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PERFORMANCE OF BAND SHAPED VERTICAL DRAIN FOR SOFT HAI PHONG CLAY

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ABSTRACT

Performance of band shaped prefabricated vertical drain (PVD) installed into soft Hai Phong clay with a 110 cm triangle arrangement is reported together with the engineering properties of the clay investigated by field and laboratory tests. Stationary piston sampling was carried out to obtain high quality undisturbed soil samples for laboratory tests and reliable engineering characteristics of the clay. It was assumed for the design of PVD spacing and preloading that the ratio of apparent value of horizontal coefficient of consolidation $c_{h(ap)}$ to vertical coefficient of consolidation c_v is equal to 1.0. The settlement monitored in the field, which clearly showed that the actual settlement was faster than expected, resulted in the $c_{h(ap)}$ value 1.5 times as much as c_v determined by the laboratory test.

Key words: consolidation, PVD, soft ground, vertical drain (**IGC**: E2/H7)

INTRODUCTION

Consolidation of the ground improved by vertical drain method is mainly controlled by horizontal coefficient of consolidation c_h, which in general is larger than vertical coefficient of consolidation c_v. Installation of vertical drain forms disturbed region around drain wells called smeared zone, which has lowered permeability. Since the disturbance around drains counteracts the effect of higher permeability in horizontal direction, it is often said that the apparent value of c_h (denoted $c_{h(ap)}$ hereinafter) is practically identical to c_v. Suzuki and Yasuhara (2007) analyzed the consolidation settlement which took place in the seawall construction for the second island of the Kansai International Airport. They obtained, with using $c_{h(ap)}$ equal to c_v, a successful result for a soft marine clay deposit improved by sand drain (SD) method. Kwong and Hian (2003) reported that $c_{h(ap)}$ three times as much as c_v well explained the actual settlement of the ground improved by prefabricated vertical drain (PVD) installed with two different arrangements. On the other hand, Foott et al. (1987) and Imanishi et al. (2000) revealed that $c_{h(ap)}$ changed from 50% to more than 130% of c_v according to PVD spacing. The facts in these literatures imply the difficulty of selecting an appropriate value of $c_{h(ap)}$.

It is possible to incorporate smear effect and the anisotropy of permeability into the calculation (Hansbo, 1981; Onoue, 1988; Shen et al., 2000). However, quantitative information about horizontal permeability and the effect of disturbance on it are not necessarily available in most routine soil investigations. Thus, accumulating field experiences is still important to decide the value of $c_{h(ap)}$ and to make the design of vertical drain method reliable. In this regard, this paper presents an experience of soil improvement work with PVD carried out for a port construction project in Hai Phong city, Vietnam.

Hai Phong city is located about 100 km east of Hanoi, the capital of Vietnam. The city has been built near the river mouth of the Red River. The port of Hai Phong, constructed along the Red River, has just expanded its berthing facility and container stacking area. Since the port area is widely covered with a soft clay deposit, an extensive application of soil improvement was required to minimize residual settlement and to open the container stacking area in a short period. In order to design soil improvement by PVD, including preloading sequence, engineering properties of the soft clay were determined by cone penetration test (CPTU), direct shear test (DST) and constant rate of strain consolidation test (CRST).

Open drive sampler (Shelby tube) is very common in this region for undisturbed sampling. However, this type of sampler can not provide high quality undisturbed soil samples for laboratory tests as revealed by Tanaka et al. (1992) and Tang et al. (1993). Therefore, undisturbed sampling was conducted with stationary piston thin wall sampler commonly used in Japan.

This paper first describes the undrained shear strength and consolidation characteristics of the soft clay found in the site according to the field and laboratory tests. Consolidation analysis for the PVD design in the project is also presented. The actual settlement induced by the preloading is then compared with the results of consolidation analysis in order to evaluate the value of $c_{h(ap)}$ in

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the field.

ENGINEERING PROPERTIES OF THE SOFT HAI PHONG CLAY

Subsoil Profile of the Site

Undisturbed sampling with stationary piston thin wall sampler was carried out near the center of the container stacking area, which is 200 m wide and 400 m long. The sampling tube has a 1000 mm length, a 75 mm inner diameter and a 1.5 mm wall thickness. The length of undisturbed samples is 850 mm when full penetration of the tube is achieved. The ground elevation around the drilling location was +2.7 m, and the ground water level was 0.8 m from the surface according to the measurement made after the drilling.

Natural water content and Atterberg limits of the subsoil are presented in Fig. 1(a). Natural water content is very close to liquid limit until the depth of 10 m, then, it is close to plastic limit below that depth. Plasticity index varies from 46 at the top to 25 below 5 m deep. Unit weight shown in Fig. 1(b) is around 16 kN/m^3 down to the depth of 10 m, then, it goes up to 19 kN/m^3.

CPTU was performed near the borehole. Figure 2 presents point resistance q_T-σ_{v0} and pore water pressure during penetration Δu. There is a crust layer on the top of the deposit where q_T-σ_{v0} is high. Low values of q_T-σ_{v0}, ranging from 150 kPa to 350 kPa, are observed from the depth of 0.8 m to 9.5 m. At the depth of 9.5 m, q_T-σ_{v0} steeply increases and Δu drops to the hydrostatic pressure. The deposit is stiff and sandy below this depth. Figure 2 demonstrates that the thickness of the soft clay found in the site is slightly less than 9 m.

Shear Strength Characteristics

In order to evaluate in-situ undrained shear strength s_{uf} of the deposit, DST was conducted with re-compression method, in which the specimen was first consolidated at an effective overburden stress σ'_{v0} in the ground, then, sheared at a deformation rate of 0.25 mm/min. The value of s_{uf} evaluated by DST is denoted $s_{uf(d)}$ in this paper. Hanzawa (2002) proposed Eq. (1) to determine undrained shear strength for stability analysis $s_{u(mob)}$ from DST result.

$$s_{u(mob)} = 0.85 s_{uf(d)} \tag{1}$$

The values of $s_{uf(d)}$ factored by 0.85 are plotted in Fig. 1(c), together with CPTU point resistance divided by cone factor N_{kT} equal to 15. The value of N_{kT} was selected to obtain the best agreement with DST results. With N_{kT} = 15, CPTU data yields $s_{u(mob)}$ as Eq. (2.1) for the surface crust where no undisturbed sample was taken. As presented in the figure, $s_{u(mob)}$ of the soft clay can be given by Eq. (2.2) against depth, where z is the depth from the surface measured in meter.

$$s_{u(mob)} = 60 \text{ (kPa)} \tag{2.1}$$
$$s_{u(mob)} = 8 + 1.6z \text{ (kPa)} \tag{2.2}$$

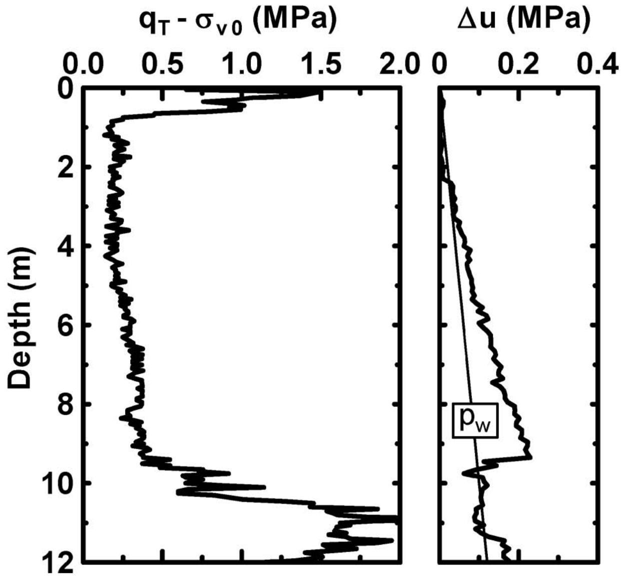


Fig. 2. Results of CPTU performed beside the borehole

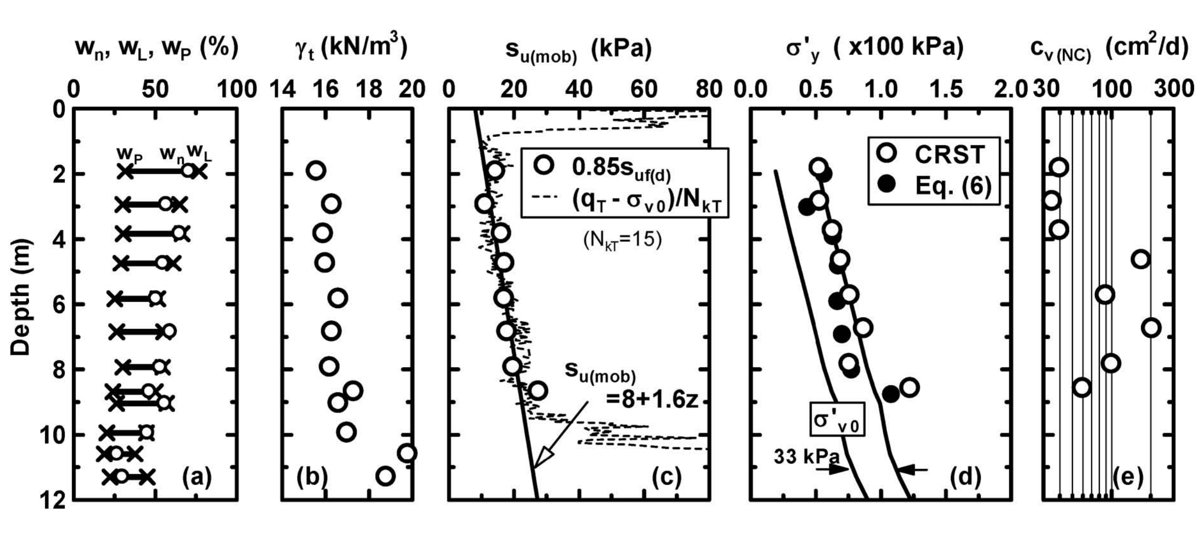


Fig. 1. Engineering properties of the soft Hai Phong clay

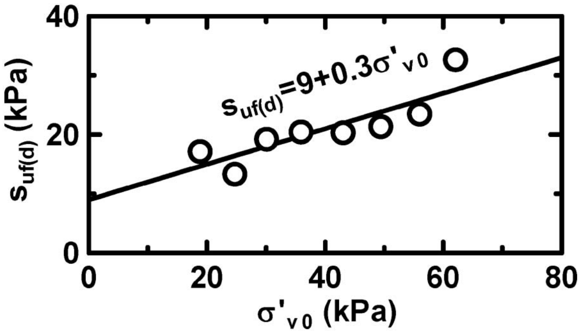


Fig. 3. Relation between s_{uf} and σ'_{v0}

It is well known that soft clays are in overconsolidated state even though they are not subjected to release of overburden stress because of chemical bonding and secondary compression. Hanzawa and Tanaka (1992) demonstrated that such clays show two patterns in the relation between s_{uf} and σ'_{v0}, which can be expressed by Eqs. (3) and (4), where s_{u0} is s_{uf} at $\sigma'_{v0}=0$ and k_1 and k_2 are constants.

$$\text{Pattern I:}\quad s_{uf}=s_{u0}+k_1\sigma'_{v0} \qquad (3)$$

$$\text{Pattern II:}\quad s_{uf}=k_2\sigma'_{v0} \qquad (4)$$

According to them, Pattern I, which represents chemical bonded zone, appears from the surface of the deposit up to a certain value of σ'_{v0}. Then, Pattern II, where overconsolidation is brought by secondary compression, follows. They also reported that k_1 was identical to undrained shear strength increment ratio in normally consolidated state s_{un}/σ'_v.

The values of $s_{uf(d)}$ are plotted in s_{uf}-σ'_{v0} diagram in Fig. 3. Pattern I is clearly observed, indicating that the soft clay deposit is in overconsolidated state due to chemical bonding. The s_{uf}-σ'_{v0} relation can be expressed by Eq. (5). This equation gives the value of 0.3 as $s_{un(d)}/\sigma'_v$ for the soft clay, in which the subscription (d) means that the value is evaluated by DST.

$$s_{uf(d)}=9+0.3\sigma'_{v0}\ (\text{kPa}) \qquad (5)$$

Consolidation Characteristics

Consolidation parameters of the clay deposit was evaluated by CRST performed at a strain rate of 0.02%/min ($=3.3\times10^{-6}$/s). Suzuki and Yasuhara (2004, 2007) reported that CRST at 0.02%/min provided preferable stress-strain curves for evaluating consolidation settlement of the ground improved by vertical drain method. Figure 4 presents e-log σ'_v curves, c_v and coefficient of permeability k determined by CRST. The inclination of e-log σ'_v curves in the figure gradually decreases after σ'_v passes consolidation yielding stress σ'_y. The values of c_v in normally consolidated state $c_{v(NC)}$ range from 35 to 200 cm^2/d.

Consolidation yielding stress can be determined by s_{uf} and s_{un}/σ'_v with Eq. (6) for overconsolidated clays formed by chemical bonding or secondary compression. The values of σ'_y determined by CRST are shown in Fig. 1(d),

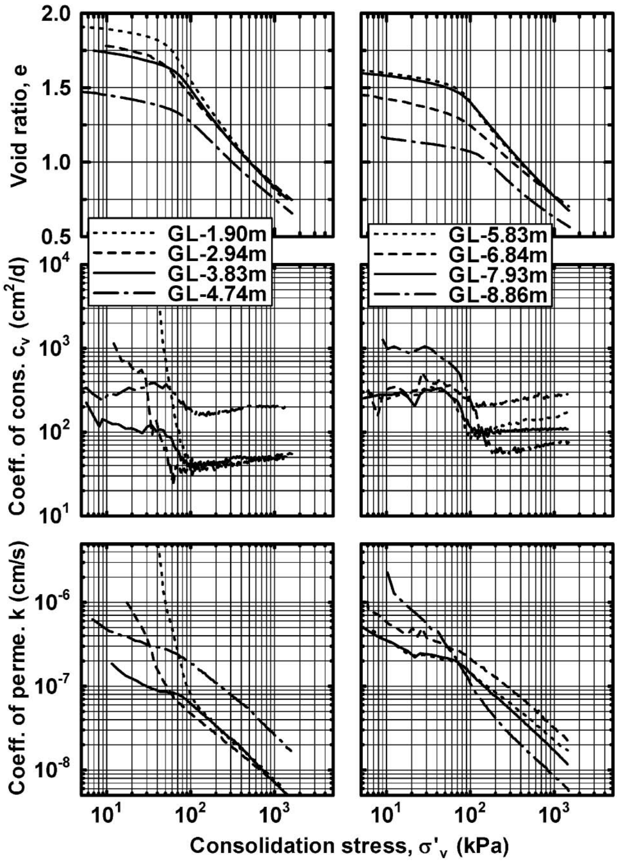


Fig. 4. Results of CRST at 0.02%/min

together with those evaluated by Eq. (6). The values of σ'_y from CRST are substantially the same as those estimated by Eq. (6). The amount of overconsolidation in terms of σ'_y-σ'_{v0} is about 33 kPa for the soft clay as shown in Fig. 1(d). The $s_{u(mob)}$ of the surface crust is 60 kPa as presented by Eq. (2.1). This value can be interpreted to $s_{uf(d)}$ as 71 ($=60/0.85$) kPa. Consequently, σ'_y of this part is 236 kPa from Eq. (6), when $s_{un(d)}/\sigma'_v=0.3$ is employed.

$$\sigma'_y=s_{uf}\div(s_{un}/\sigma'_v) \qquad (6)$$

The values of $c_{v(NC)}$ are plotted versus depth in Fig. 1(e). They are very diverse for the depth from 4 m to 9 m.

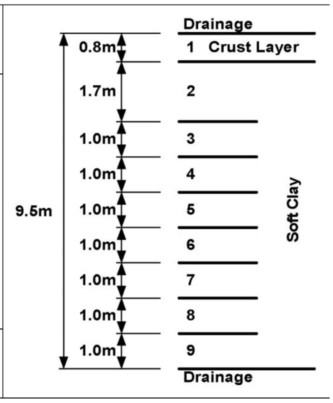
Parameters for Consolidation Analysis

The subsoil is divided into 9 sub-layers for the purpose of consolidation analysis. Table 1 summarizes the subsoil stratification and consolidation parameters. It is assumed that each CRST result represents consolidation characteristics of each sub-layer. How to determine the parameters is the same as Suzuki and Yasuhara (2007). Since sub-layer 1 (the surface crust) has very high value of σ'_y as mentioned above, the parameters for this sub-layer are chosen to eliminate any compression strain to take place. The same value of c_v as sub-layer 2 is assumed for sub-layer 1. In order to express compressibility from overconsolidated state to normally consolidated state, three values of compression indices are derived for each sub-layer from corresponding CRST result.

Table 1. Parameters used for consolidation analysis

Sub layer	Sample Depth (m)	e_0	σ'_{v0} (kPa)	σ'_y (KPa)	σ'_b (kPa)	C_{c1}	C_{c2}	C_{c3}	$c_{v(NC)}$ (cm^2/d)
1	—	1.000	6.2	236.0	—	0.001	—	—	40
2	1.90	1.869	19.0	52.7	180	0.15	0.91	0.70	40
3	2.94	1.731	24.9	53.0	500	0.30	0.65	0.50	35
4	3.83	1.680	30.3	63.1	300	0.14	0.75	0.56	40
5	4.74	1.389	36.1	69.4	500	0.14	0.53	0.47	170
6	5.83	1.536	43.2	76.3	200	0.14	0.77	0.60	90
7	6.84	1.333	49.6	86.9	500	0.25	0.48	0.43	200
8	7.93	1.495	56.2	75.9	500	0.15	0.65	0.58	100
9	8.86	1.098	62.2	122.6	400	0.12	0.52	0.42	60

σ'_b: σ'_v at the boundary of C_{c2} and C_{c3}. C_{c2}: Compression index for $\sigma'_y < \sigma'_v < \sigma'_b$.
C_{c1}: Compression index for $\sigma'_{v0} < \sigma'_v < \sigma'_y$. C_{c3}: Compression index for $\sigma'_v > \sigma'_b$.



PVD DESIGN FOR THE PROJECT

Determination of $c_{h(ap)}$

In order to determine the value of $c_{h(ap)}$, experience from Kansai International Airport and some published data were studied as described in the appendix of this paper. According to the study in the appendix, it was decided that the PVD improvement for the soft Hai Phong clay could be designed with using $c_{h(ap)}$ equal to $c_{v(NC)}$ in Table 1. Since this decision was made by studying the data from different clays found in different regions, its appropriateness for Hai Phong clay was uncertain. However, the design of PVD for the project was carried out according to this decision.

Installation of PVD makes holes in the ground. If they do not close immediately after the withdrawal of mandrel, they would affect the settlement behavior. However, since there is no available information regarding this matter, the effect of openings in the ground was not considered in the design of PVD.

Design of PVD Spacing and Preloading

Consolidation analyses were conducted to find out an appropriate PVD installation pattern, the minimum thickness of the preload embankment and the minimum duration of the preloading that can satisfy the requirements in the specification and can meet the construction schedule. The calculations by finite difference method were performed with the same way as Suzuki and Yasuhara (2007) with using the parameters summarized in Table 1.

An example of the calculation is demonstrated in Fig. 5. Design elevation of the container stacking area is +5.0 m and the specification requires that the residual settlement should be less than 10% of the final settlement considering 40 kPa as the operation load. The swelling during unloading (removal of the preload embankment) and the settlement during reloading (operation load) were calculated with compression index one-tenth of C_{c2} as long as the deposit stayed overconsolidated. It was assumed that the unit weight of the filling sand and the embankment was 18 kN/m^3. A number of cases were analyzed, and the PVD work for the project was designed as follow.

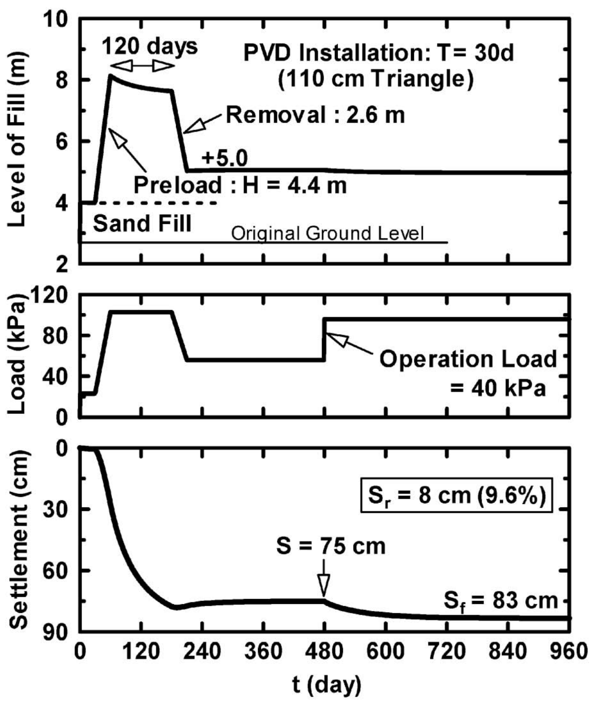


Fig. 5. An example of calculation for PVD design

PVD installation pattern:	110 cm, triangle arrangement
Preload embankment:	up to +8.0 m
Preloading period:	minimum 120 days

It was decided that PVDs should be installed down to the bottom of the sandy layer resting beneath the soft clay to ascertain the drainage from this layer. Slope gradient of the preload embankment was determined to satisfy the minimum safety factor of 1.1 against circular slip with using Eqs. (2.1) and (2.2).

PERFORMANCE OF PVD

PVD installation was carried out after the container stacking area was filled with filling sand up to +4.0 m. The mandrel used has an outer diameter of 13 cm. Material for the preload embankment was changed from sand to crushed stone in order to use it as pavement material after the end of preloading. The preloading was conducted by dividing the container stacking area into six plots. The dimension of the preload embankment for the area including the borehole location is shown in Fig. 6, together with the arrangement of settlement plates placed inside the filling sand. The placement of the settlement plates were carried out one to two weeks after the PVD installation.

Figure 7 presents the monitored settlement and the filling process of the preload embankment obtained for 6 locations around the borehole. The monitored values were

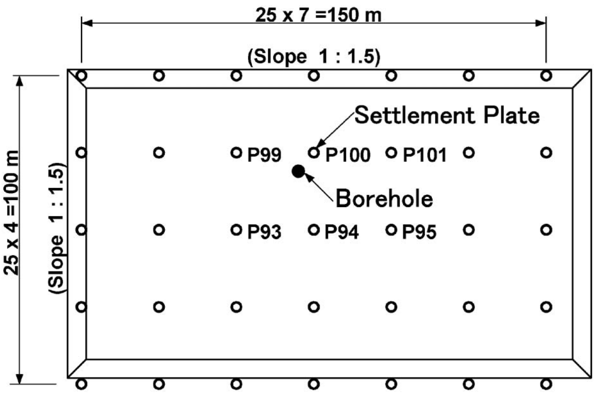


Fig. 6. Dimension of the preload embankment and monitoring plan

moved upward several centimeters (shown in the blankets in the figure) for the purpose of comparison between the measured and calculated time-settlement relations. The measured settlements reach about 100 cm. Since the change of the material for the preload embankment causes a larger preloading stress than that used in the design, the actual settlement is greater than that predicted in Fig. 5.

All of the measurements in Fig. 7 demonstrate that the settlement of the ground mostly finished within 120 days after the embankment reached the designed elevation. This phenomenon is well observed in P101 where the filling from +4.0 m to +8.0 m was carried out instantaneously at $t=175$ days. It can be concluded from the figure that the settlement proceeded as expected.

The monitored settlements in Fig. 7 seem mostly finished at $t=250$ days. Therefore, there is possibility of secondary compression at $t>250$ days. Secondary compression is usually estimated with assuming that it proceeds proportionally to time in log scale. Since the coefficient of secondary compression can not be determined directly from CRST, it is assumed that $0.05C_c$ proposed by Mesri and Castro (1987) is applicable. The values of C_{c2} in Table 1 factored by 0.05 are used to evaluate secondary compression in the site. When it is assumed that secondary compression takes place at $t>250$ days, its estimated time-settlement relation can be given by the line in

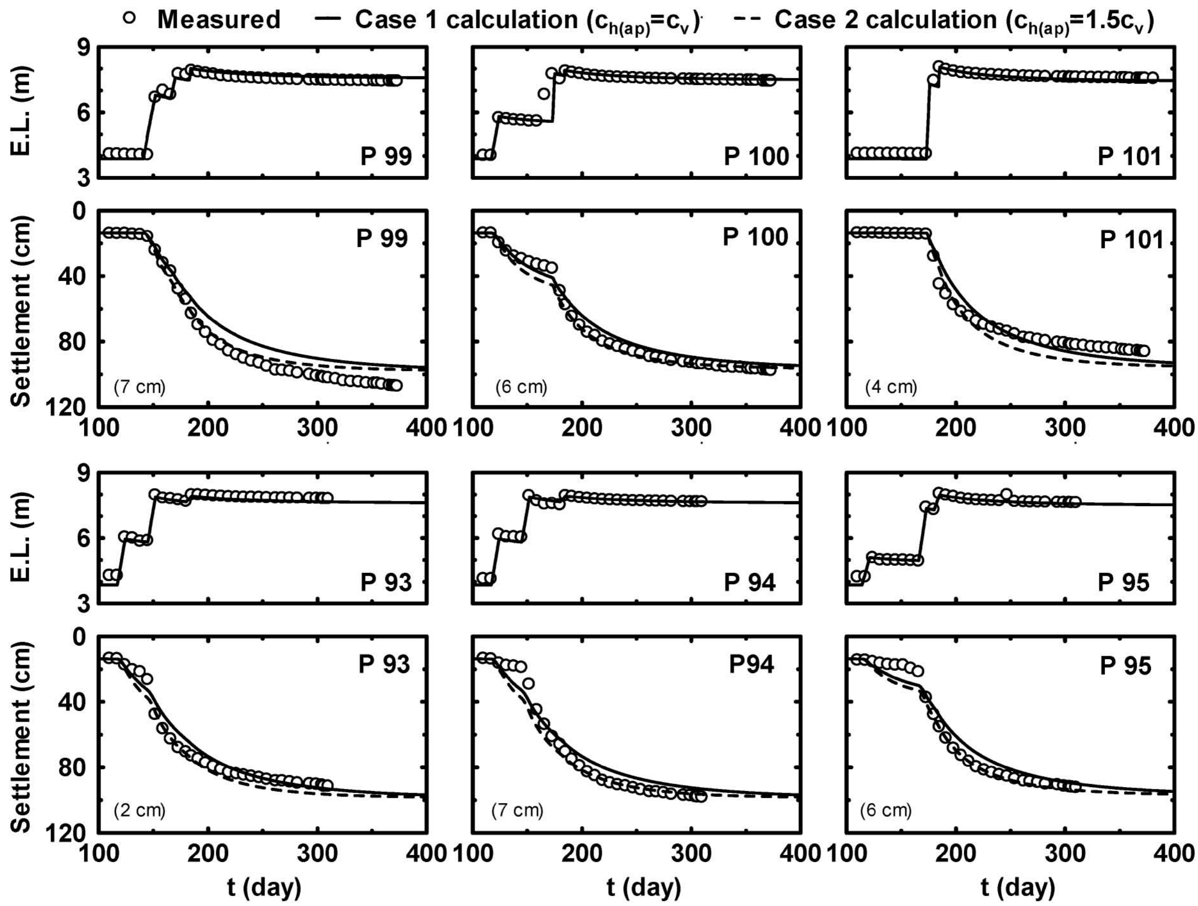


Fig. 7. Calculation with $c_{h(ap)}/c_v=1$ and 1.5 compared with the actual settlement behavior

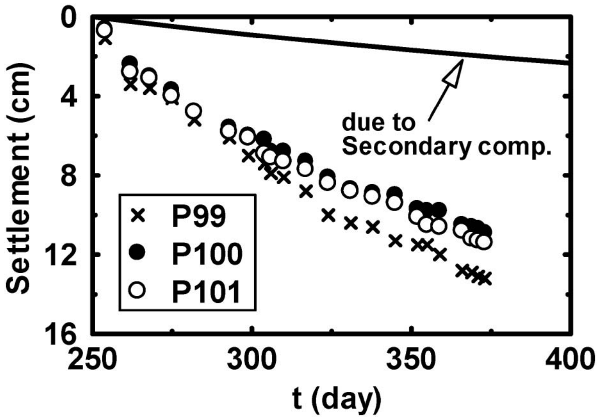


Fig. 8. Measured settlement after $t=250$ days and assumed secondary compression

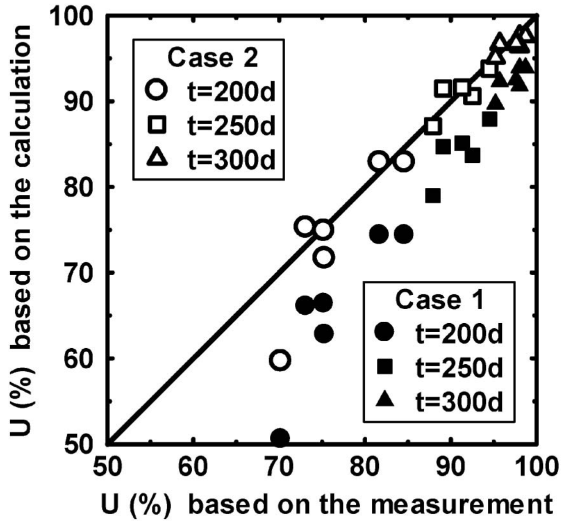


Fig. 9. Degree of consolidation based on the measurement and calculation

Fig. 8, in which the measured values at P99, P100 and P101 are also presented. The settlement due to secondary compression is much smaller than the observed ground settlement after $t=250$ days. As can be seen in the figure, the contribution of secondary compression to the monitored settlement is negligible, even if its occurrence is assumed.

THE VALUE OF $c_{h(ap)}$ BASED ON THE MEASUREMENT

Effect of Well Resistance

Coefficient of permeability of the clay k_c is about 2×10^{-7} to 3×10^{-7} cm/s at around $\sigma'_v=100$ kPa as indicated in Fig. 4. While, according to the catalog data, coefficient of permeability of drain well k_w is 1 cm/s. When the permeability in horizontal direction is higher than that in vertical direction by a couple of times, it is expected that the ratio of k_c/k_w is at most 1×10^{-6}. If this is the case, coefficient of well resistance L (Yoshikuni, 1979) becomes 0.026 when drainage distance of PVD is 450 cm (double drainage condition) and d_w is 5 cm. The ratio of k_c/k_w mentioned above does not include the effect of smear around PVD, which reduces apparent value of k_c. Therefore, the value of L must be less than 0.026. This implies that the effect of well resistance is very small and ignorable.

Comparison between the Monitored and Calculated Settlement

Measured value of dry unit weight of the embankment material is 19.5 kN/m^3. When water content of 10% and particle density of 2.7 g/cm^3 are used, the dry unit weight mentioned above gives 21 kN/m^3 as wet unit weight of the material. The consolidation analysis for the purpose of comparison with the monitored settlement is carried out with this value. The effect of well resistance is not included in the calculation.

Figure 7 also demonstrates the calculations with $c_{h(ap)}/c_v=1.0$ (Case-1) and $c_{h(ap)}/c_v=1.5$ (Case-2). The following can be pointed out.

1) Case-2 calculation is better than Case-1 up to around $t=300$ days. For the most period of the consolidation, it proceeds with $c_{h(ap)}/c_v=1.5$.
2) In spite of that the actual settlement is still continuing at $t>300$ days, Case-2 calculation produces almost no settlement. Rather, Case-1 calculation is close to the actual behavior at $t>300$ days.

In order to make clear which case well represents the actual consolidation behavior in the site, degree of consolidation U is compared in Fig. 9 for $t=200$, 250 and 300 days. For the calculation of U based on the measurement, observational method (Asaoka, 1978) was applied to the monitored values from $t=200$ to 300 days to estimate the final settlement. The values of U based on the Case-2 calculation show good agreement with those based on the measurement when U is greater than 70%. This figure clearly demonstrates that Case-2 ($c_{h(ap)}/c_v=1.5$) is better than Case-1 ($c_{h(ap)}/c_v=1.0$) for the purpose of construction control of the preloading work. Although, as previously pointed out, the actual settlements do not follow the Case-2 calculation all the way through the consolidation, it can be concluded, from practical point of view, that the value of $c_{h(ap)}/c_v$ in the field is 1.5.

Reason of $c_{h(ap)}$ Higher than Expected

The APPENDIX of this paper suggests that the ratio of $c_{h(ap)}/c_v$ is practically equal to one, and it is substantially independent of PVD spacing. The PVD improvement for the soft Hai Phong clay was designed based on $c_{h(ap)}/c_v=1$. But actual value of $c_{h(ap)}/c_v$ resulted in 1.5 according to the comparison between the field monitoring and the consolidation analyses.

Subsoil condition sometimes influences the consolidation with vertical drain. Even though their distribution is sporadic, sand seams or lenses existing within a soft clay deposit become drainage boundary after vertical drains are installed, and make an increase in $c_{h(ap)}$. Sand seams and lenses cause a sudden increase in CPTU point resistance q_T-σ_{v0} and a sudden drop in Δu. The CPTU results presented in Fig. 2 have no such indication inside the soft clay. Therefore, the subsoil condition is not the reason for the $c_{h(ap)}$ value higher than expected.

The ratio of $c_{h(ap)}/c_v$ equal to 1 is derived with assuming $\alpha(=c_h/c_v)=2$ as described in the APPENDIX. Another possible reason of larger $c_{h(ap)}$ is that the actual value of α is higher than expected. Figure A1 in the APPENDIX suggests that the value of α should be between 3.0 to 3.5 to obtain $c_{h(ap)}/c_v=1.5$ for $n=23.1$ (110 cm triangle arrangement and $d_w=5$ cm). As mentioned in the APPENDIX, Mizukami et al. (1994) concluded that α was at most two. However, when their data is scrutinized, some data of α reach 3.0, and there is a trend in which the value of α increase as c_v decreases. The values of c_v in their data vary from 50 to 200 cm^2/d. While, those in Fig. 1(e) range from 35 to 200 cm^2/d. Therefore, $\alpha>3$ seems possible for the soft Hai Phong clay. Although it is very difficult to draw a clear conclusion only from the comparison between the field measurement and the calculation, the value of $c_{h(ap)}$ larger than expected by 1.5 times may be resulted from the c_h value close to 3.5 times as much as c_v.

CONCLUSIONS

PVD application for the soft Hai Phong clay was reported together with its engineering properties investigated by field and laboratory tests, and the value of $c_{h(ap)}/c_v$ was examined according to the field measurement and the consolidation analysis. The field monitoring of the settlement presented an implication that the consolidation with PVD progressed with $c_{h(ap)}/c_v>1$ immediately after the loading and with $c_{h(ap)}/c_v=1$ as the end of consolidation was approached. However, practical point of view, the field monitoring can be concluded as the actual settlement proceeded with $c_{h(ap)}/c_v=1.5$.

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APPENDIX: DISCUSSIONS ON $c_{h(ap)}$

Barron's approximation is very popular in the design of vertical drain method. It can be given by Eqs. (A1) to (A3), where $U_{(B)}$ is the degree of consolidation from Barron's approximation, n is the ratio of the diameter of effective circle d_e to that of drain well d_w, and t is time. In order to obtain a reliable prediction from this approximation, an appropriate value of $c_{h(ap)}$ should be selected.

$$U_{(B)}=1-\exp\left(-\frac{8T_{h(B)}}{F_{(B)}}\right) \tag{A1}$$

$$T_{h(B)}=\frac{c_{h(ap)}}{d_e^2}t \tag{A2}$$

$$F_{(B)}=\frac{n^2}{n^2-1}\left(\ln n-\frac{3}{4}\right)\quad\left(n=\frac{d_e}{d_w}\right) \tag{A3}$$

Hansbo (1981) derived an approximation that can consider smear effect and well resistance. According to Miura et al. (1993), the effect of well resistance on the consolidation with PVD is usually small. When the effect of well resistance is ignored, his approximation can be expressed by Eqs. (A4) to (A6), in which $U_{(H)}$ is the degree of consolidation from Hansbo's approximation, α is the ratio of c_h to c_v, s is the ratio of the diameter of smeared zone d_s to d_w, and K is the ratio of the permeability of undisturbed clay k_c to that in smeared zone k_s.

$$U_{(H)} = 1 - \exp\left(-\frac{8T_{h(H)}}{F_{(H)}}\right) \tag{A4}$$

$$T_{h(H)} = \frac{c_h}{d_e^2}t = \frac{\alpha c_v}{d_e^2}t \quad \left(\alpha = \frac{c_h}{c_v}\right) \tag{A5}$$

$$F_{(H)} = \frac{n^2}{n^2-1}\left(\ln\frac{n}{s} + K\ln s - \frac{3}{4}\right) + \frac{s^2}{n^2-1}\left(1 - \frac{s^2}{4n^2}\right) + \frac{K}{n^2-1}\left(\frac{s^4-1}{4n^2} - s^2 + 1\right) \quad \left(s = \frac{d_s}{d_w},\ K = \frac{k_c}{k_s}\right) \tag{A6}$$

Since Eqs. (A1) and (A4) should produce the same value of degree of consolidation, the two equations yield Eq. (A7) that gives the ratio of $c_{h(ap)}/c_v$.

$$\frac{c_{h(ap)}}{c_v} = \alpha\frac{F_{(B)}}{F_{(H)}} \tag{A7}$$

It is needed to determine the values of n $(=d_e/d_w)$, s $(=d_s/d_w)$, K $(=k_c/k_s)$ and α $(=c_h/c_v)$ to obtain the ratio of $c_{h(ap)}/c_v$ from Eq. (A7). In this report, they were determined by studying published data, excepting the value of n.

According to laboratory experiment for Boston Blue Clay, Onoue (1991) recommended the combination of $s=1.6$ and $K=3$. He determined the value of K by comparing the permeability of remolded samples with that of undisturbed samples. In the case of PVD, the value of s is influenced by the dimensions of mandrel. However, mandrel installation for PVD makes holes in the ground, which close up after its withdrawal and during consolidation. This does not take place for SD installation because the diameter of sand pile is practically the same as mandrel. Thus, formation of smeared zone during PVD installation seems somewhat different from SD installation. Although, applicability to PVD is not clear, $s=1.6$ given by Onoue (1991) is used here for further discussion.

When it is assumed that Onoue's conclusion of $s=1.6$ is applicable to mandrel diameter, the value of s for PVD becomes 4.2 as shown by Eq. (A8) when mandrel diameter is 13 cm and equivalent value of d_w of PVD is 5 cm (100 mm wide and 3 mm thick). Figure A1 demonstrates the ratio of $c_{h(ap)}/c_v$ determined by Eq. (A7) for $K=3$, and for the values of α from 1.5 to 3.5. The left side of the figure presents the curves for SD ($s=1.6$), and the right side for PVD ($s=4.2$). Since it is assumed that the size of smeared zone is independent of drain spacing, smear effect on $c_{h(ap)}$ decreases as drain spacing increases and the ratio of $c_{h(ap)}/c_v$ increases as the value of n increases.

$$s_{(\text{for PVD})} = (13\ \text{cm} \times 1.6)/5\ \text{cm} = 4.2 \tag{A8}$$

In order to determine the value of α, consolidation settlement of a soft clay deposit under the seawall of the second island of the Kansai International Airport (KIA) was analyzed. The soft clay in KIA site was improved by SD ($d_w=40$ cm). The installation pattern is 1.6 m × 2.5 m, which is equivalent to 2.0 m square arrangement ($n=5.65$). The calculation with $c_{h(ap)}=c_v$ was already presented in Suzuki and Yasuhara (2007). Additional calculations were carried out with $c_{h(ap)}=1.2\,c_v$ and $c_{h(ap)}=2\,c_v$. The results are compared with the monitored behavior in Fig. A2. It can be pointed out for $t=200$ days to 600 days that 1) the calculation with $c_{h(ap)}=c_v$ well agrees with the measurement, 2) the difference between 1.0 c_v and 1.2 c_v is very small, and 3) the calculation with $c_{h(ap)}=2\,c_v$ yields the time-settlement relation faster than the actual one. The reason of the discrepancy between the measurement and the calculation observed at the beginning of consolidation and $t>650$ days can be found in Suzuki and Yasu-

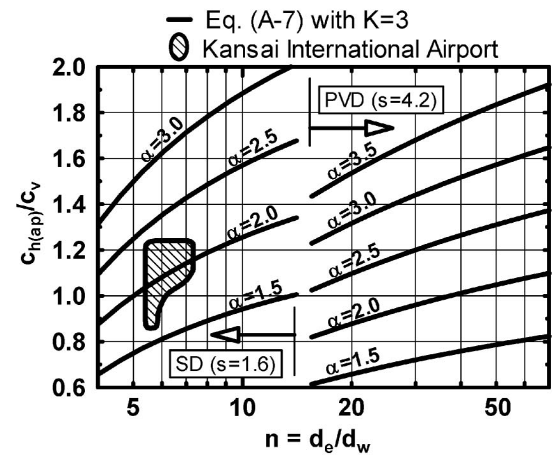

Fig. A1. The ratio of $c_{h(ap)}/c_v$ given by Eq. (A7) with $K=3$ plotted versus n

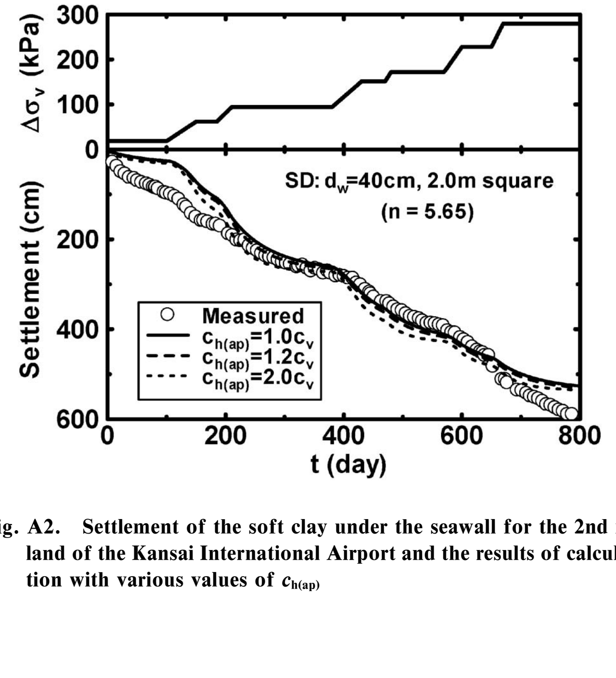

Fig. A2. Settlement of the soft clay under the seawall for the 2nd island of the Kansai International Airport and the results of calculation with various values of $c_{h(ap)}$

hara (2007). Arai et al. (1991) presented their SD experience in the first island of KIA. They used $c_{h(ap)}=0.9\ c_v$ under the seawall where $n=5.65$, and $c_{h(ap)}=1.2\ c_v$ under the reclamation area where $n=7.06$. The relation between $c_{h(ap)}$ and n obtained from KIA construction are also presented in Fig. A1. The best agreement between Eq. (A7) and KIA experience is given by the curve which corresponds to $\alpha=2.0$.

Mizukami et al. (1996) and Nishimura et al. (2004) experimentally investigated the value of α, and found that it was at most two. The conclusion drawn from Fig. A1 is on the upper limit of the variation in α determined by them.

When $\alpha=2.0$ is employed, it can be concluded for PVD from Fig. A1 that $c_{h(ap)}/c_v$ varies from 0.8 to 1.1 for a wide variation of n from 15 to 70. As long as the value of n stays within this variation, difference in $c_{h(ap)}/c_v$ yields no significant effect on resulting time-settlement prediction, as implied by Fig. A2 where the calculation with $1.0\ c_v$ and $1.2\ c_v$ are demonstrated.

With the experimental data of $s=1.6$ and $K=3$ given by Onoue (1991), sand drain experience at KIA construction yields the conclusion that the value of α is 2. Then, this conclusion and Eq. (A7) provides another conclusion that the ratio of $c_{h(ap)}/c_v$ is substantially equal to 1 for a wide range of PVD spacing. Since these conclusions are based only on the data provided by Onoue (1991), they are examined by other values of s reported by other researchers. For example, Shen et al. (2000) recommended $s=3$, and Sharma and Xiao (2000) reported $s=4$. When the same manner as Eq. (A8) is used, these values of s are equivalent to $s=7.8$ and 10.4, respectively, for the case of PVD with a 13 cm diameter mandrel and a 5 cm diameter drain well. In order to get $c_{h(ap)}/c_v$ being close to 1 from Eq. (A7) under the condition of $\alpha=2$, it is needed to assume $K=2.4$ when $s=7.8$, and $K=2.2$ when $s=10.2$. These values of K are 70 to 80 % of the value given by Onoue (1991). Since the difference is not so large, it can be said that the conclusions mentioned above are acceptable.