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Improvement of soft clay at a site in the Mekong Delta by vacuum preloading

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Abstract. Soil improvement by preloading with PVD in combination with vacuum is helpful when a considerable load is required to meet the desired rate of settlement in a relative short time. To facilitate the vacuum propagation, vertical drains are usually employed in conjunction. This ground improvement method is more and more applied in the Mekong delta of Vietnam to meet the needs of fast infrastructure development. This paper reports on a pilot test that was carried out to investigate the effect of ground improvement by vacuum and PVD on the rate of consolidation at the site of Saigon International Terminals Vietnam (SITV) in Ba Ria-Vung Tau Province, Viet Nam. Three main aspects of the test will be presented, and namely, instrumentation and field monitoring program, calculation of consolidation settlement and back-analysis of soil properties to see the difference before and after ground improvement.

Keywords: Mekong Delta; soft clay; soil improvement; preloading; PVD; vacuum; back-analysis

1. Introduction

Mekong delta has one of the most extensive soft clay deposits in the world (Giao *et al*. 2008). With the fast development of infrastructure in Southern Vietnam many ground improvements techniques are considered and applied in practice. Vacuum consolidation preloads the soil by reducing the pore pressure while maintaining constant total stress instead of increasing the total stress. The net effect is an additional surcharge ensuring early attainment of the required settlement and an increased shear strength resulting in increased embankment stability.

Kjellman (1948) first introduced the concept of using vacuum preloading to improve the soil strength. Significant progress in the design of vertical drains for accelerating consolidation of foundation soils has been made in the past two decades through theoretical analysis, laboratory and field performance observations, e.g., Hansbo 1979, 1981, Holtz *et al*. 1991, Bergado *et al*. 1998 and Chai and Miura 1999, Chai *et al*. 2006, Chai *et al*. 2010. Recently, many successful field applications have been reported, e.g., Shang *et al*. 1998, Tang and Shang 2000 and Chu *et al*. 2000. However, a proper selection of design parameters remains difficult (Hansbo 1997) due to the soil disturbance caused by installation of PVDs and three-dimensional nature of the consolidation progress. One therefore often has to resort to pilot tests with pore-pressure and settlement

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measurements for verification of design and determining the appropriate consolidation parameters (Cao *et al*. 2001).

This paper summarizes the results of a pilot test employing prefabricated vertical drains combined with vacuum and surcharging for improvement of foundation clays at a land reclamation site in Viet Nam. Settlement analysis and back-calculation of the consolidation parameters from the field records of settlement and pore-pressure were made. The results of back-analyses were compared with those obtained from laboratory and field soil investigation tests before and after treatment.

2. Site conditions

A full-scale embankment test was constructed in stages on a subsoil improved by preloading with PVDs and vacuum. The pilot test area of 85 × 73 × 251 m in plan dimension is shown in Fig. 1(a). A soil investigation program, including boring, undisturbed sampling, piezocone and field vane testing was conducted to ascertain the soil properties at the site.

The soil profile is relatively uniform, consisting of a 2.5 m thick weathered crust (WC) overlying a layer of very soft to soft clay of approximately 10 m thick. Beneath the soft clay is a medium clay layer about 7 m thick, followed by a sand layer, which is underlain by a layer of hard clay. The natural water contents are uniform across the test site and lie close to the liquid limit between depths from 0 to 17 m (Fig. 2).

Consolidation tests using oedometer and constant rate of strain (CRS) tests were performed on undisturbed samples obtained from the site before treatment. Soil properties, including unit weight (γ) water content (w), void ratio (e), liquid limit (LL), compression ratio (CR), recompression ratio (RR), vertical coefficient of consolidation (C_v) and undrained shear strength (s_u) with depth were shown in Fig. 2 and Table 1. The C_v values were determined from the oedometer and CRS tests using Casagrande's method. The s_u values and horizontal coefficient of consolidation (C_h) values were derived from field vane shear and piezocone tests, respectively.

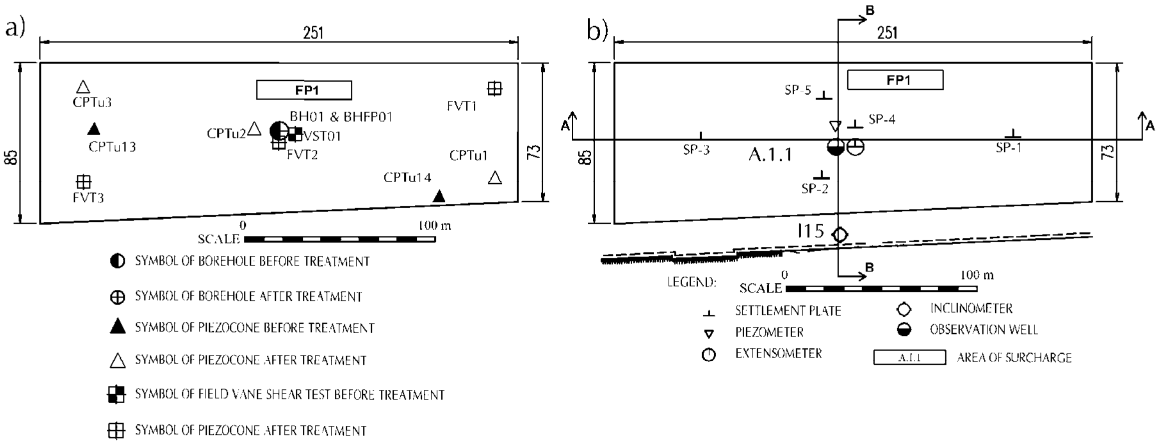


Fig. 1 (a) Locations of boreholes; and (b) Layout of installation of field instrumentations

Table 1 Summary of soil parameter used in analysis

Physical & mechanical properties		Units	Upper layer		Lower layer	
			WC	Very soft clay	Soft clay	Medium clay
			Depth (m)			
			+2.5 ~ 0.0	0.0 ~ –6.0	–6.0 ~ –10.0	–10.0 ~ –17.0
w		%	67.8	79.7	70.2	60.9
γ		kN/m^2	15	15	15.5	16.2
e		-	1.73	2.02	1.84	1.57
CR		-	0.179	0.31	0.23	0.23
RR		-	0.023	0.034	0.034	0.032
C_v90（from CRS and oedometer test）		m^2/yr	2.3	1.1	1.1	1.2
C_h (from piezocone test)	Before correction	m^2/yr	25.6	19.1	11.5	12.9
	After correction*	m^2/yr	3.3	2.1	1.7	1.8

*C_h (NC) = (RR/CR). C_h (piezocone) (Baligh and Levadoux's equation (1986)), NC is normal consolidation state

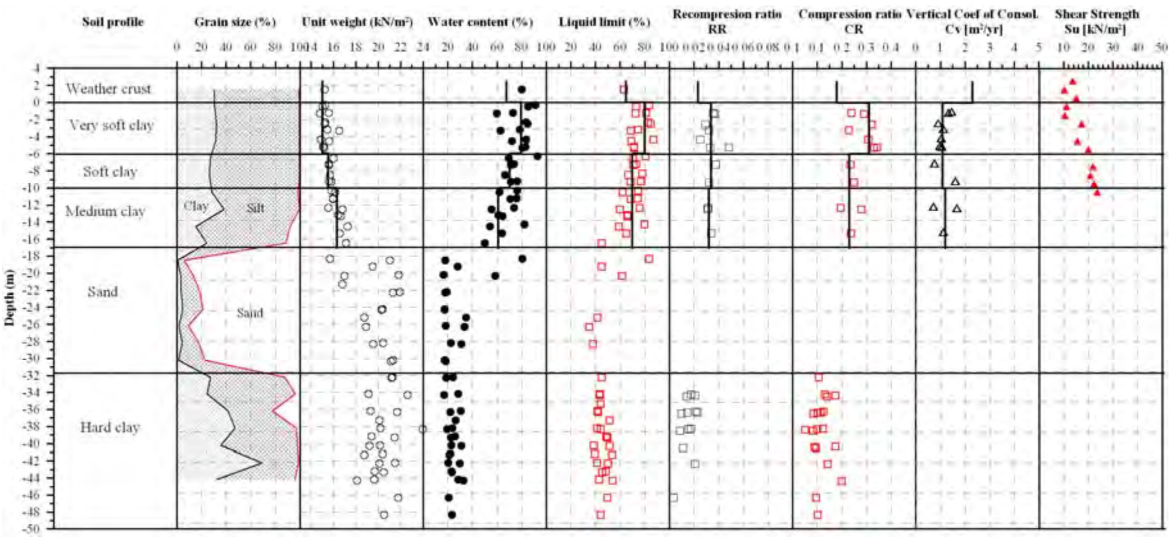
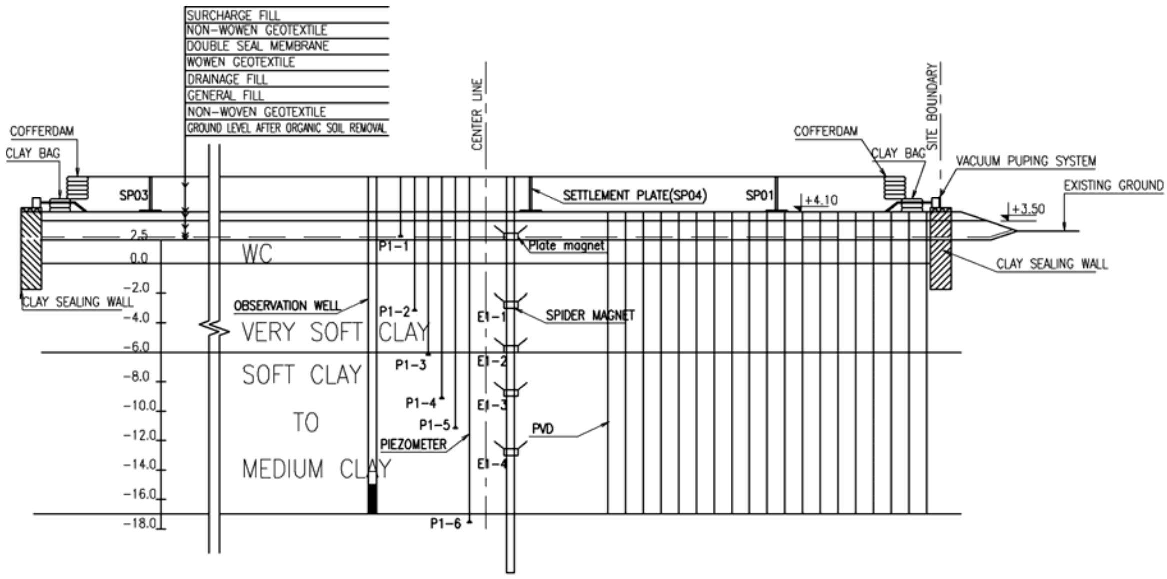


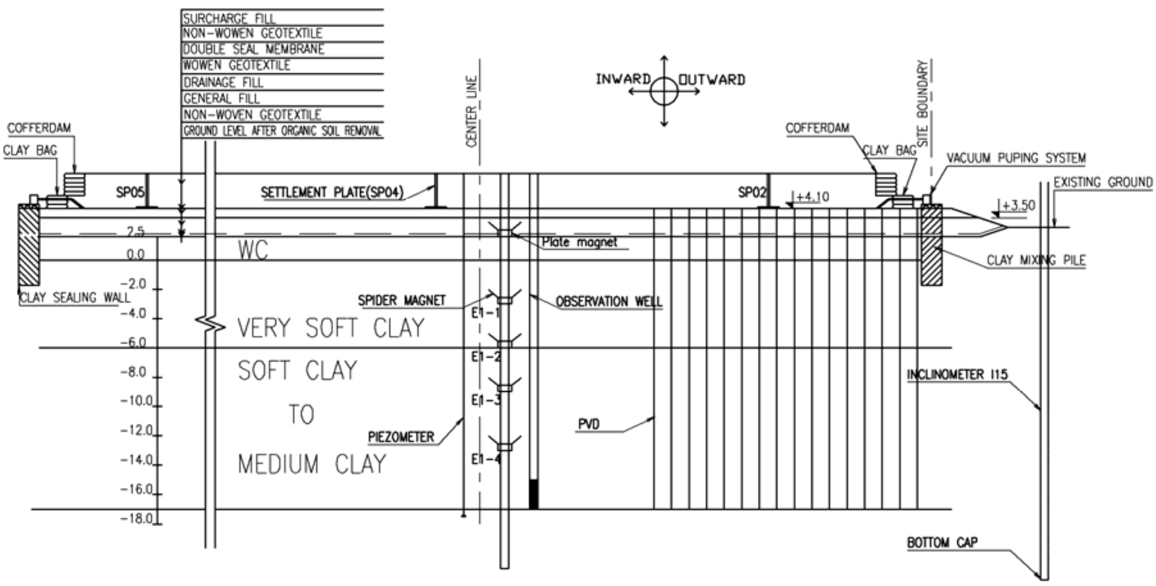
Fig. 2 Physical and mechanical properties of the subsoil at the testing site

3. Construction and instrumentation of test embankment

The sequence of construction and instrumentation is shown in Figs. 3(a)-(b). At the test embankment site, the original ground was cleared of grass roots and excavated to 0.5 m below mean sea level. Organic soil was removed at + 2.5 m Chart Datum (CD), a non-woven geotextile was laid, and then backfilled with sand layer to level of + 3.5 m CD, followed by a drainage fill layer of 0.6 m thick. At + 4.1 m CD, the PVDs were installed to the depth of 21 m, on a triangular pattern with 1.2 m spacing. A layer of woven geotextile was laid between the drainage fill and geomembrane when the placement of vacuum pipe was finished. Two layers of polyethylene (or PVC) geomembrane were laid upon the woven geotextile with a thickness of about 0.12 ~ 0.14 mm. Before applying surcharge loading, a layer of non-woven geotextile was laid on surface of the



(a) Section A-A



(b) Section B-B

Fig. 3 Views of instrumentation sections

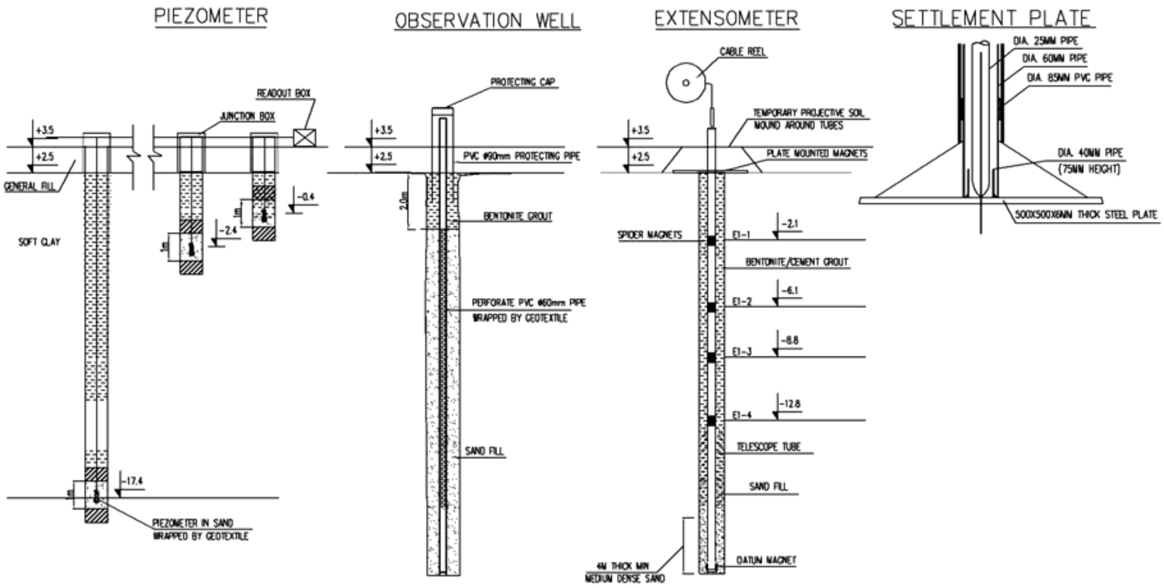


Fig. 4 Details of field instrumentations

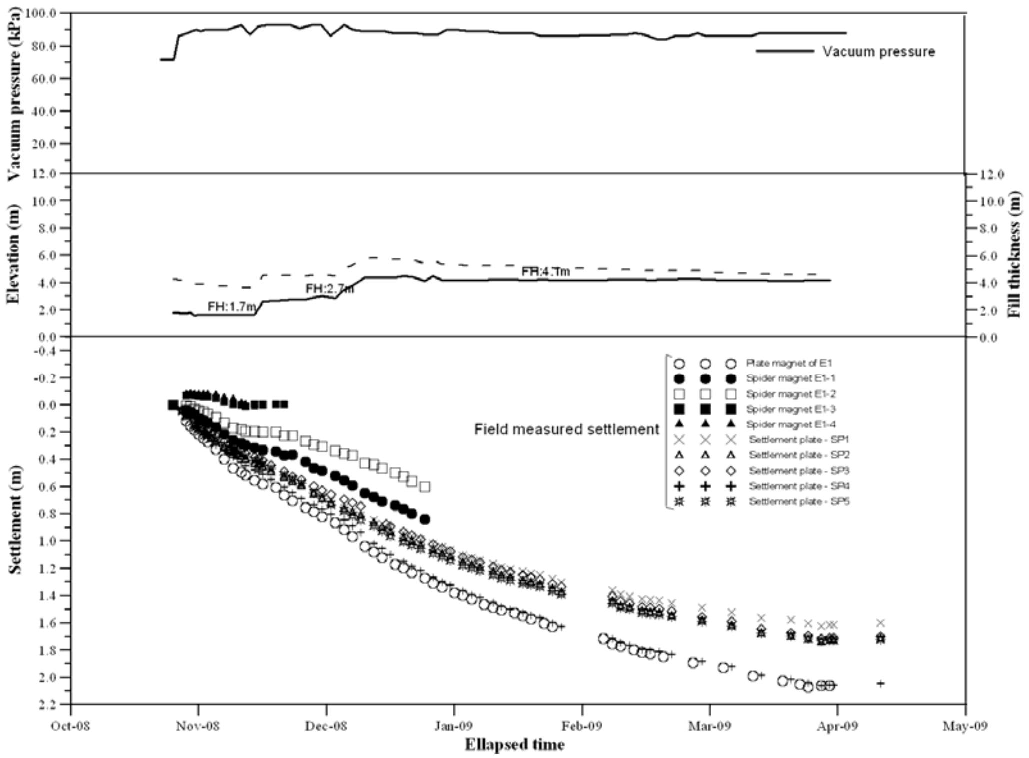


Fig. 5 Field settlement measurements

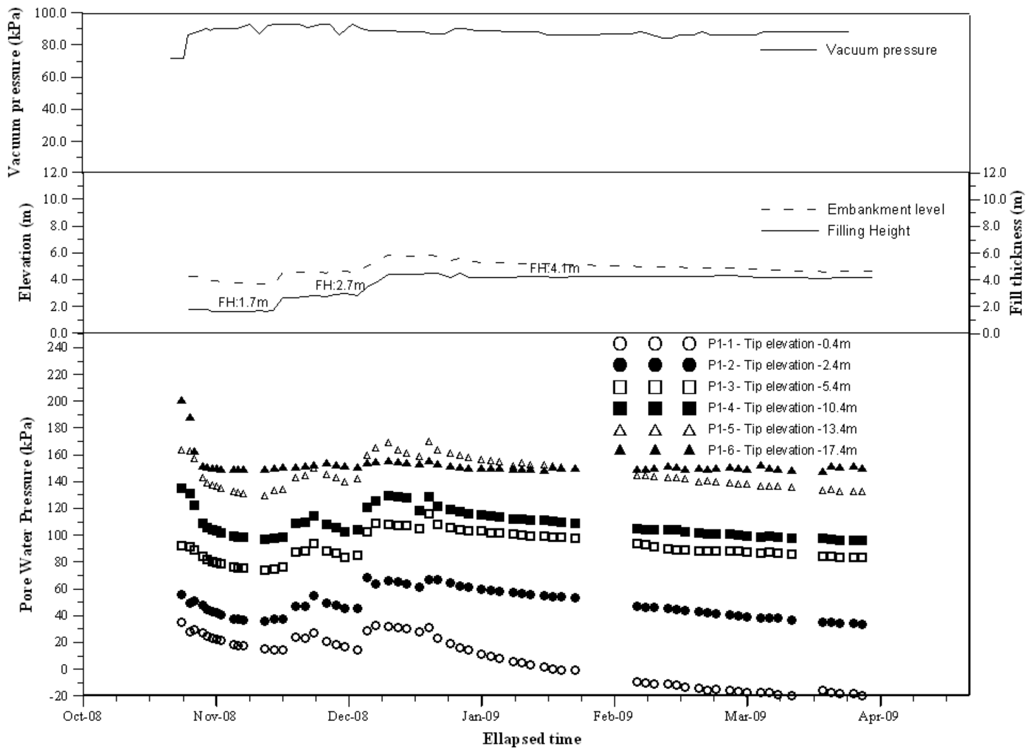


Fig. 6 Field pore-pressure measurements

seal membrane. When the vacuum pressure under seal membrane reaches or exceeds 80 kPa for 7 days, a surcharge fill with the thickness of 2.5 m to 3.5 m was placed on the non-woven geotextile. The final platform elevation was + 5.7 m CD. The design load on sand cushion consists of an 80 kPa vacuum pressure and a surcharge fill. The duration of vacuum loading had lasted for about five months. The clay bags made by woven geotextile bags filled with waste clayey soils were backfilled into the periphery wall to prevent the water overflow to surrounding area and cofferdams were installed along the borderline of every vacuum preloading zone. Cofferdam is made by woven geotextile bags filled with sand. The cofferdam is heightened with the increase of surcharge height.

Vacuum pipe system consists of main pipes of φ76 mm and filter pipes of φ50 mm, both buried in the drainage fill at 0.3 m depth. The placement spacing of filter pipe is about 6.0 m. They are PVC pipes, perforated along their length and firmly wrapped with non-woven geotextile. The main pipes were connected with the filter pipes and the vacuum pumping system by special linkers. The probe of vacuum gauge was placed at the middle of two adjacent rows of filter pipes and covered by a 0.3 m thick sand layer.

A field monitoring program was established to monitor surface and subsurface settlement, lateral displacement and pore pressure. In total, 05 settlement plates, 01 inclinometer hole, 01 extensometer hole, 01 observation well and 01 piezometer hole were installed in site tested. There are 05 settlement plates installed at + 4.1 m CD whereas SP04 was in the center of embankment and SP01, SP02, SP03, SP05 were in the boundary of embankment. The extensometer, piezometer and observation well were installed in the center of the test embankment. The extensometers were installed at about each 3 m vertical interval, e.g. plate magnet was installed at + 2.5 m CD to record surface settlement and 04 spider magnets were installed at – 2.1, – 6.1, – 8.8 and – 12.8 m CD, respectively. The piezometers were installed between the PVDs. Totally, 06 piezometers were also installed at – 0.4, – 2.4, – 5.4, – 10.4, – 13.4 and – 17.4 m CD. Inclinometers were placed along the boundary of the soft ground treatment project, the bottom of which was embedded into 3.0 m below the top of firm clay layer. The groundwater level was measured by observation well, which was placed in the center of each vacuum and surcharge combined preloading zone. The layout and detail of field instrumentations were shown in Figs. 1(b) and 4, respectively.

Settlements obtained from settlement plate and extensometer measurements were shown in Fig. 5. In this figure, field measurement data of plate magnet were fully recorded about 160 days and those by spider magnets installed at – 2.1, 6.1 m CD were recorded only for 60 days. The spider magnets, installed at – 8.8 and – 12.8 m CD, were accidentally damaged, thus they were not considered. Fig. 6 shows the pore pressure measured by piezometers. Although the rate of dissipation decreased with depth, there was a definite dissipation of excess pore pressures.

4. Calculation of consolidation settlement caused by preloading with pvd and vacuum

The settlement induced by vacuum method is somewhat similar to that caused by PVD preloading, except less lateral deformation is expected in the vacuum system. The method of estimating the rate of settlement in both systems is the same, but the surcharge of the PVD system is placed in stages to minimize any instability of the embankment (Seah 2006).

The total consolidation settlement was estimated using Terzaghi's one-dimensional (1D) consolidation theory with soil parameters given in Table 1. The primary and the time-dependent settlements are calculated using the following equation

$$S_c = H \cdot \left[RR \cdot \log \frac{\sigma'_{vm}}{\sigma'_{vo}} + CR \cdot \log \frac{\sigma'_{vo} + \Delta\sigma_v + |\Delta u|}{\sigma'_{vm}} \right] \tag{1}$$

where S_c is primary settlement, CR is compression ratio, RR is recompression ratio, σ'_{vo} is effective overburden stress, σ'_{vm} is precompression stress, $\Delta\sigma_v$ is increment loading, Δu is vacuum pressure.

For radial flow as in vertical drains, Barron (1948) proposed a solution for consolidation by radial drainage only as follows

$$U_h = 1 - \exp\left(\frac{-8T_h}{F}\right) \tag{2}$$

$$T_h = \frac{C_h t}{D_e^2} \tag{3}$$

The average degree for combined vertical and radial consolidation can be obtained by Carillo's equation (1942)

$$U = 1 - (1 - U_h)(1 - U_v) \tag{4}$$

where C_h is horizontal coefficient of consolidation, D_e is diameter of equivalent soil cylinder, U_h and U_v is the horizontal and vertical degree of consolidation, respectively.

For estimation of settlement, the analysis divided the subsoil into two layers: upper layer of weathered crust and soft clay at depth of + 2.5 m to – 6.0 m and lower layer of medium clay at depth of – 6.0 m to – 17.0 m. The settlement analysis results are shown in Table 2.

5. Back analysis of degree of consolidation

5.1 Calculation of ultimate consolidation settlement using Asaoka's Method (1978)

This method was used to predict the ultimate primary settlement. Asaoka (1978) showed that one-dimensional consolidation settlements at certain time intervals (Δt) could be described as a first order approximation

$$S_n = \beta_0 + \beta_1 . S_{n-1} \tag{5}$$

where: S_1, S_2, …, S_n are settlement observations, S_n denotes the settlement at time t_n, $\Delta t = (t_n - t_{n-1})$ is time interval. From Eq. (5) one can see that β_0 and β_1 are given by the intercept of the fitted straight line with the S_n – axis and the slope of the graph, respectively. The ultimate primary settlement is considered to be reached when $S_n = S_{n-1}$ and can be calculated by the following

$$S_{ult} = \frac{\beta_0}{1 - \beta_1} \tag{6}$$

For estimating the in-situ coefficient of consolidation, Magnan and Deroy (1980) pointed out that the in situ C_h can be estimated as follow

$$\frac{\ln \beta_1}{\Delta t} = \frac{8 c_h}{\mu D_e^2} \tag{7}$$

5.2 Calculation of ultimate consolidation settlement using the hyperbolic method (Tan and Chew 1996)

The hyperbolic method as proposed by Tan and Chew (1996) had its origins in the rectangular hyperbola fitting method proposed by Sridharan and Rao (1981), which is a method to obtain the coefficient of consolidation (C_v) from oedometer test by fitting laboratory settlement of the hyperbolic plot between the U_{60} and U_{90} points of the theoretical T_v / U_v vs. T_v plot.

In field monitoring of the consolidation of soils, plot of settlements (δ) versus time (t) are recorded. The ultimate primary settlement can be obtained from any of the three equations

$$\delta_{ult} = \frac{\alpha_i}{S_i} = \frac{\delta_{60}}{0.6} = \frac{\delta_{90}}{0.9} \tag{8}$$

Since this procedure also identifies the time for 60% and 90% consolidation (t_{60} and t_{90}) for the combined flow consolidation in the vertical drain system, it is possible to estimate the gross average in situ field consolidation coefficient C_h, assume that the value of C_v from the laboratory oedometer test on high quality undisturbed field sample is known. For the vertical drain case, U_v

will rarely to exceed 50%. Therefore at u_{60} and u_{90} for the combined flow known, U_v can be calculated from Terzaghi theory. Using Eq. (4), U_h at the time t_{60} or t_{90} can be calculated, since U = 0.6 or 0.9. The values of c_h corresponding to t_{60} or t_{90} are given as

$$C_h = \frac{-D_e^2 \mu \ln(1-U_h)}{8t} \quad \text{where} \quad t = t_{60} \text{ or } t_{90} \tag{9}$$

5.3 Determination of degree of consolidation from pore-pressure (Chu and Yan 2005)

Another possibility of assessing the degree of consolidation is based on pore water pressure measurements (Chu and Yan 2005). The average degree of consolidation at the end of preloading can be calculated as follows

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s] dz}{\int [u_0(z) + \Delta\sigma - u_s] dz} \tag{10}$$

$$u_s(z) = \gamma_w z - s \tag{11}$$

where $u_0(z)$ is initial pore water pressure at depth z; $\Delta\sigma$ is the stress increment due to surcharge at a given depth; $u_t(z)$ is pore water pressure at depth z and at time t; $u_s(z)$ is suction line; γ_w is unit weight of water; s is suction pressure applied.

The time dependent pore-pressure profile corrected due to settlement of piezometers in the site tested was shown in Fig. 7. Initially, pore-pressure increases due to surcharge loading. Then, it decreases gradually due to dissipation during consolidation and shifts to the left side of the hydrostatic water pressure line until reaches to the suction line. And, that is the time pore-pressure was fully dissipated and degree of consolidation derived is 100%.

The estimated time-dependent settlements were plotted in comparison with the field measured values as shown in Fig. 8. If it is considered that measured values at E1-1, E1-2 and SP4 are representative of upper clay, lower clay layers and total stratum. The estimated settlements using 1D consolidation theory were shown as solid lines in the time-settlement plots which are good agreement with measured values plotted as “dotted” lines for upper clay, lower clay layers and total stratum. Due to shortage of field settlement measurements in upper and lower layers, the estimated settlements used to extend the field future settlements based on the trend of the settlement curve.

The degree of consolidation of the clay layers below the test embankments was back-calculated both from measured pore-pressures based on Eq. (10) and from the measured settlements at given time divided by ultimate settlement using Eqs. (6)-(8). If the compression ratio is assumed to be constant, then the degree of consolidation can be obtained from the measured pore pressures. The corresponding values of the degree of consolidation can also be obtained from the measured settlements. The estimated degree of consolidation from various methods was shown in Table 3. The degree of consolidation obtained from analytical method is slightly less than that obtained from the field settlement measurements because the ultimate settlement calculated by analytical method is higher than that from back analysis of the measured values. Furthermore, the degree of consolidation obtained from pore pressures (U_P) is consistently less than that from settlements (U_S). These problems have also been observed by Crawford *et al*. (1992), Hansbo (1997) and Bergado *et al*. (2002). The delay in calculated degree of consolidation from pore-pressure observations

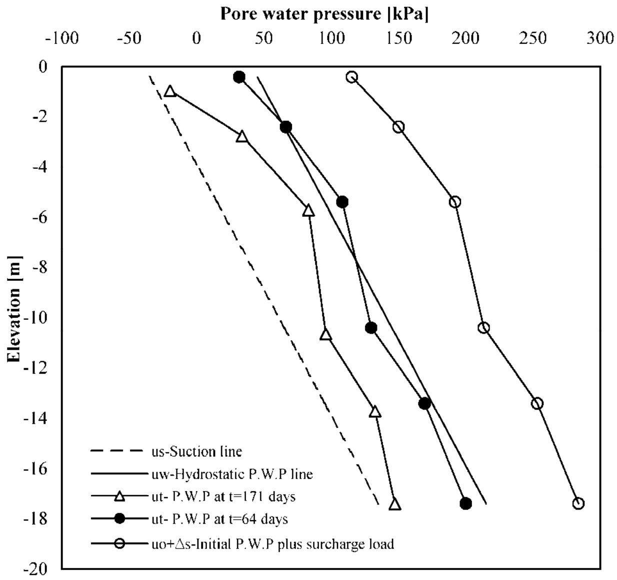


Fig. 7 Pore water pressure versus depths

Table 2 Results of consolidation analysis

Depth (m)	CR	RR	σ'_{vo} (kPa)	σ'_{vm} (kPa)	$\Delta\sigma_v$ (kPa)	σ'_{vf} (kPa)	S_c (m)
0 – 2.5	0.179	0.023	7.9	30.573	140	148	0.34
2.5 – 4.5	0.31	0.034	5	12	138	143	0.70
4.5 – 7.0	0.23	0.034	18	40	133	151	0.36
7.0 – 9.5	0.23	0.034	33	43	128	161	0.34
9.5 –12.0	0.23	0.034	48	63	123	171	0.26
12.0 – 14.5	0.23	0.034	63	83	118	181	0.21
14.5 – 19.5	0.23	0.034	86	112	113	199	0.31
						Sum	2.51

Table 3 Comparison of degree of consolidation and horizontal coefficient of consolidation C_h from settlement, pore pressure data and piezocone test from soil investigation after treatment

Average degree of consolidation	Analytical method	By field measurements		
		Settlement data		Pore pressure data
		Asaoka's method	Hyperbolic's method	Barron's method
	84%	87.2%	86.5%	74.6%
Average horizontal coefficient of consolidation C_h (m^2/yr)	By piezocone test	Asaoka's method	Hyperbolic's method	Barron's method
	1.7 – 2.1	2.1	1.9	2.0

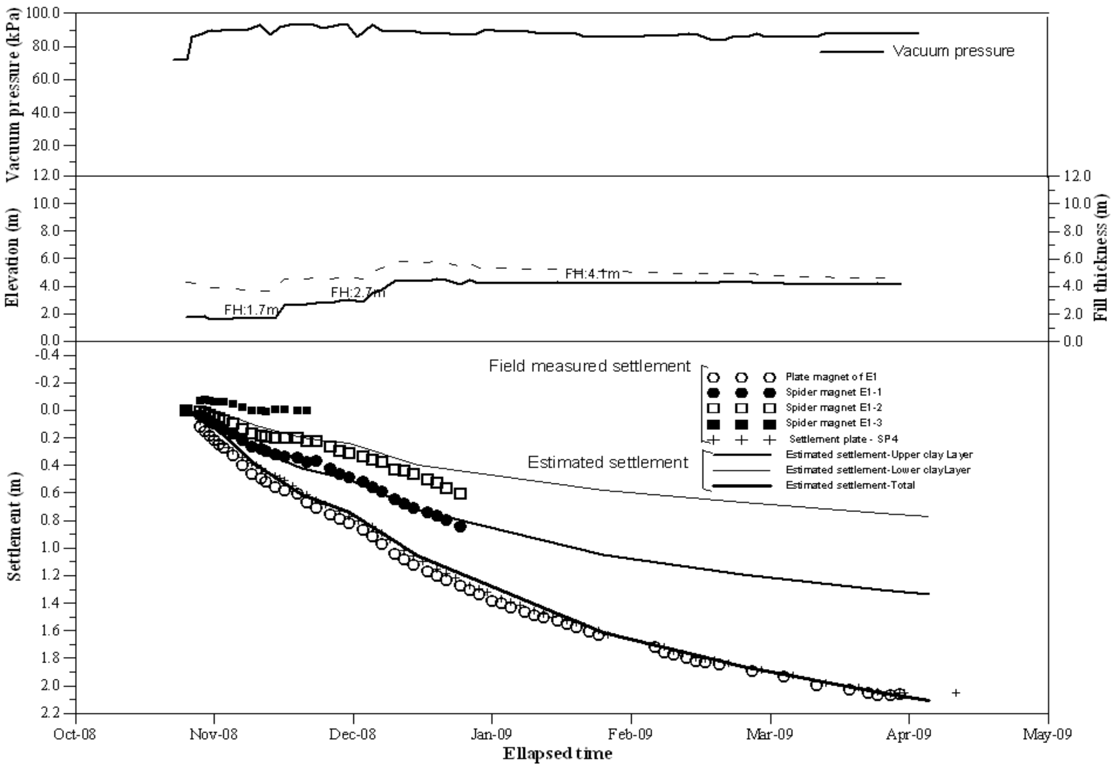


Fig. 8 Comparison of settlement between analytical results and monitoring data

obtained here is in accordance with Mikasa consolidation theory (Mikasa 1965). During the compression and rearrangement of the soil structure, the excess pore pressures were maintained at higher levels.

6. Back analysis of degree of consolidation

6.1 Reduction of water content

Changes in water content can be estimated based on field settlement data (Stamatopoulos and Kotzias 1985) as follows

$$\Delta w_n = -\left(w_n + \frac{1}{G}\right)\frac{\delta}{h} \tag{12}$$

where w_n, Δw_n are the original and change of natural water content, G is the special gravity of soil grains, δ is the settlement under preloading, and h is the thickness of compressible soils.

Fig. 9 illustrates the reduction of water content with depth for test embankment after 160 days of preloading compared with the mean values of the initial water contents. The back-calculated values of water content from settlements after treatment are also plotted in Fig. 9. The results indicate that there is good agreement between predicted and measured values.

6.2 Increase in undrained shear strength

The increase in undrained shear strength, S_u, was predicted by the SHANSEP technique (Ladd 1991) as follows

$$\left(\frac{S_u}{\sigma'_{vo}}\right)_{OC} = \left(\frac{S_u}{\sigma'_{vo}}\right)_{NC} OCR^m \tag{13}$$

where OCR is the over consolidation ratio, σ_{vo} is the effective overburden pressure, and NC and OC denote normally consolidated and overconsolidated, respectively. In this project, the SHANSEP equation can be obtained from field vane shear test, oedometer test and constant rate of strain (CRS) test as follows

$$\frac{S_u}{\sigma'_{vo}} = 0.215 * OCR^{0.805} \tag{14}$$

Changes in undrained shear strength can also be estimated from the following equations based on field settlement data (Stamatopoulos and Kotzias 1985)

$$\Delta S_u = \left(\frac{1 + w_n G}{0.434 C_c}\right) S_u \frac{\delta}{h} \tag{15}$$

where S_u, ΔS_u are the original and change of undrained shear strength, w_n, Δw_n are the original and

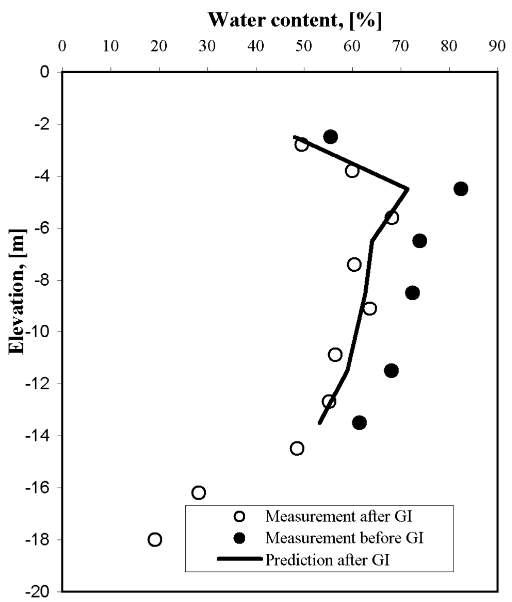
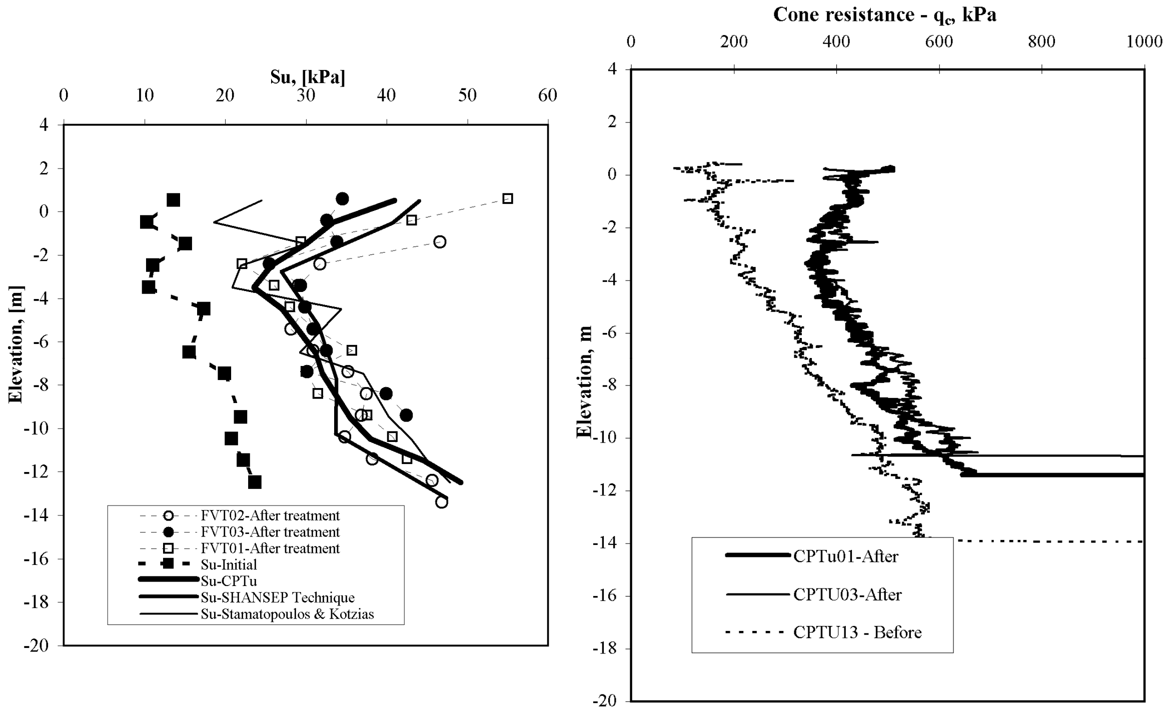


Fig. 9 Back-calculated water content from settlements



(a) S_u values before and after treatment (b) q_c values before and after treatment

Fig. 10 Comparison S_u and q_c values before and after treatment

change of natural water content, G is the special gravity of soil grains, C_c is the coefficient of compressibility, δ is the settlement under preloading, and h is the thickness of compressible soils.

The increase in undrained shear strength, S_u, was also obtained from piezocone penetration tests as follows

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \tag{16}$$

where q_t, is the corrected cone resistance; σ_{vo} is the total overburden stress, N_{kt} is the cone factor ($N_{kt} = 12$ for soft clay in this area).

The predicted undrained shear strengths can be comparable with the initial measured values and indicated by “solid lines” in Fig. 10(a). The corrected undrained shear strengths measured by field vane shear tests after treatment are also plotted in this figure by “dotted lines”. Although, at depths of 0 ~ –2 m, the predicted values are scatter, but it was clearly indicated that there is an excellent agreement between the measured and predicted values with regards to the increase in undrained shear strength due to preconsolidation and drainage. To confirm the effect of treatment, cone resistance values (q_c) measured by piezocone test before and after treatment are also plotted in Fig. 10(b). The results imply that after treatment, the q_c values increase substantially comparing with the initial values and this is consistent with the increase of undrained shear strength.

6.3 Determination of Ch based on field settlement measurements

Hyperbolic and Asaoka's method require settlement data beyond the 60% consolidation stage in order to provide accurate estimates of the ultimate primary consolidation settlement and the in situ consolidation coefficients (Tan and Chew 1996). Since C_h values obtained from field settlement measurements were mainly taken from settlement plates (SP04) and magnet plate (E01) due to sufficient data for long duration.

6.3.1 Asaoka's Method (Asaoka 1978)

The values of C_h can be back-calculated from settlement measurements using Eq. (7). The time increment used was 07 days. Tan and Chew (1996) indicated that the choice of time interval did not really matter for predicting ultimate settlement and coefficient of consolidation. As shown in Fig. 11, C_h values derived from settlement of settlement plate (SP04) and extensometer (E01) are 2.17 and 1.95 m^2/yrs, respectively.

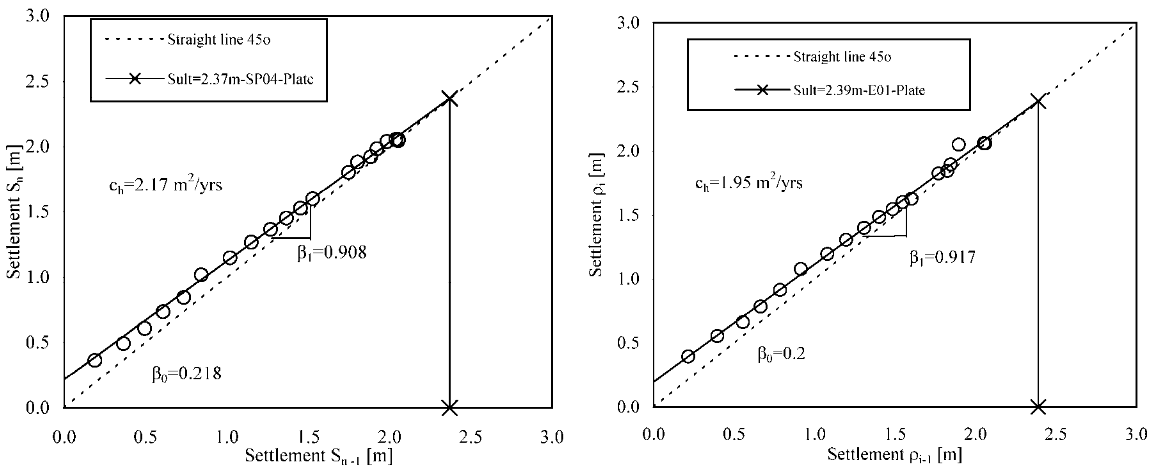


Fig. 11 Field settlement, Asaoka's plot for case study

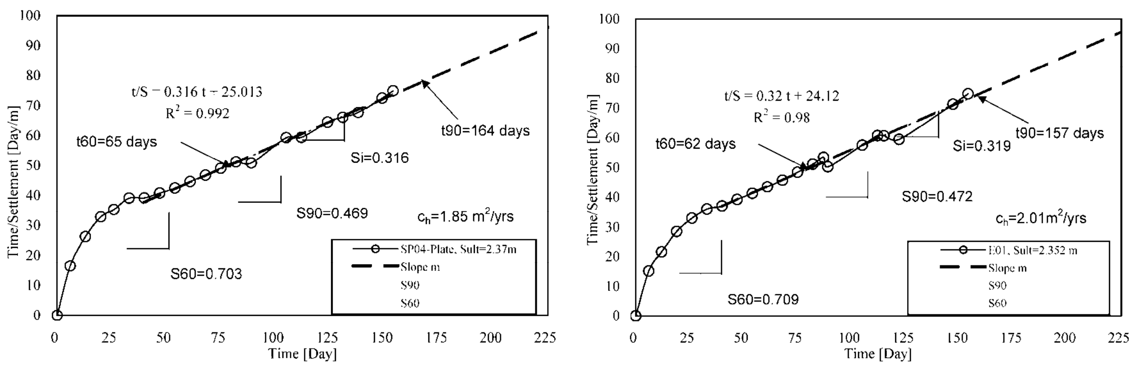


Fig. 12 Field settlement, hyperbolic's plot for case study

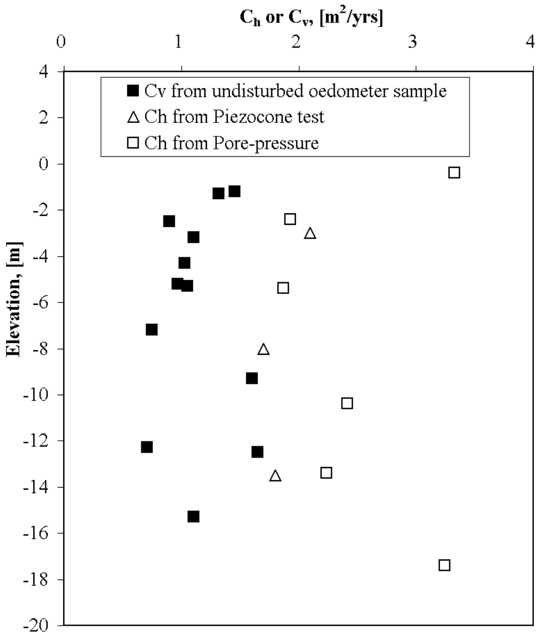


Fig. 13 Comparison of results of C_h from field measurements and C_v from oedometer test

6.3.2 Hyperbolic's Method (Tan and Chew 1996)

The values of C_h based on hyperbolic's method can be back-calculated from settlement measurements using Eq. (9). As shown in Fig. 12, C_h values derived from settlement of settlement plate (SP04) and extensometer (E01) are 1.85 and 2.01 m^2/yrs, respectively.

As the results shown that both Asaoka and hyperbolic's method gave a good predictions of ultimate primary settlement and horizontal coefficient of consolidation. A similar observation was reported earlier by Tan and Chew (1996). The agreement in prediction of the ultimate settlement and coefficient of consolidation between the two methods are well within an error of 3%.

6.4 Determination of c_h based on field pore water pressure observation

Aboshi and Monden (1963) presented a curve fitting method using log U and linear t. This method is developed by taking "log" of both sides of Barron's solution (Eq. (2)), which results in the following expression

$$T_h = \frac{-\mu \ln(1-U_h)}{8} \tag{17}$$

where T_h is time factor

$$T_h = \frac{C_h t}{D_e^2} \tag{18}$$

By combining Eqs. (17)-(18), the coefficient of radial consolidation C_h can be calculated as follows

$$C_h = \frac{-D_e^2 \mu \ln(1-U_h)}{8t} \tag{19}$$

Back-calculation of C_h values based on field settlement and pore pressure measurements were shown in Table 3. The C_h values deduced from settlement were slightly higher than those estimated from pore-pressure back-analysis. Furthermore, as seen in Fig. 13, the values of C_h derived from pore-pressure match well with those obtained from the dissipation results of piezocone test. The C_h value deduced from pore-pressure measurement at -0.4 m CD is slightly higher since it is within weathered crust clay layer having high over consolidated ratio and at level of -17.4 m CD, the C_h value is higher than 3 m^2/yr because this point is adjacent to the lower sand layer. The back-calculated $C_{h(\text{field})}$ values range from 1.7 to 2 m^2/yr as shown in Fig. 13. The corresponding laboratory values range from 1.1 to 1.2 m^2/yr. The ratio of $C_{h(\text{field})}/C_{h(\text{lab})}$ in this study is about 2 times.

7. Result discussions

The results of a test embankment applying PVD combined with vacuum preloading were investigated. Settlement analysis and back-calculation of the consolidation parameters from the field settlements and pore-pressures were done. After that, the results of back-analysis were compared with those obtained from laboratory and field ground investigation tests before and after treatment. The following conclusions can be revealed:

- The average degree of consolidation estimated from the settlement is higher than the corresponding values derived from the pore water pressure due to maintenance at higher levels of the excess pore pressures during the progress of consolidation.
- There is a good agreement between the measured and predicted increase in undrained shear strength. The prediction applied various methods. The measured water contents of the treated soil also match well with the estimated values from the consolidation settlements.
- The C_h values obtained from pore-pressures match well with those obtained from the dissipation results of piezocone test. The back-calculated $C_{h(\text{field})}$ values range from 1.7 to 2 m^2/yr, and the ratio of $C_{h(\text{field})}/C_{h(\text{lab})}$ in this study is about 2 times.

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