Load History of Cedar Foundation Pile in the Mietsu Naval Facility World Heritage

Takaharu SHOGAKI
Professor, Civil and Environmental Engineering, National Defense Academy, Yokosuka, Japan
Email: shogaki@nda.ac.jp
Daishi OKUDA
Graduate students, Civil and Environmental Engineering, National Defense Academy, Yokosuka, Japan
Email: panappu237@gmail.com

ABSTRACT

The Mietsu Naval Facility (MNF) is a Dutch Western-style naval base constructed in 1858 by the Saga clan at Hayatsue River 6 km north of the Ariake Sea. The MNF is divided into three sections: the shipbuilding/repair docks and metal working section, the training grounds, and the small boat docks section (SBD). All elements have been maintained in largely good condition. Furthermore, archaeological surveys have shown that underground structural remains in this site have been preserved in good condition and have retained their positions. In 2015, the MNF was registered as one of a series of sites that make up the World Heritage Sites of Japan's Meiji Industrial Revolution. The SBD contains traces of large wooden buildings, and excavation of the remains commenced in January 2017.

As part of an engineering investigation of the remains, this study involved the measurement of the consolidation properties of a sample obtained through tube sampling, the undrained shear strength c_u obtained from cone penetration tests for the vertical walls of the trench cut around the pile and changes in the natural water content w_n around the cedar foundation pile. The load history of the cedar foundation pile preserved in the SBD was examined using these measured values to determine the building weight and how the pile was installed.

The changes in c_u and w_n measured in the area around the cedar foundation pile indicated that the cedar pile was driven into the clay ground. The ultimate load was estimated to be 1.15 to 2.15 times the consolidation stress of the layer of soil under the pile. The total energy required to drive the pile to a penetration depth of 30 mm with a hammer of 500 N in weight from a height of 2 m was calculated as 49.7 kN・m, and 50 hammer blows were required to drive the pile to a depth of 1.5 m.

Key words: *World Heritage, pile foundation, load history, ultimate load, total energy, Hasuike clay*

1. Introduction

In the nineteenth century, Saga clan established a Western-style navy and built Western-style ships based on the Western naval knowledge and technologies that it had acquired at the Tokugawa government's Nagasaki Naval Training Institute. The Mietsu Naval Facility (MNF) provided the optimal location for developing such operations. The MNF is thought to have closed down after the abolition of the feudal clans and the establishment of prefectures at the beginning of the Meiji era. The site was used as a maritime academy from 1902 until 1933. During this time, no major changes were made to the ground surface; therefore, a number of remaining structures, which can provide insights into the nature of the MNF when it was in operation, are well preserved underground.

Archaeological surveys and document reviews carried out since 2009 indicate that the Mietsu Naval Dock (MND) was the operation base for the Saga clan navy from 1859 to 1871. The investigations showed that the construction of the Ryofu-maru, Japan's second steamship, was completed at the MND through independent efforts of the Saga clan; and that at the end of the Edo period, it was the first place in Japan where Western-style ships were repaired using a combination of Western and traditional Japanese technologies. In 2015 the MNF was registered as a world heritage site as part of the group of sites named the World Heritage Sites of Japan's Meiji Industrial Revolution: Iron and Steel, Shipbuilding and Coal Mining.

The small-boat docks section (SBD) of the MNF contains traces of large wooden buildings, and excavation of the remains commenced in January 2017. As part of an engineering investigation of the remains, the consolidation properties of a clay sample were obtained through tube sampling. The undrained shear strength c_u was derived from a cone penetration test of the vertical walls of the trench cut around one of the piles and changes in the natural water content w_n were measured around the cedar foundation pile. However, the loading history and the installation method of the cedar foundation piles preserved in the SBD remained unclear.

In this paper, the deformation behavior around the foundation piles is examined based on the movement of soil particles in a laboratory model of Kawasaki clay penetrated by a foundation pile. The experiment results are compared with measurements of the changes of w_n and c_u around the pile in the SBD to infer the installation method of the cedar foundation piles preserved in the SBD.

2. Soil properties of the small-boat docks section site

Fig. 1 shows the layout of the foundation piles found during the excavation at the SBD. Piles A1–A11 are 1.5-m-long cedar piles stripped of bark with a diameter of 20 cm. Piles B1–B11 consist of groups of three pine piles 10 cm in diameter with the bark. Two bore samples were taken using a two-chambered hydraulic piston sampler (45-mm) (Shoghaki and Sakamoto, 2004) according to Japanese standards (JGS, 2015a) at boreholes #6 (Bor. 6) and #7 (Bor. 7), and three portable cone penetration tests (PCPTs) according to Japanese standards (PCPT: JGS 2015b) were taken between the cedar foundation piles. The SBD area comprises a clay layer with 70%–80% shells at a depth of 1.4 m (Tokyo Pail (T.P.) level of -0.7 m) from the excavated ground surface (Shogaki et al., 2018). The 5-m-thick clay layer changes to sand at T.P. level -4.4 m.

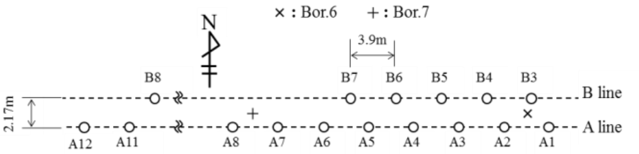


Fig. 1 Layout of the foundation piles at the small boat docks site

The number of standard penetration test blows (N-value) in the clay layer ranged from 3 to 5 (Shogaki *et al.*, 2018).

The c_u values obtained in the CPT from the penetration resistance are plotted against the elevation E in **Fig. 2** The clay at the test site is Hasuike clay. The c_u values at E = 1.5-1.8 m reached 25-50 kN/m^2, caused by the excavations down to about 2 m and the hardened ground surface. The c_u values at E = 0.48-1.58 m were in the range of 10-20 kN/m^2. The horizontal continuity of the Hasuike clay at the SBD site is higher along the A piles (43 m) as shown in Fig. 1. At E = 0-0.3 m the CPT did not penetrate under the weight of two people (=1.08 kN); this layer corresponds to the clay layer with shells observed at Bors. 6 and 7.

The layout of the CPTs around Bor. 6 and their results are shown in **Figs. 3** and **4**, respectively. The variation of the c_u obtained from nineteen CPT tests (A2-1 to A2-19) is smaller than that of Fig. 2 because the former tests covered a smaller area and the c_u values gradually decreased with depth. The mean values $\bar{c}_u$ of

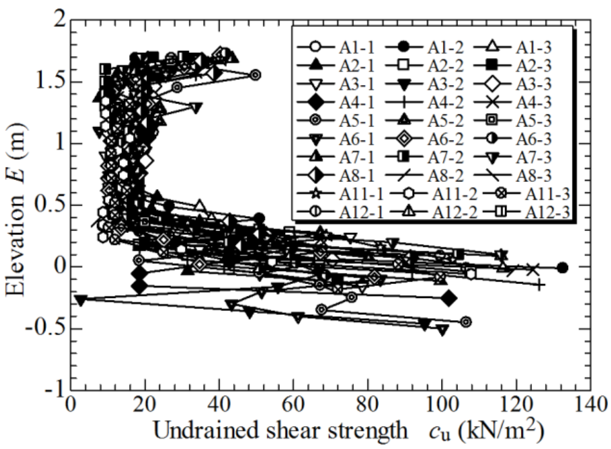


Fig. 2 Relationship between c_u from PCPTs and elevation for piles A1 to A12

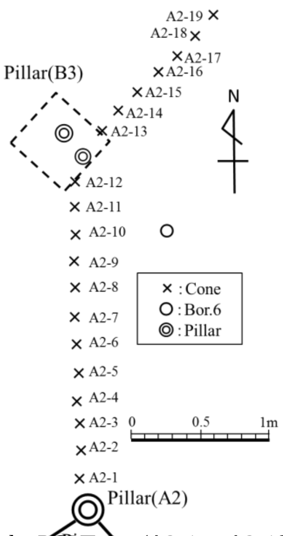


Fig. 3 Layout of the PCPT site (A2-1 to A2-19)

c_u are 17 kN/m^2 at E = 0.6 m and 19 kN/m^2 at E = 1.5 m, and the variation of c_u is 7 kN/m^2 unrelated to depth.

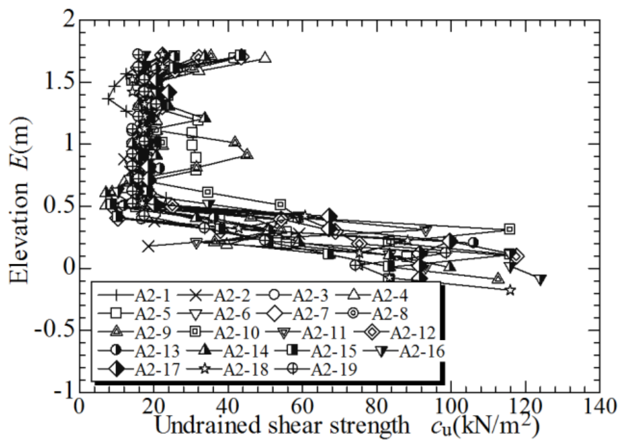


Fig. 4 Relationship between c_u and elevation based on PCPTs A2-1 to A2-19

3. Changes in the natural water content and strength of the soil around the pile

The bottom of the excavated cedar pile is cut off horizontally (**Photo 1**). A group of three pine piles in the B-line is shown in **Photo 2**. Each pile is about 2 m long and the bottom ends are cut at an angle of about 30° (**Photo 3**), indicating these were driven piles. **Fig. 5** shows the layout of the site where the CPTs were conducted at the trench cut around pile A11. The tests were performed along five horizontal levels at the excavated surface (**Photo 4**). The test results of the CPTs for lines (a) to (f) on the southern wall of the trench are shown in **Figs. 6 (a)** to **(f)**, respectively. The penetration depth (D_p) is 40 cm. The construction method of the piles of line A can be inferred from these c_u results, assuming that the load history from the construction of the foundation piles remains as the variable strength of the soil around the foundation piles.

Photo 1. Bottom end of cedar pile A1

Photo 2. Group of three pine piles (B8)

Photo 3. Bottom end the pine pile (B8)

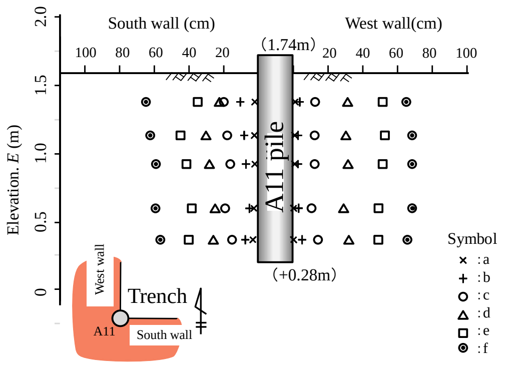
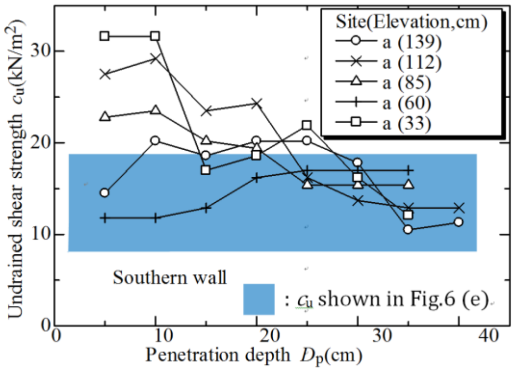


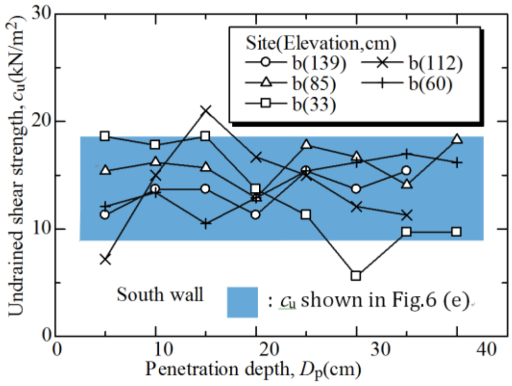
Fig. 5 Layout of the PCPT and w_n measurements around pile A11

Photo 4. PCPT site at the excavated vertical surface

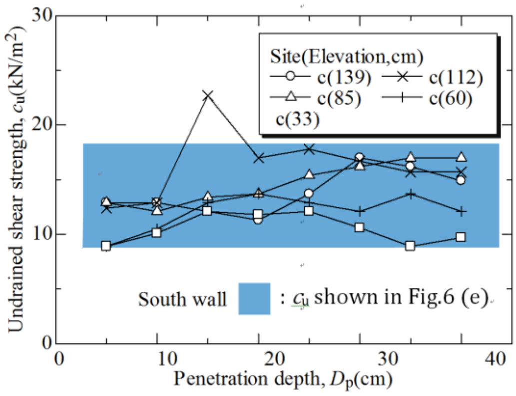
c_u decreases with increasing D_p along the CPT line(a), 2cm from the foundation pile (Fig. 6 (a)). This indicates that the c_u values are greatest close to the foundation pile. A similar trend was observed for the plots of E = 33, 85 and 112 cm, 5-8 cm from the foundation pile. However, along CPT line (e), 50 cm from the foundation pile (Fig. 6 (e)) c_u = 9-18 kN/m^2 and is almost constant at for the different elevations. The w_n values obtained from the southern and western walls are plotted against the distance from the surface of pile A11 (D_s) in **Fig. 7** w_n decreases by about 30% from D_s = 60 cm to D_s = 2 cm. This decreasing trend may reflect the draining effect of the cedar piles and/or the dissipation of the pore-water pressure caused by the movement of the soil particles driven by the penetrating pile. The c_u behavior in Fig. 6 corresponds to the change of w_n. This is supported by the visual assessment that the soils around the piles were dense. Therefore, the cedar foundation pile is considered to be a driven pile based on the changes in c_u and w_n measured in the area around it.



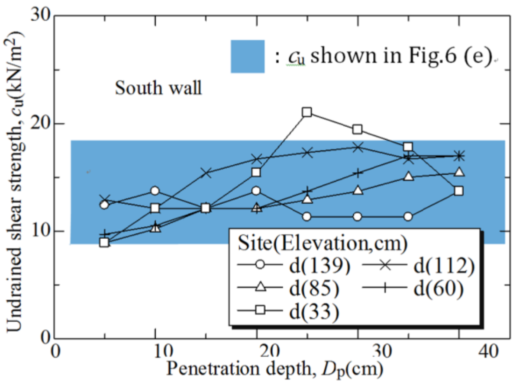
(a) At line (a) on the southern wall of the trench



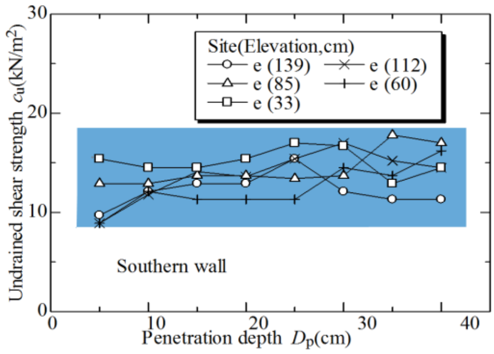
(b) At line (b) on the southern wall of the trench



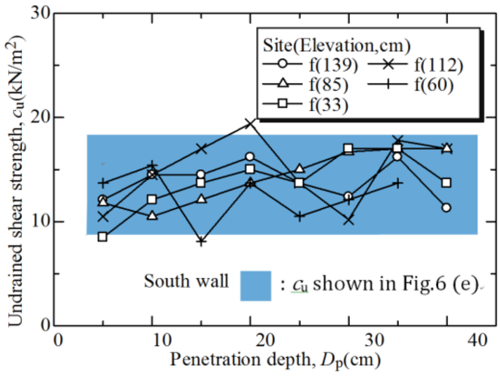
(c) At line (c) on the southern wall of the trench



(d) At line (d) on the southern wall of the trench



(e) At line (e) on the southern wall of the trench



(f) At line (f) on the southern wall of the trench

Fig. 6 Relationship between c_u and D_p around pile A11

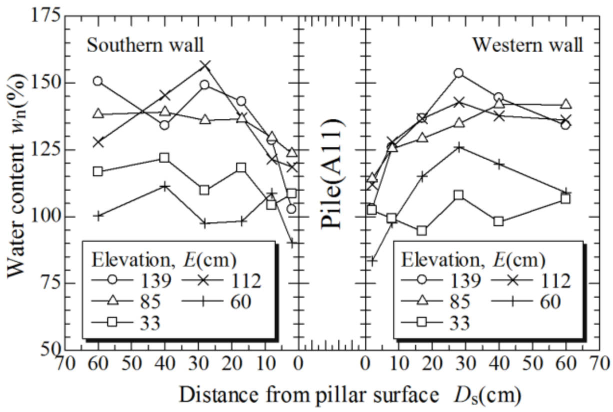
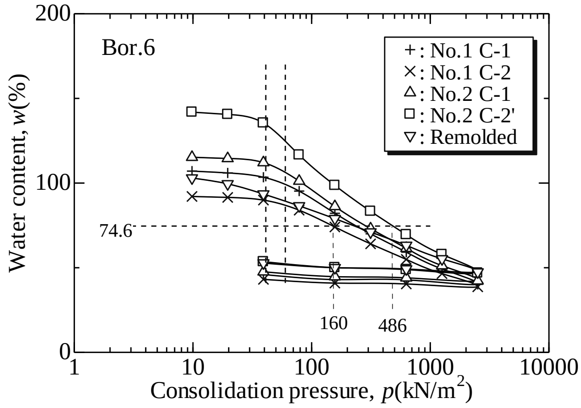


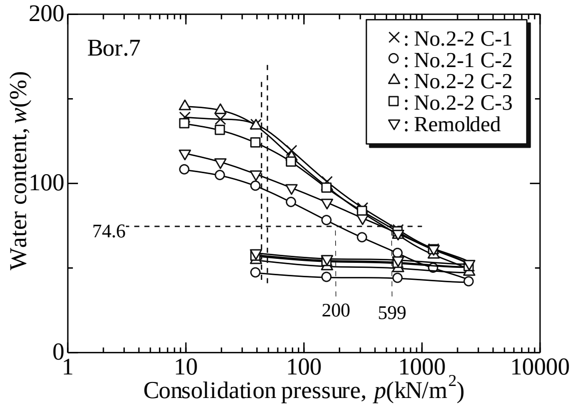
Fig. 7 Relationship between w_n and D_s around pile A11

4. Estimation of the driven load for the pile

The relationship between w_n and the consolidation pressure (σ'_v) on the incremental loading oedometer test (Japanese Standard Association, 1993) for the samples obtained from the 45-mm core are shown in **Figs. 8 (a)** and **(b)** for Bor. 6 and Bor. 7, respectively. The over-consolidation ratios (*OCR*s) of the samples obtained from Bors. 6 and 7 are 1.1 and 1.3, respectively. These are normal values for normally consolidated clay deposits, indicating that the clay layer was not affected by a load greater than that of the effective overburden pressure (σ'_{v0}) of 41 kN/m^2.



(a) Borehole 6



(b) Borehole 7

Fig. 8 Relationship between w_n and p

Fig. 9 shows a sketch of a basic structure that may have stood over the piles of lines A and B. The lower ends of the A-line piles are located 20 cm above the shell layer and the B-line pillars penetrate 30 cm into the shell layer. The w_n value of the clay under the A-line piles is 74.6%, which is less than w_n = 110%-140% for the clay not affected by the driven pile. The clay with w_n = 74.6% is hard and could not be penetrated by the CPT. The main reason for the decrease in w_n is most likely the driven load of the piles, the weight of the building supported by the piles and the drainage effect of the piles. These load histories equivalent to the w_n of the clay below the A-line piles can be estimated as 160-486 kN/m^2 (mean value of 323 kN/m^2) and 200-599 kN/m^2(mean value of 400 kN/m^2) from Figs. 8 (a) and (b) (Bors. 6 and 7), respectively. These values were used

$$R_a = 1/3(6c_uA_p+N_s/3\bullet L_s\bullet\psi + q_u/2\bullet L_c\bullet\psi)-W_p \quad (1)$$

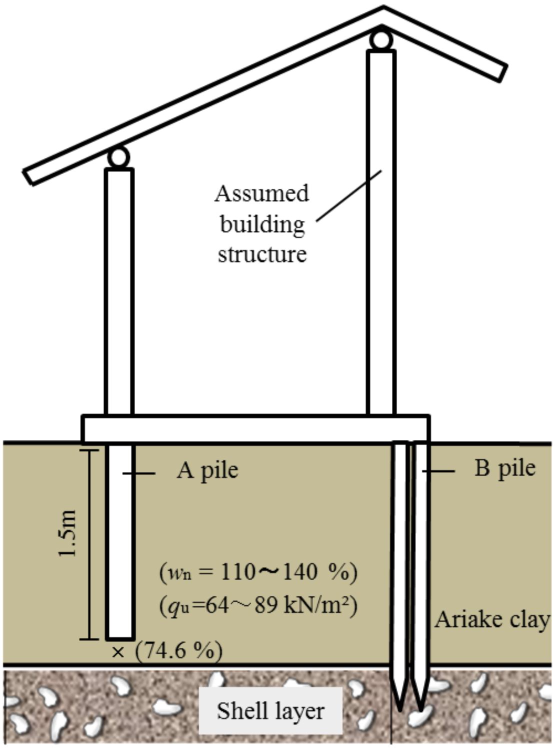


Fig. 9 Sketch of a basic building assumed to have been constructed on top of A- and B-line pile

to estimate the set-up of the driven pile and the load including the building in next section.

5. Ultimate bearing capacity, load history and driving energy of the piles

The load history estimated from **Fig. 8** is calculated as the sum of the weight of the building and the allowable bearing capacity (R_a) of the piles. *R*a, the long-term vertical allowable bearing capacity (R_a) of the pile (in kN), is estimated from **Eq. (1)** established by the Architecture Society of Japan (Japan Architectural Society, 1961), where Ap is the area of the pile tip (m2), N_s is the mean N-value obtained from a standard penetration test for the pile in sand layer, L_s is the length of the pile surface in contact with the sandy soil (m), L_c is the length of the pile surface in contact with the clayey layer (m), q_u is the unconfined compressive strength under the clayey layer for the pile (kN/m²), provided that $q_u \leq 157$ kN/m², c_u is the undrained shear strength of the clayey layer (kN/m²), ψ is the circumference of the pile (m), W_p is the weight of the pile defined as the effective weight minus the displaced effective soil weight (kN). The weight of the wooden building is estimated at 1.962 kN/m^2 based on the first floor of a wooden structure from a plan of the mid-nineteenth century.

Table 1 shows the values of q_u, c_u, W_p and the dimensions used to calculate R_a, and the R_a values I obtained from Bors. 6 and 7, respectively. $c_{u\,(cone)}$ is the c_u obtained from the PCPT and $q_{u(I)}/2$ is the estimated in situ c_u obtained using the Shogaki method (Shogaki, 2006). **Table 2** shows the estimated results for I, II and III, where III is the allowable stress for a pile, in kN/m² Equation (1) describes the long-term vertical allowable bearing capacity of a pile; thus, II represents the ultimate bearing capacity as 3Ra/Ap. The value III = 288.387kN/m^2 is the allowable vertical stress for a pile and the long-term vertical allowable bearing capacity of the pile weight of a one-story wooden house (1.962 kN/m^2 (in kN), is estimated from Eq. (1) established by the Architecture Society of Japan (Japan Architectural Society, 1961). The ratio of (II+III) / I is 2.23–3.13 for qu(I)/2 and 1.55-1.82 for qu/2. The ratio II / I is 1.07-1.23 for c_u (= q_u/2) and 1.75-2.54 for c_u (= $q_{u(I)}$/2). If these ratios are considered as the safety factor for similar current construction methods. The ultimate bearing capacity is estimated to be 1.15 to 2.15 times the

Table 1. Long-term vertical allowable bearing capacity of a pile

Borehole		6	7	6	7	6	7
Undrained shear strength, c_u		q_u/2		$q_{u(I)}$/2		$c_{u(cone)}$	
(Ⅰ): R_a(kN/m²)	Maximum value	470	560	470	560	470	560
(w_n=74.6%)	Mean value	323	400	323	400	323	400
(Ⅱ): $3R_a/A_p$(kN/m²)		597.000	642.000	1234.000	1047.000	606.000	515.000
(Ⅲ): Allowable stress of pile (1.962kN/m² for one-storied)		288.387(1.962 kN/m² for one-storied)					
(Ⅱ+Ⅲ)/ Ⅰ	Maximum value	3.310	2.846	6.971	5.634	3.418	2.494
	Mean value	4.816	4.935	10.143	9.768	4.973	4.324
Ⅱ/ Ⅰ (Maximum value)		1.230	1.070	2.540	1.750	1.250	0.860

Table 2. Estimated value for load history of a pile

Borehole		6	7	6	7	6	7
Undrained shear strength, c_u		$q_u/2$		$q_{u(I)}/2$		$c_{u(cone)}$	
R_a (kN)		48.810	50.011	102.861	99.056	50.406	43.818
c_u (kN/m²)	z=0.2～0.8m	20.213	22.473	39.641	29.584	20.225	16.966
	z=0.9～1.5m	16.015	16.174	31.573	31.363	16.531	14.726
A_p (m²)		0.031	0.031	0.031	0.031	0.031	0.031
ψ (m)		0.628	0.628	0.628	0.628	0.628	0.628
L_c (m)		1.5					
W_p (kN)		0.638					
Pile diameter (m)		0.2					
(Ⅰ): R_a from Bor.6 (kN/m²)		486 (Maximum value)			543 (Mean value)		
(Ⅰ): R_a from Bor.7 (kN/m²)		599 (Maximum value)					
(Ⅱ) : R_a/A_p (kN/m²)		1554.470	1592.716	3275.834	3154.638	1605.274	1395.464

Table 3. Driving energy of a pile

H (m)	Hammer weight W_H (kN)		Dropped height H (m)		
			1	2	3
	kgf	kN	Driving energy (kN・m)		
1	101	0.995	0.995	0.990	2.985
2	51	0.497	0.497	0.994	1.491
3	34	0.332	0.332	0.664	0.996

consolidation stress of the clay under the pile.

The ultimate bearing capacity (R_d) of a pile based on the Basic Architectural Structure Design Standards and commentary (Japan Architectural Society, 1961) is estimated as

$$R_d = (0.6W_H \bullet H) / (S + 0.02) \quad (2)$$

in (t), where W_H is the hammer weight (t), H is the dropped height of the hammer (m), and S is the final penetration length of the pile (m).

Table 3 shows the relationship between H and W_H estimated for S = 0.015 m. For H = 2 m, W_H for I is calculated with 51 kgf. The driving energy of I at H = 1-3 is also shown in Table 3. The driving energy of W_H = 51 kgf at H = 2 m is 0.994 kN・m. The total driving energy to drive the pile to a depth of 1.5 m is 99.4 kN・m, assuming S = 15 mm in all the cases; therefore, 100 impacts would be required to drive the pile into the clay ground to a depth of 1.5 m. It is possible to underestimate S since q_u in the soft clay layer is 40 kN/m^2. If we assume for pile driving, they can be used as design criteria S = 0.03 m, the total energy and number of impacts would be 49.7 kN•m and 50, respectively. The total energy required to drive the pile to a depth of 30 mm with a 550-N hammer dropped from a 2-m height is calculated as 49.7 kN•m; thus, 50 hammer blows would be required to drive the pile to a depth of 1.5 m.

6. Conclusions

The conclusions obtained in this study can be summarized as follows.

1) The natural water content w_n decreased by about 30% at a distance of 2 cm from the pillar surface. This is likely caused by the draining effect of the cedar pile and/or the dissipation of the pore-water pressure caused by movement of soil particles as a result of pile penetration.
2) The deformation around the pile caused by pile penetration increased within the range equal to the pile diameter. This is likely due to the decreased water content and increased shear strength caused by increased pore-water pressure and dissipation.
3) The area influenced by the pile penetration as measured on site was similar to the affected area in the model (adjusted to the model scale of 20:3.5) for clays with q_u = 57 and 77 kN/m^2. Thus, based on the changes in c_u and w_n measured around pile A11 and the model test results, we conclude that the cedar foundation pile was driven into position.

4) The ultimate bearing capacity was estimated to be 1.15 to 2.15 times the consolidation stress of the layer of soil under the pile. The total energy required to drive the pile to a penetration depth of 30 mm with a hammer of 500 N in weight dropped from a 2-m height was calculated as 49.7 kN•m. Thus, 50 hammer blows would be required to drive the pile to a depth of 1.5 m.

References

Japanese Architectural Society. 1961. Basic architectural structure design standards and commentary. (in Japanese)

Japanese Geotechnical Society. 2015a. Method for obtaining soil samples using thin-walled tube sampler with fixed piston (JGS 1221-1995), Standard of Japanese Geotechnical Society for Soil Sampling -Standards and Explanations-, 1-7 (English version)

Japanese Geotechnical Society. 2015b. Method for portable cone penetration test (JGS 1431-2003), Japanese standards for geotechnical and geoenvironmental investigation methods –Standards and explanations-, 290-295. (in Japanese)

Japanese Standard Association. 1993. Test method for one-dimentional consolidation properties of soils, JIS A 1217-1990, 1-13. (in Japanese)

Shogaki, T, 2006. An improved method for estimating *in-situ* undrained shear strength of natural deposits, Soils and Foundations, Vol. 46, No. 2, pp. 1-13.

Shogaki, T. 2017. Mechanism of sample disturbance caused by tube penetration (Model test on Toyoura sand), Soils and Foundations, 57 (3), 527-542.

Shogaki, T. and Sakamoto. R. 2004. The applicability of a small diameter sampler with a two-chambered hydraulic piston for Japanese clay deposits, Soils and Foundations, 44 (1), 115-126.

Shogaki, T., Okuda, D., Nakano, Y., and Suzuki, M. 2018. Strength and deformation properties of Ariake clay deposit in Mietsu Naval Facilities, The 53rd Annual Conference on Japanese Geotechnical Conference, 335-336. (in Japanese)