (kN)	7495	22218
Design horizontal load on a pile (kN)	187	**555**
Horizontal pile head displacement at design load (mm)	4.5	11.4
Allowable horizontal pile head displacement (mm)	15	15

Table 3 Specifications of steel pipe pile

Property	Value
Length (m)	30.0
Embedment length (m)	29.2
Outer diameter (mm)	800
Inner diameter (mm)	770 (upper) 782 (lower)
Cross-sectional area (cm^2)	369.2 (upper) 223.7 (lower)
Young's modulus (kN/m^2)	2.06×10^8
Bending rigidity ($kN\ m^2$)	5.87×10^5 (upper) 3.61×10^5 (lower)
Pile capacity from empirical equation (kN)	4880 (perfect friction pile)

In the design stage, it was assumed that the soil to the depth of the characteristic length, β = 3.8 m, is uniform having a coefficient of horizontal subgrade reaction k_H of 14330 kPa/m, based on the results of borehole lateral pressure tests.

As mentioned above, in the pile design, the volcanic soils at the site were treated as sand, because of the lack of design specifications for volcanic soils in the *Specifications for Highway Bridges*. Therefore, the pile performance was confirmed by performing vertical and horizontal load tests of one of the constructed piles in the site.

Test Method and Test Procedure

Dynamic vertical pile load tests were carried out during pile driving and at 5 hours and one week after the end of pile driving. Static alternating cyclic horizontal load test was carried out 13 days after the pile driving. Horizontal loads were applied to the pile at 0.5 m above the ground surface by means of two hydraulic jacks, and horizontal displacement was measured by means of dial gauges, as shown in Figure 2. Pairs of axial strains were measured at 6 levels down the pile shaft to obtain bending moments down the pile axis.

Figure 2 Horizontal load test set-up at Kiusu

Test Results

(a) Vertical dynamic load test

The vertical bearing capacity of the pile from the dynamic pile load testing was 2100 kN at the end of driving and 2900 kN at 5 hours after the end of pile driving. The pile capacity from the dynamic load test at 7 days after the end of pile driving exceeded 3900 kN showing so-called 'set-up'. This pile capacity satisfies the design pile capacity of 3854 kN for the case of conducting pile load test at the site. Hence, the dynamic load test was stopped at that time.

(b) Horizontal load test

Alternating cyclic horizontal loads shown in Figure 3 were applied to the pile.

Figure 4 shows the relationships between the horizontal load *H* and the horizontal displacement *u*. The residual displacement was measured at full recovery of horizontal load to 0 in each load step, and the elastic displacement was obtained by subtracting the residual displacement from the total displacement measured at the maximum load in each loading step. The residual displacement and the corresponding elastic displacement at the maximum load in each load step are also shown in Figure 4.

It should be noted that the boundary conditions of the head of a pile in the pile group for the abutment and the single pile, which is tested, are totally different. Rotation of the head of the former pile is constrained, while the pile head moment of the single pile is 0 and the pile head rotation is free. The horizontal displacement of a single pile is calculated by the following equation (Japan Road Association 2002):

$$u = (1+\beta)H/(2EI\beta^3) \qquad (11)$$

The horizontal displacement at the design load of 555 kN is calculated as 29.4 mm for the coefficient of horizontal subgrade reaction k_H = 14330 kPa/m that has been used in the pile design. The measured total displacement at *H* = 555 kN is almost equal to this predicted value. Consequently, it was confirmed that the pile satisfies the design requirement for horizontal loading.

Figure 5 shows the distributions of bending strains down the pile shaft measured at the maximum load in each loading step. Here, bending strain is defined as the difference

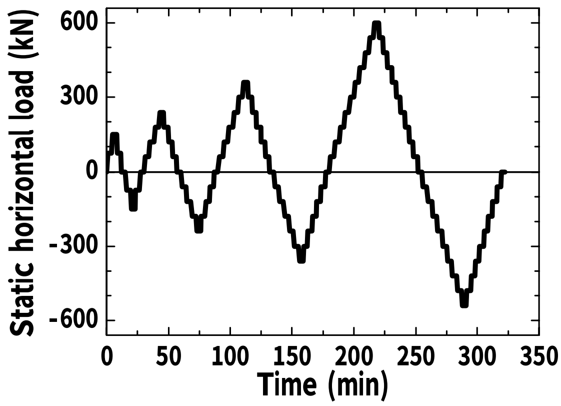

Figure 2 Loading procedure in horizontal load test

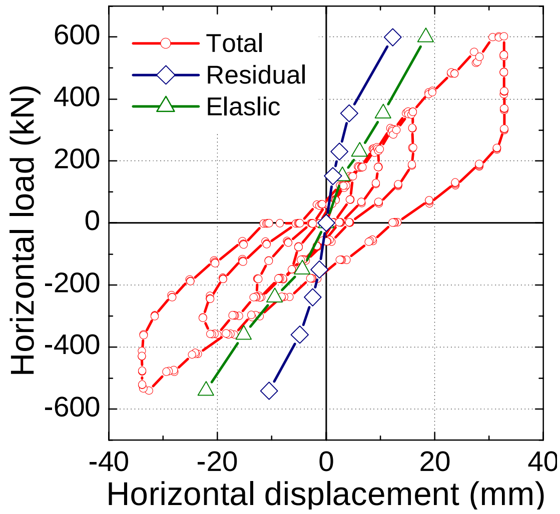

Figure 4 Horizontal load vs horizontal displacement in Kiusu site

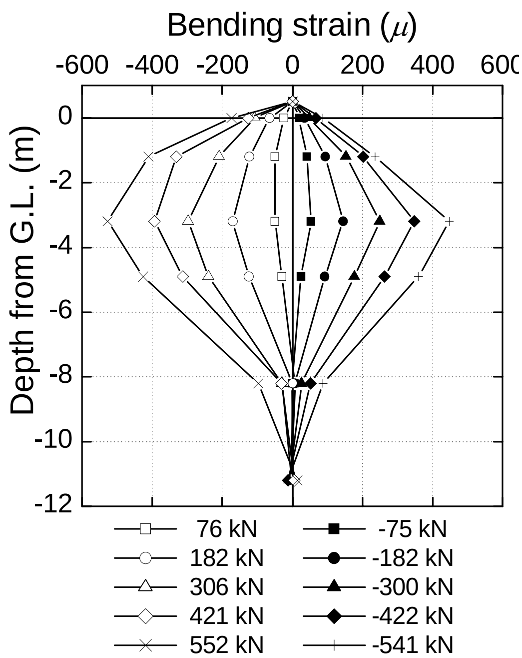

Figure 5 Distributions of bending strains down the pile shaft

between the axial strains measured at the front and the back sides of the pile shaft divided by 2. It can be seen from the figure that the pile body remained in elastic state during the load test and that the maximum bending moment was generated at a depth of about 3 m below the ground surface irrespective of load amplitude.

In order to use the results of the horizontal load test in future, k_H was back-calculated from the measured horizontal load versus total horizontal displacement, and plotted against the normalised horizontal pile head displacement u/D in Figure 6. It can be seen that the k_H from back-analysis decreases exponentially with increasing u/D, and that the value of k_H = 14330 kPa/m that was used in the pile design corresponds to u/D = 3 % (u = 24 mm).

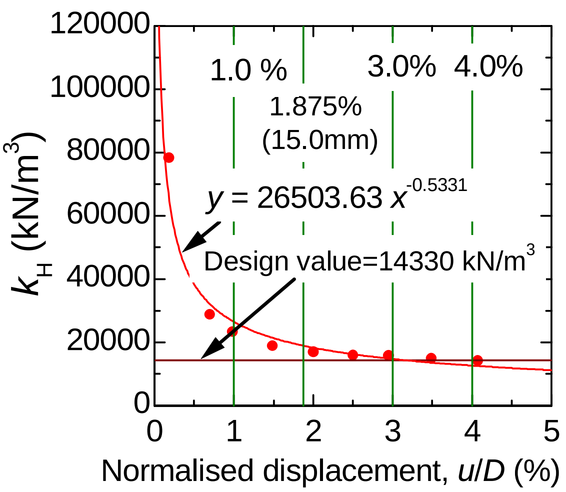

Figure 6 Values of k_H from back-analysis

SHINOTSU SITE

The purpose of the load test at Shinotsu site was similar to that at Kiusu site, i.e. confirmation of the pile design, except for that soil improvements were conducted for the peat soil at Shinotsu site.

Test Site and Test Pile

Figure 7 shows the profiles of soil layers and SPT N-values. A very soft peat exists to a depth of 5 m. Below the peat, very soft soils of clayey silt and organic clayey silt exist to a depth of 19 m, of which N-values are very low from 1 to 2. Soils having N-values from 10 to 30 exist from a depth of 21 m to 36 m. Below a depth of 42 m, gravel having N-values greater than 50 exists. The gravel layer was selected as the bearing stratum for the piles.

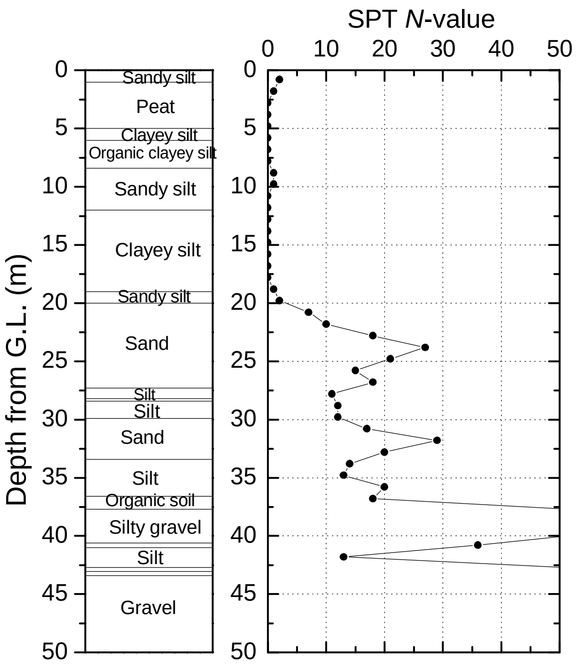

Figure 7 Profiles of soil layers and SPT N-values of the original ground at Shinotsu site

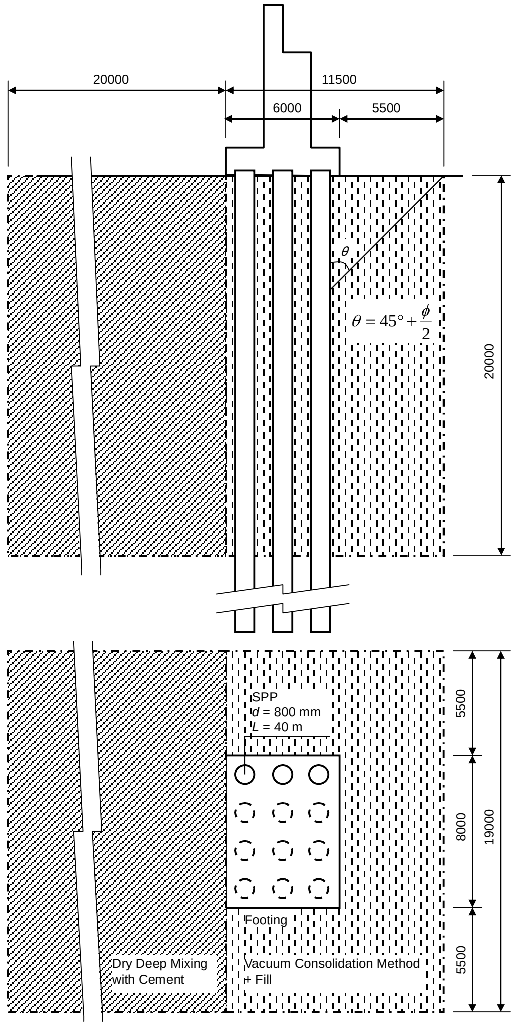

Figure 8 Soil improvements around abutment and piles

Table 4 Design of pile for vertical load at Shinotsu site

	Usual	Earthquake
Design vertical load on abutment (kN)	16665	14623
Number of piles	12	
Design vertical load on pile (kN)	1058 to 1718	-232 to 2669
Safety factor	3	2
Design pile capacity (kN)	5154	**5334**
Safety factor with pile load test	2.5	1.7
Design pile capacity with pile load test (kN)	4295	**4537**

Table 5 Design of pile for horizontal load at Shinotsu site

	Usual	Earthquake
Design total horizontal load on the abutment, H (kN)	3140	7480
Design horizontal load on pile (kN)	262	**623**
Horizontal pile head displacement at design load (mm)	7.2	13.2
Allowable horizontal pile head displacement (mm)	15	15

Table 6 Specifications of pile in Shinotsu site

Property	Value
Length (m)	44.0
Embedment length (m)	43.2
Outer diameter (mm)	800
Inner diameter (mm)	776 (upper) 782 (lower)
Cross-sectional area (cm^2)	297.1 (upper) 223.7 (lower)
Young's modulus (kN/m^2)	2.06×10^8
Bending rigidity ($kN\ m^2$)	4.76×10^5 (upper) 3.61×10^5 (lower)
Pile capacity of the pile from empirical equation (kN)	6199 (end capacity = 2006 kN) (shaft capacity = 4133 kN)

Figure 9 Load test set-up at Shinotsu site

The soil around the piles was improved to a depth of 20 m before construction of piles as shown in Figure 8. Combination of two methods of soil improvement was employed. One of them was vacuum consolidation method with fill on the ground surface. The height of the fill was 10 m (surcharge of 190 kPa). The other method was dry deep mixing with cement. It was assumed that SPT *N*-values to a depth of 20 m increases to 7 times the original *N*-values. Pile design was carried out based on this assumption.

Tables 4 and 5 summarises the pile design against vertical and horizontal loads, respectively. The coefficient of subgrade reaction k_H = 18485 kPa/m was used to the characteristic depth β = 5 m in the pile design for horizontal load, considering the increased *N*-values by the soil improvements. The value of E_0 was evaluated by means of Equation (10) using increased *N*-value expected in the improved soil, because borehole lateral pressure test was not carried out for the improved soil.

It is interesting to note that 21 piles (3 × 7 piles) were required from the pile design without soil improvement. The required number of piles was decreased to 12 piles (3 × 4 piles) by conducting soil improvement prior to pile construction.

<u>Test Method and Test Procedure</u>

Vertical pile load test was not carried out at Shinotsu site, because the pile toe was seated on the very hard gravel layer.

Three piles alone were constructed out of 12 piles required from the pile design. Alternating cyclic horizontal load test was carried out on one of the constructed piles, in order to examine the validity of the pile design. It was intended to change the design of piles, if the results of the horizontal load test did not satisfy the design requirements.

Loading method and measurements of signals were the same as those at Kiusu site. The set-up for the horizontal load test at Shinotsu site is shown in Figure 9.

<u>Test Results</u>

Figure 8 shows the relationships between the horizontal load, H, and the horizontal displacement, u. The residual displacement and the corresponding elastic displacement at the maximum load in each loading step are also shown in Figure 8.

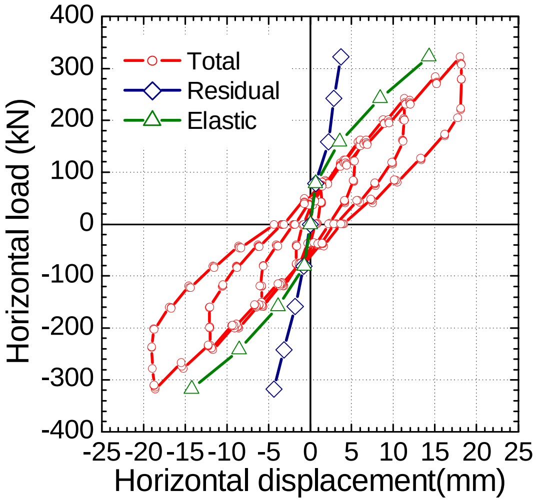

Figure 10 Horizontal load vs horizontal displacement in Shinotsu site

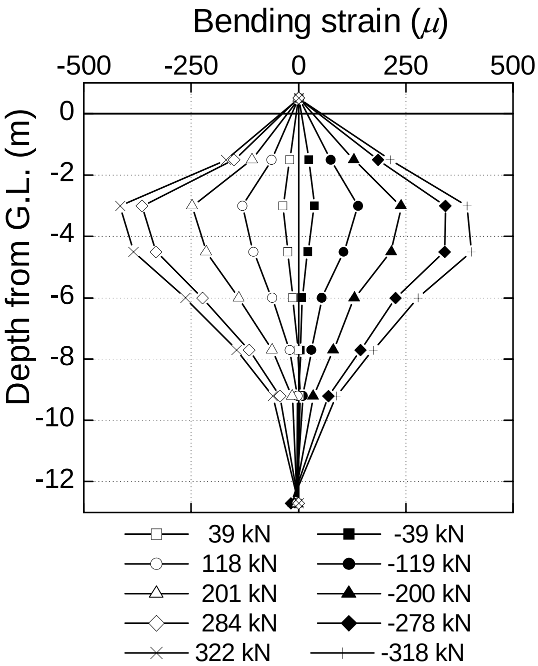

Figure 11 Distributions of bending strains down the pile shaft

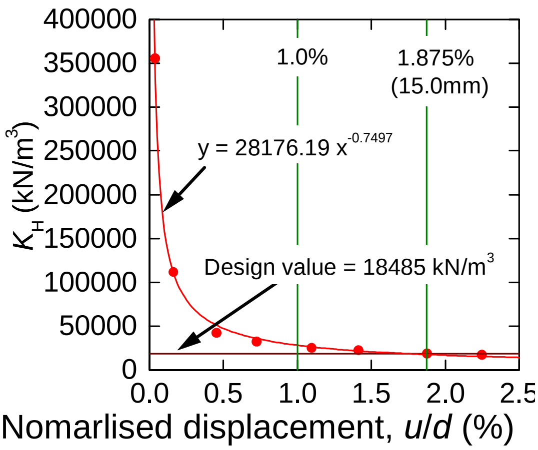

Figure 12 Back-figured values of k_H

It can be seen from Figure 8 that the response of the total horizontal displacement is almost equal to the elastic response.

For the horizontal load test on the single pile in which the moment at the pile head was 0, the predicted horizontal displacement at the pile head was 12.3 mm at horizontal load H = 262 kN (design usual load) and was 29.3 mm at H = 623 kN (design earthquake load). The measured horizontal displacement was 11 mm at H = 262 kN, which satisfied the design requirement. The load test was stopped at horizontal load of 322 kN because the pile displacement exceeded 15 mm at this load and the tested pile was used as actual foundation pile for the abutment.

It may be reasonable to apply the test results until the horizontal load of 623 kN, because the bending strains down the pile axis at H = 322 kN still had enough margin to the bending failure of the pile (Figure 9). Hence, it can be said that the design requirements for the pile in the pile group, of which rotation at the pile head is constrained, for the abutment shown in Table 5 were confirmed from the horizontal load test on the single pile.

In order to use the results of the horizontal load test in future, k_H was back-calculated from the measured horizontal load versus total horizontal displacement and plotted against the normalised horizontal pile head displacement u/D in Figure 10 in the same way as at Kiusu site (Figure 5). It can be seen that the k_H from back-analysis decreases exponentially with increasing u/D, and that the value of k_H = 18485 kPa/m that was used in the pile design corresponds to u/D = 2 % (u = 16 mm).

It is interesting to mention that total cost of the bridge foundation at Shinotsu site was decreased by 20 %, compared with the original design stage in which soil improvement was not employed.

CONCLUSIONS

In this paper, the results of alternating cyclic horizontal load tests on driven open-ended steel pipe piles constructed for foundations of bridge abutments at two different construction sites on the route of the Central Hokkaido Connection Road, Japan, have been presented and discussed. Applicability of the current *Specifications for Highway Bridges* that are widely used in Japan to volcanic soils and improved soils was also discussed, based on the results of the horizontal load tests.

It was confirmed from the load tests that the piles at the sites satisfy the design requirements.

It was also found that the design soil parameters for sand in the current *Specifications* can be applied to volcanic soils which distribute widely in Hokkaido, and that the design soil parameters estimated empirically from SPT *N*-value can be applied to improved soils with enough safety margin.

The design soil parameters in the current *Specifications* can accommodate horizontal pile displacements of more than 2 % of pile diameter, although the allowable horizontal displacement is specified as 1 % of pile diameter or 15 mm. Pile design in which more horizontal pile displacement is allowed could be possible using the design soil parameters in the current *Specifications*, if the design requirements from the superstructure permit it.

The authors are planning to carry out other pile load tests to gather more information on pile design parameters, in order to improve the current pile design towards the framework of the performance based design or the limit state design.

REFERENCES

Japan Road Association, 2002. *Specifications for Highway Bridges Part IV Substructures* (in Japanese).

Kitamura, R., 2000. Topics on Crushable Soil and Crushable Ground in Geotechnical Engineering, *Tsuchi-to-Kiso*, Vol.48, No.10, *Japanese Geotechnical Society*, pp. 3-6 (in Japanese).

Miura, S. and Yagi, K., 2002. Mechanical behaviour and particle crushing of volcanic coarse grained soil in Japan, *Proceedings of Conference on Characterization and Engineering Properties of Natural Soil*, Singapore, pp.1169-1204.