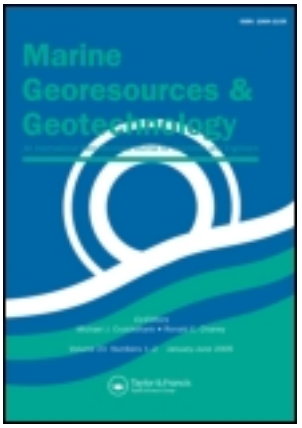

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Field Evaluation on the Strength Increase of Marine Clay under Staged Construction of Embankment

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This article presents a case history of an embankment constructed on the soft Ariake clay on the bank of Rokkaku River, Saga, Japan. The staged method was used in the construction of embankment. To shorten the time of construction, the soft subsoil was treated by a prefabricated vertical drain (PVD). The physical and mechanical properties of soft subsoil, such as water content, unit weight, and unconfined compressive strength, were investigated before and during embankment construction. Based on the field data, the strength increase of Ariake clay during construction was revealed. A nonlinear relationship between strength increment and the degree of consolidation was found. Finite element method (FEM) was employed to investigate the strength increase behavior. The results showed that it is difficult to predict the filed behavior of strength increase using Cam-clay model. Based on the concept of structure loss of structured clay, the strength variation of Ariake clay during consolidation is discussed.

Keywords FEM, field measurement, soft structured clay, staged construction, strength increasing pattern

Introduction

There are widely distributed soft deposits along the coastal regions all over the world. In coastal regions, a large amount of infrastructure, such as road and river embankments, should be constructed over the soft deposit. Since the soft subsoil has very low shear strength, it cannot support rapidly constructed high embankments. Generally, in order to increase the strength of soft subsoil with consolidation under embankment loading, high embankment must be constructed in several stages (Ladd 1991; Leroueil et al. 1980). However, it needs long period of waiting in staged construction for high embankment on natural subsoil. In some cases, for an embankment with a height of

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6 m constructed on natural Ariake clay, it requires 3 years for completion of construction (Takeo office 1996; Chai et al. 2001); in the coastal region of China, it also requires 2 to 3 years for the embankment with a height of 8 m on natural silty clayey ground (Shen et al. 2005). In some cases, a long period of construction is not cost-effective. In order to speed up the consolidation process, prefabricated vertical drain (PVD) or plastic board drain (PBD) was generally used (Bo 2004; Bergado and Teerawattanasuk 2008; Chai et al. 2005; Farnsworth et al. 2008; Lorenzo et al. 2004; Ozcoban et al. 2007; Rowe et al. 2008; Shen et al. 2005; Tanchaisawat et al. 2008). With the application of more than three decades, PVD has been proven to be an efficient way to accelerate the rate of consolidation and improve the performance of the soft subsoil. Combining with the PVD, stage loading has been used more widespread in embankment construction (Ladd 1991).

Thus, in the design of staged construction of embankment over PVD-improved soft clayey subsoil, the following two aspects should be confirmed: i) evaluation of the degree of consolidation, and ii) variation of strengths of subsoil during and after consolidation. For the first aspect, FEM is very efficient. There are some approaches used to model the PVD in soft subsoil (Chai et al. 1995; Chai et al. 2001; Hird et al. 1995, 1992; Indraratna and Redana 1997; Sekiguchi et al. 1986). One of the most effective ways is using the equivalent value of vertical hydraulic conductivity (k_{ve}) of PVD-improved subsoil (Chai et al. 2001; Shen et al. 2005). In this way, the influence of PVD can be simply modeled as an increasing of vertical hydraulic conductivity. In this study, this approach is employed to analyze the behavior of staged construction of embankment on PVD improved soft subsoil.

For the second aspect, there are many evaluation methods on the strength based on the ratio between undrained shear strength Cu and consolidation stress p' (hereafter simply called C_u/p' ratio) after complete consolidation (Ladd 1991; Leroueil 1980). After primary consolidation, the relationship between undrained shear strength and effective vertical stress is a function of plastic limit, which can be drawn as $C_u/p' = f(I_p)$ (Bjerrum 1972). However, very limited researches have been conducted during the consolidation of soft clays. Takayama (1984) and Umesaki and Ochiai (1993) studied the strength obtaining mechanism for Ariake clay via laboratory tests, even though there is much less field data on the strength increase in field. Almeida et al. (2000) and Sinha et al. (2007) assumed the strength was linearly increased with the degree of consolidation, which follows the implication of modified Cam-clay model (Almeida et al. 2000).

However, all of these studies were conducted on remolded clay. For structure clay, the strength is related to yield stress and water content after yield (Hong et al. 2007), *e.g.*, $C_u/p' = f(\sigma'_y, w)$. According to the limited laboratory tests, the strength increase behavior of structured clay under staged embankment loading is related to the degree of disturbance, effective consolidation stress, and water content (Hong et al. 2006, 2007). The objective of this article is to reveal the mechanism of strength obtaining process of structured Ariake clay improved by PVD under a staged construction of an embankment.

Project Background

The construction site of the case history is on Saga Plain, Japan, as shown in Figure 1. The embankment was constructed for a sluice gate at the southern side of Rokkaku River on Kyushu Island. The subsoil is soft Ariake clay, which is a typical problematic

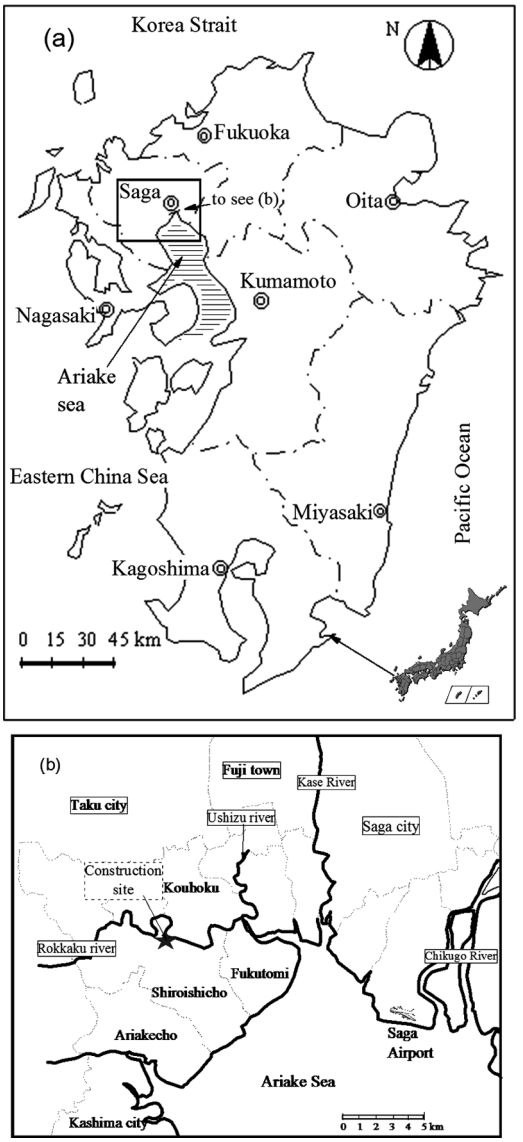

Figure 1. Location of the test site at Ashikari, Saga, Japan.

soil in Japan (Miura et al. 1998; Hong et al. 2004). The Ariake clay has a very high sensitivity and low shear strength. This soft subsoil is deposited under marine environment (Miura et al. 1998; Hong et al. 2007). Figure 2 plots the plan view of the test embankment and layout of instrumentation. Figure 3 depicts the sectional view of embankment at the tested section. Table 1 tabulates the parameters of embankment.

Geotechnical Profile and Soil Properties

The soil profile and properties of soft subsoil at this location are depicted in Figure 4. As shown in Figure 4, there is a thin weathered crust from the surface to about

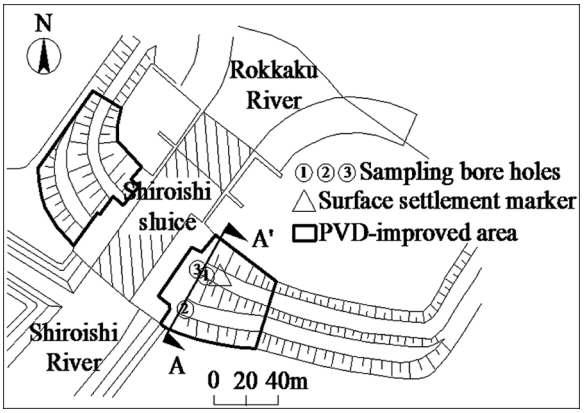

Figure 2. Plan view of embankment and layout of instrumentation.

1–1.5 m depth, underlain by a soft high sensitive Ariake clay layer with variable properties to about 20 m depth. Below the Ariake clay, there are layers of dense clayey sand, which have much higher permeability and strength than those of the lower layer. At this site, the grain size distribution of the Ariake clay includes 56–75% clay particles, 20–36% silt particles, and 2–8% sand particles. For the Ariake clay layer, the physical properties, as depicted in Figure 4, have an obvious change at 10 m deep (the depth of 10 m). Therefore, it can be separated into two layers, called the upper Ariake clay and the lower Ariake clay (Hanzawa et al. 1990).

Ground Improvement and Embankment Construction

As shown in Figure 3, the total height of embankment is 6.5 m. In order to improve the properties of subsoil and shortening the construction period, ground treatment methods were conducted before the construction of embankment. Before placing the sand mat, PVD had been installed to act as vertical drains. PVD was installed

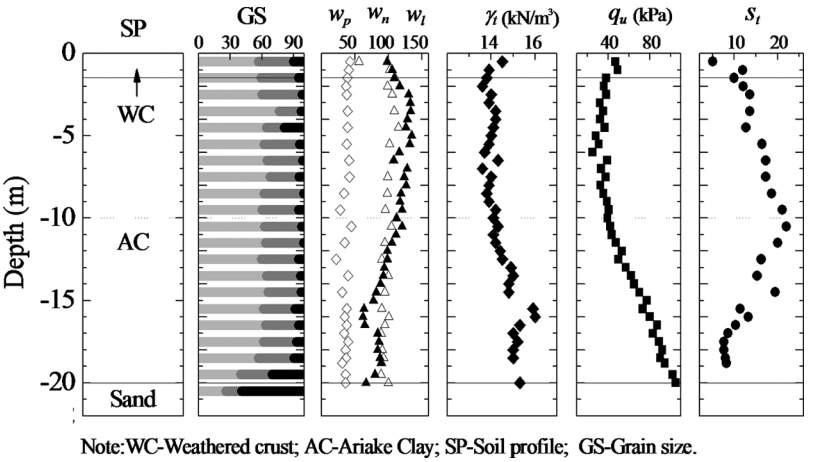

Figure 3. Geotechnical profile and soil properties.

Table 1. Embankment descriptions

H (m)	n	D_T (m)	D_{clay} (m)	Remark
6m	1:2	0.5	20	PVD, $t = 6$ mm, $D = 20$ m, Natural subsoil

Note: H = embankment height; n = side slope of embankment; D_T = thickness of crust layer; D_{clay} = thickness of clay layer; PVD = plastic drain improved; t = thickness of PVD; D = depth of drain.

in a square pattern with spacing 1 m, and to a depth of 20 m. The loading-time history for the embankment, as plotted in Figure 5. The embankment was constructed in four consolidation stages, labeled as I, II, III, and IV, respectively. At stage I, the

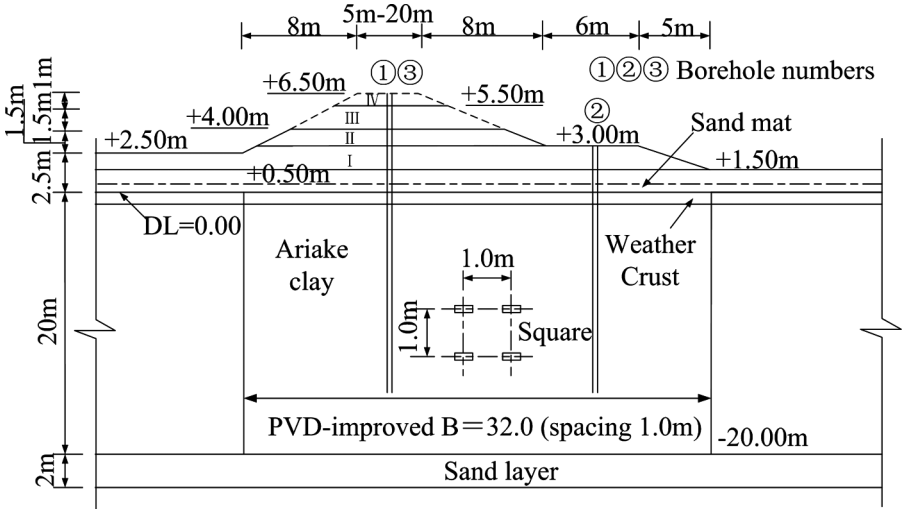

Figure 4. Cross sectional view of embankment.

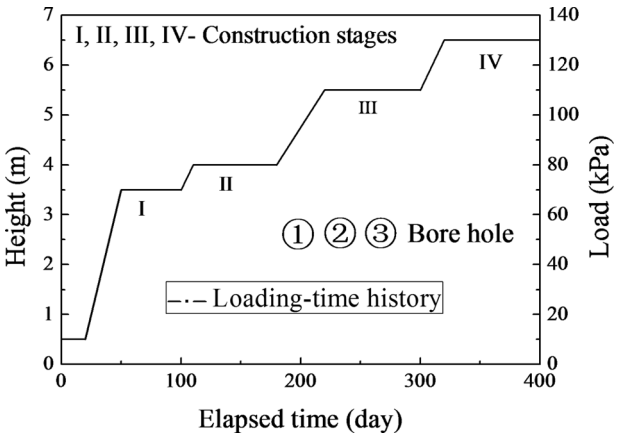

Figure 5. Embankment construction procedure.

height of embankment reached 3.3 m. In this stage, filling was compacted in a thickness of 200 to 300 mm. After stage I, the subsoil was consolidated for 75 days. Next stages were similar to the first one, and the height of filling were 0.7 m, 1.5 m, and 1.0 m, respectively. After each stage, subsoil was also consolidated for a period. From the beginning until the end of the construction, the properties of the subsoil were tested in stage I, II and III by borehole. The location of borehole is shown in Figures 2, 3, and 5, labeled as ①, ②, ③, respectively.

Analytical Methods

In order to analyze the behavior of PVD-improved soft subsoil under staged embankment loading, FEM was employed. The following aspects in the modeling are presented:

Modeling of PVD

In the FEM analysis, the effect of PVD-improvement was modeled as an equivalent value of vertical hydraulic conductivity (k_{ve}) as proposed by Chai et al. (2001). The equivalent vertical hydraulic conductivity of PVD improved subsoil, k_{ve}, can be defined from the following expression (Chai et al. 2001):

$$k_{ve} = \left(1 + \frac{2.5l^2}{\mu D^2}\frac{k_h}{k_v}\right)k_v \tag{1}$$

Where l = drainage length; D = diameter of unit cell; and k_h, k_v = horizontal and vertical hydraulic conductivity of subsoil, respectively. Parameter μ can be expressed as following:

$$\mu = \ln\frac{n}{s} + \frac{k_h}{k_s}\ln(s) - \frac{3}{4} + \pi\frac{2l^2k_h}{3q_w} \tag{2}$$

Where, $n = D/d_w$ (d_w = diameter of vertical drain); $s = d_s/d_w$ (d_s = diameter of smear zone); k_s = horizontal hydraulic conductivity in the smear zone; and q_w = discharge capacity of PVD in field. All of the parameters of PVD used in this study are listed in Table 2.

Table 2. Parameters of PVD

Item	Symbol	Value
Drain diameter (mm)	d_w	48.3
Drainage length (m)	l	20
Diameter of influential zone (m)	D_e	1.13
Smear zone diameter (mm)	d_s	300
Discharge capacity (m^3/a)	q_w	100

Mesh and Boundary Conditions

In FEM analysis, the plain strain condition is adopted. The model range in vertical direction is 28.5 m and in horizontal direction is 250 m. Because of the difference between the left and right side of embankment, the model is not built axisymmetrically and the embankment locates in the center. The displacement and drainage boundary conditions are set as: at bottom, both vertical and horizontal displacements were restrained, and bottom line (sand layer) was set as drained. Left and right vertical boundaries were set as undrained and the horizontal displacement of them was fixed. The ground surface was set as drained. The finite element analysis mesh is presented in Figure 6.

Model and Parameters

The mechanical behavior of soft clay was modeled by the Modified Cam-Clay model (MCC) (Britto and Gunn 1987; Roscoe and Burland 1968) and the sand layer was modeled as elastic material. The parameters of soft clay used in this study are listed in Table 3. In Table 3, since there is variation of soil properties with depth, clay layer was divided into 5 layers with the different parameters. All of these parameters are determined from laboratory test on undisturbed samples, except for Poisson's ratio and hydraulic conductivity (k_{ve}). The over-consolidation ratio (OCR) for the top crust layer is set as 2. The lateral earth pressure coefficient is based on the equation proposed by Mayne and Kulhawy (1982).

$$K_0 = (1 - \sin\phi')(OCR)^{\sin\phi'} \tag{3}$$

Where ϕ' = effective internal friction angle of soil, OCR = over-consolidation ratio.

Calculation of Strength and Degree of Consolidation

The undrained shear strength of the soft soil can be derived from constitutive model (Wood 1990; Roscoe et al. 1968). The MCC model is used in this study for soft clay.

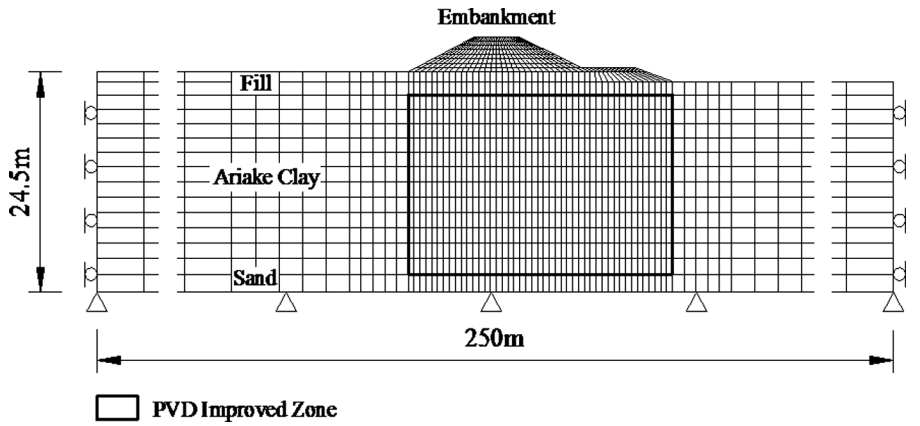

Figure 6. Finite element analysis mesh.

Table 3. Parameter of soft clay

Layer	*Depth* (m)	ν	κ	λ	M	e_0	Γ	γ_t (kN/m^3)	k_h (10^{-4} m/d)	k_v (10^{-4} m/d)	k_{ve} (10^{-4} m/d)	k_{ve}/k_v
C	$0 \sim -2$	0.15	0.036	0.25	1.2	3.00	4.17	15	9.893	6.530	177	27.1
Ac1	$-2 \sim -8$	0.15	0.137	0.96	1.2	3.35	5.08	13.5	5.089	3.171	115	36.3
Ac2	$-8 \sim -12$	0.15	0.096	0.67	1.2	3.00	4.60	14	2.281	1.348	64	47.5
Ac3	$-12 \sim -14$	0.15	0.080	0.56	1.2	2.41	4.03	15	2.280	1.348	64	47.5
Ac4	$-14 \sim -16$	0.15	0.063	0.44	1.2	1.66	3.30	15	2.864	1.620	84	51.9
Ac5	$-16 \sim -20$	0.15	0.073	0.51	1.2	2.28	3.91	18	3.175	1.737	100	57.6

Note: C = crust layer; AC = Ariake clay; γ_t = unit weight; e_0 = initial void ratio; λ = virgin loading slope in e-ln(p') plot (p' is effective mean stress); κ = reloading/unloading slope in e-ln(p') plot; M = slope of failure line in p' versus q plot (q is deviator stress); ν = Poisson's ratio; ϕ' = effective internal friction angle.

This model yields undrained shear strength of soil as follows: (Wood 1990)

$$C_u = \frac{1}{2^{\Lambda+1}} M p_0' (OCR)^{\Lambda} \tag{4}$$

$$p_0' = \left(\frac{M^2 + \eta^2}{M^2}\right)^{\Lambda} p' \tag{5}$$

Where p' = effective mean stress, η = stress ratio q/p', p_0' = corresponding effective mean stress on isotropic consolidation line, M = slope of the failure line in (q, p') plot, and Λ equals ($1-\kappa/\lambda$), κ and λ = the slopes at void ratio versus logarithm mean effective stress plot during reloading, respectively.

The degree of consolidation (U_p) used here is governed by (Takayama 1984),

$$U_p = \frac{\left[\left(\frac{p_f}{p_0}\right)^{U_\varepsilon} - 1\right]}{\left(\frac{p_f}{p_0} - 1\right)} \tag{6}$$

Where $U_\varepsilon = (e_0 - e)/(e_0 - e_f)$, e = void ratio, e_0 = initial void ratio, e_f = final void ratio, p_f = effective stress, p_0 = initial effective stress.

Results and Analysis

Settlement

Figure 7 plots measured and calculated settlement-time curve of the embankment. The FEM result of the settlement of embankment on PVD-improved subsoil was depicted in solid black line and the measured settlement was plotted in circle. The settlement data of ① and ③ were measured at the centerline of the embankment, and settlement data of ② were measured at the south-west side of embankment. As seen in Figure 7, the settlement of subsoil is very small before the consolidation

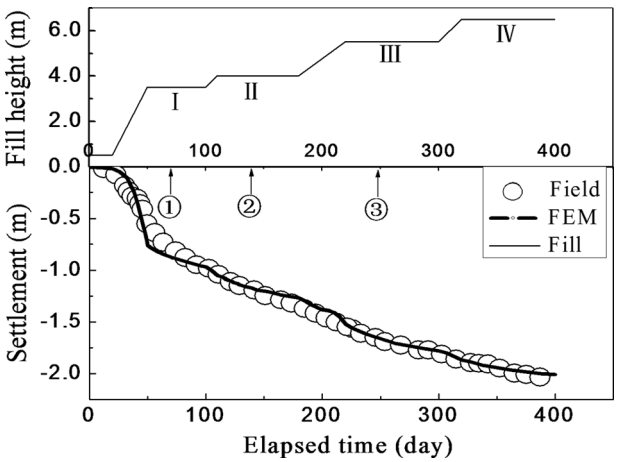

Figure 7. Settlement-time curve of the embankment.

stage I. When the construction of sand mat finished, the settlement only reached 5 mm. This is because of the OCR of crust layer is set as 2 and applied load in this step is less than its yield stress of the top crust. The settlement of subsoil increased with the increase of the height of embankment. At the end of construction, the maximum settlement of embankment reached 2.2 m. As illustrated in Figure 7, FEM can predict field value fairly well.

Physical Properties

Figure 8a shows the distribution of water content with depth in the different consolidation stages. Figure 8b illustrates the variation of water content with the degree of consolidation. As shown in Figure 8, water content decreased with the construction stage of embankment; in stage I, maximum decreased value of water content was about 10%, and the depth of water content decrease is about 8 m; in stage III, the maximum decreased value of water content is about 15% to a depth of about 16 m. This phenomenon is in conflict with the general knowledge that the upper layer will consolidate first. The reason may be due to the effect of the structure of top crust (Hong and Han 2007). In stage I, the applied load of embankment (70 kPa) is less than the yield stress (71 kPa) so that there is less compression in the top crust than in the lower soft Ariake clay layer, of which the smallest yield stress was only about 28 kPa at the depth of 8 m so that more compression occurred in the lower layers.

Figure 9a plots the variation of unit weight and Figure 9b plots the variation of unit weight with the degree of consolidation. As shown in Figure 9, the unit weight increased with the construction stage. The maximum value of the increase is about 1 kN/m^3. The variation of unit weight with depth is similar to the water content. At the stage III, the variation depth expanded to 17 m.

Mechanical Properties

Figure 10 plots the variation of unconfined compressive strength (UCS) during embankment construction. As shown in Figure 10, the UCS value increased with

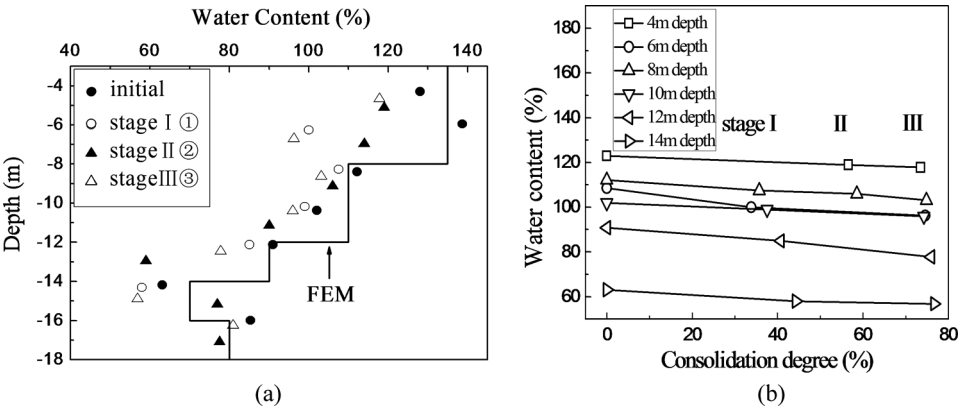

Figure 8. (a) Variation of water content; (b). Variation of water content with U_p.

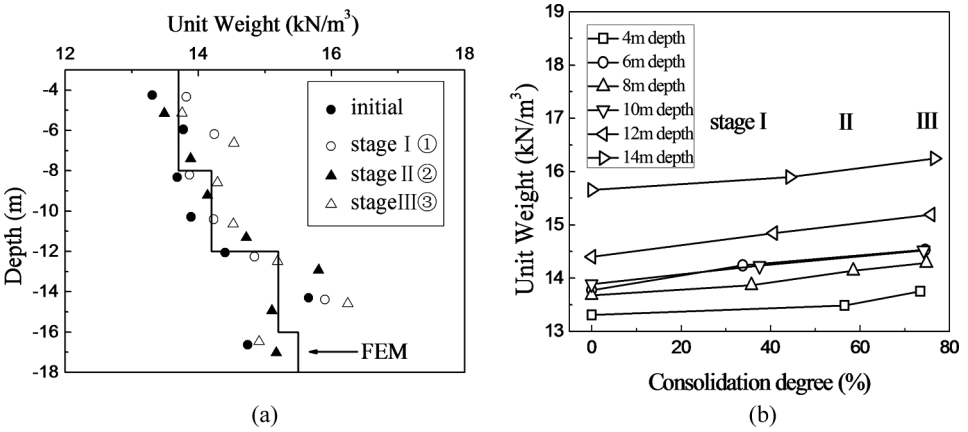

Figure 9. (a) Variation of unit weight; (b) Variation of unit weight with U_p.

the construction stages. The increment of the strength of subsoil distributes with depth at different stages. This result means that the gaining of strength is related with the effective stress and degree of consolidation. Figure 11 plots the relationship between incremental shear strength ratios (ISSR), $\Delta C_u/\Delta p'$, and the degree of consolidation (U_p) as described Eqs. (6) of stage I and III. In Figure 11, the incremental shear strength was expressed by following equation:

$$\Delta C_u = \frac{\Delta q_u}{2} = \frac{q_{ut} - q_{u0}}{2} \tag{7}$$

where ΔC_u = incremental of undrained shear strength, Δq_u = incremental UCS, q_{ut} = UCS at time t, q_{u0} = UCS before construction.

The black line in Figure 11 presents the MCC predicted value on the relation between $\Delta C_u/\Delta p'$ and U_p, which means that the increasing in the shear strength is in linear proportion to the increasing in the effective stress during consolidation

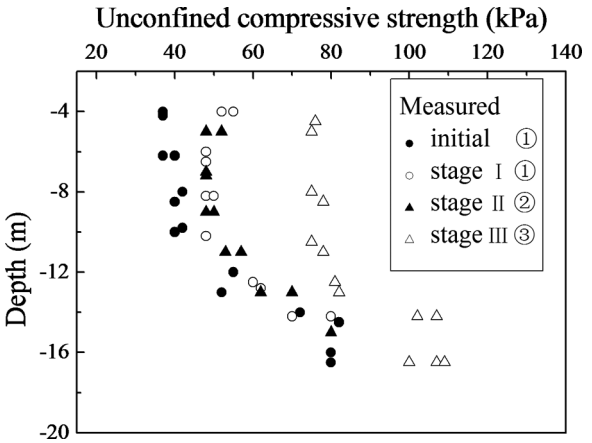

Figure 10. Variation of strength during embankment construction.

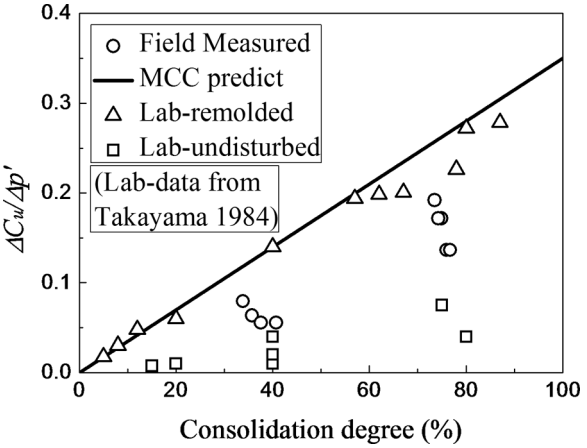

Figure 11. Relation between $\Delta C_u/\Delta p'$ and U_p in field.

(Almeida et al. 2000; Sinha et al. 2007). The circles in Figure 11 are field measured data. The triangles are laboratory test data of remolded Ariake clay and the squares are laboratory test data of undisturbed Ariake clay as presented by Takayama (1984). As shown in Figure11, the data of remolded Ariake clay are on the MCC predicted value; however, the field data and the data of undisturbed Ariake clay locate under the MCC predicted value. This means that MCC overestimates the ability of the strength–gaining nature of Ariake clay during consolidation. Figure 11 also illustrates that the rate of strength gaining in the field is small at the beginning. The rate of strength obtaining increases with the progress of consolidation.

Discussion

As shown in Figure 11, the ISSR from field measured data and laboratory data on undisturbed soft Ariake clay is much lower than that from the MCC predicted value (the bold solid line in Figure 11). These results indicate that although MCC model can predict the settlement behavior of the structured clay (Figure 7), MCC model may overestimate the strength gaining for the structured clay during the consolidation process, which means that if we design the construction schedule based on the MCC, the soft subsoil becomes dangerous. The possible reason can be elucidated as follows.

Figure 12 shows the relationship between undrained shear strength and consolidation pressure for Ariake clay, as presented by Hong et al. (2007). As shown in the figure, the behavior of strength increase with consolidation pressure of remolded clay sample is much different from that of undisturbed soil sample of structured clay. The strength of remolded clay sample linearly increases with the consolidation pressure. For the structured clay sample, if the consolidation pressure is less than the yield stress, the strength almost keeps constant. However, if the consolidation pressure is larger than the yield stress, the strength behaves the same tendency as that of remolded sample. Hong et al. (2006) and Hong and Han (2007) also pointed out that if the soil is disturbed, the strength obtaining behavior still follows the behavior, as shown in Figure 12, *i.e.*, there is a bilinear relationship between strength and

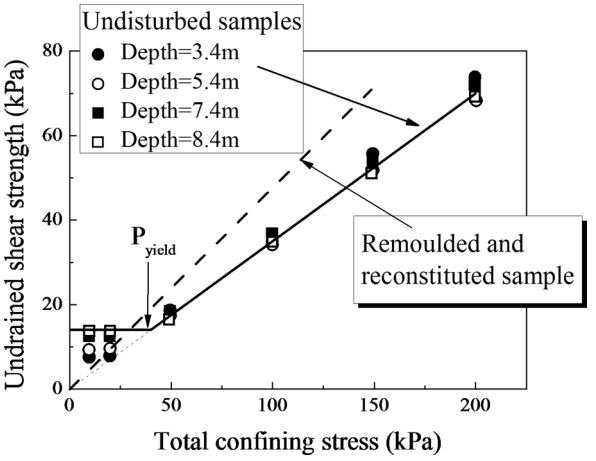

Figure 12. Relation between undrained shear strength and consolidation pressure (based on the concept proposed by Hong et al. 2007).

consolidation pressure. However, strength increase rate with consolidation pressure before yielding (SRBY) is much smaller than that after yielding and the SRBY related to the degree of disturbance.

This strength behavior under consolidation pressure implies that for structured clay, even under disturbed state, the consolidation pressure must overcome the structure at first, thus the strength increase is smaller than that from MCC prediction. The related state factors on the strength increase behavior of structured clay under staged embankment loading may include *i*) the degree of disturbance, *ii*) yield stress, *iii*) consolidation pressure, *iv*) water content, and *v*) degree of consolidation.

In the stage construction of embankment, the waiting period between two stages is a significant aspect of concern for engineers. Generally, this waiting period is determined based on the 90% of degree of consolidation combined with the experience of engineer. There are also some empirical methods used to estimate the waiting period, *e.g.*, Asaoka (1978) and Sinha (2008). However, these methods are still based on the increase of shear strength of clay during consolidation being linearly in proportion to the increase in the effective stress, such as the inflection point method (Sinha 2008). The results from this field case indicate that it is not safe to employ the MCC concept to predict the strength increase of nature structured clay. Thus, the prediction approach to predict the strength acquirement of structural clay during consolidation should be investigated in the future.

Conclusions

This study presented a case history of an embankment constructed on soft Ariake clay, which is structured clay. The relationship between strength increase and the degree of consolidation has been investigated. Based on the field data and FEM analysis, the following conclusions can be drawn:

1. The physical properties of soft Ariake clay have been measured at different construction stages. With embankment construction, water content decrease and unit

weight increases. The yield stress in top crust is much higher than that in the lower part. So the variations of water content and unit weight in the lower part (under 8 m) are larger than those in the upper layer.

2. Field investigation results show that the relationship between strength increase of soft Ariake clay and the degree of consolidation is nonlinear. At the beginning stage of consolidation, the ratio of strength increase is very low; however, after completion of consolidation the ratio becomes large. This is different with the conventional consideration based on MCC concept, in which the ratio of strength increase is considered to increase linearly with the degree of consolidation.
3. Although the modified Cam-clay can predict the settlement during consolidation fairly well, MCC can not predict the variation of shear strength during consolidation in an acceptable range. The reason is due to the soil structure of soft Ariake clay, which can not be considered in the MCC model.
4. The strength increase behavior of structured clay under staged embankment loading may be related to the degree of disturbance, yield stress, consolidation pressure, water content, and degree of consolidation. Since the conventional way to predict the behavior of strength increase based on MCC concept does not include the aspects mentioned before, it is not applicable to the soft structured clay. Thus, the relationship between strength increase and degree of consolidation of soft structured clay still needs to be investigated.

Nomenclature

C_u	undrained shear strength ($M/L/T^2$)
ΔC_u	incremental of undrained shear strength ($M/L/T^2$)
d_s	diameter of smear zone (L)
d_w	diameter of vertical drain (L)
D	diameter of unit cell (L)
D_e	diameter of influential zone (L)
D_T	thickness of crust layer (L)
D_{clay}	thickness of clay layer (L)
e	void ratio (dimensionless)
e_0	initial void ratio (dimensionless)
e_f	final void ratio (dimensionless)
H	embankment height (L)
I_p	plastic limit (dimensionless)
k_0	at-rest earth pressure coefficient (dimensionless)
k_h	hydraulic conductivities in horizontal directions (L/T)
k_v	hydraulic conductivities in vertical directions (L/T)
k_s	hydraulic conductivity of smear zone (L/T)
k_{ve}	equivalent vertical hydraulic conductivity of PVD-improved subsoil (L/T)
l	drainage length of PVD (L)
M	slope of the failure line in p'-q plot (dimensionless)
n	side slope of embankment (dimensionless)
OCR	over-consolidation ratio (dimensionless)
p'	effective mean stress ($M/L/T^2$)
p_0	initial effective stress ($M/L/T^2$)
p_f	final effective stress ($M/L/T^2$)
q_{u0}	unconfined compressive strength before construction ($M/L/T^2$)

q_{ut} unconfined compressive strength at time t ($M/L/T^2$)
q_w discharge capacity of PVD in field (L^3/T)
Δq_u incremental of unconfined compressive strength ($M/L/T^2$)
t thickness of PVD (L)
U_ε consolidation degree by void ratio (dimensionless)
U_p consolidation degree by mean stress (dimensionless)
ν Poisson's ratio (dimensionless)
w water content (dimensionless)
σ'_y vertical effective stress ($M/L/T^2$)
ϕ' effective internal friction angle of soil (dimensionless)
γ_t unit weight of soil ($M/T^2/L^2$)
λ virgin loading slope in e-lnp' plot (dimensionless)
κ reloading/unloading slope in e-lnp' plot (dimensionless)

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