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Ground Improvement of Mekong Delta Soft Clay by Preloading with PVD and Vacuum at A Site in Southern Vietnam

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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 relatively new ground improvement method is more and more applied in the Mekong delta to meet the needs of fast infrastructure development in this southern part of Vietnam. 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.

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 (1952) 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 et al. 1981, Holtz et al. 1991, Bergado 1996 and Chai 1999. Recently, many successful field applications have been reported, e.g., Shang et al. (1998), Tang and Shang (2000) and Chu et al. (2000) etc. 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 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

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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

The Saigon International Terminals Vietnam (SITV) occupies an area of 33.7 hectares, consisting of a container terminal with three berths along 730 meters of quay at Thi Vai-Cai Mep area in Ba Ria-Vung Tau Province in Southern Vietnam as shown in Fig. 1. It is located approximately 75 km south of Ho Chi Minh City.

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 x 73 x 251 m in plan dimension is shown in Fig. 2. A soil investigation program, including boring, undisturbed sampling, piezocone and field vane testing was conducted to ascertain the soil properties at the site.

Plasticity chart of the soil profile is shown in Fig. 3. Most of Atterberg limit values lie above the “A” line, confirming the high plasticity of the marine soft clay. The water table is found at the ground surface. Site investigation revealed that the seabed in the area, varied between +2.0 and 2.5 m Chart Datum (CD). 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. 1 Location of the SITV site

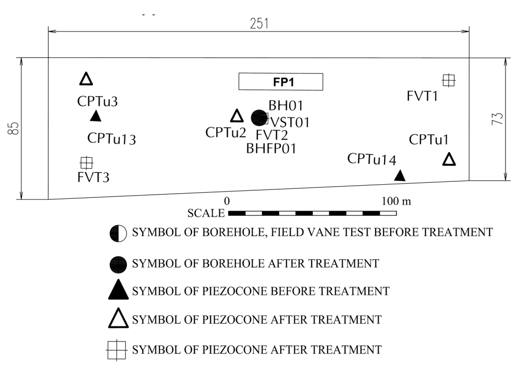


Fig. 2 Locations of boreholes

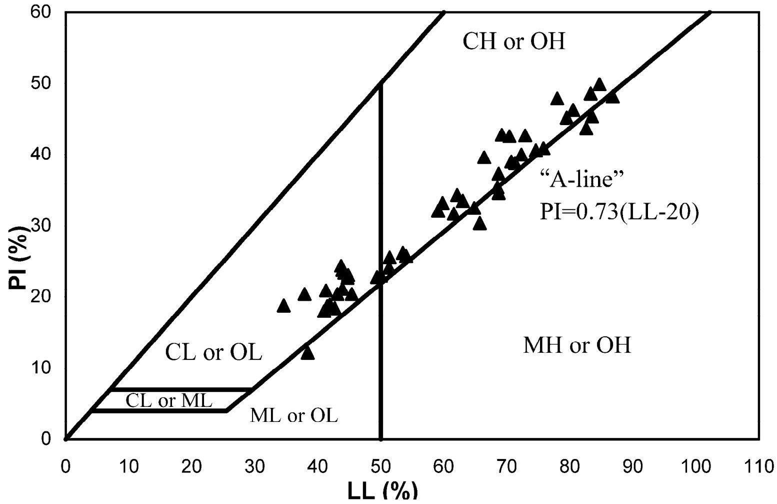


Fig. 3 Plasticity chart

Consolidation tests using oedometer and CRS cells and undrained triaxial compression tests were performed on undisturbed samples obtained from the site before treatment. Figs. 4&5 and Table 1 show soil characteristics, including the water content (w), liquid limit (LL), plastic limit (PL), preconsolidation pressure (σ'_p), compression ratio (CR), recompression ratio (RR), vertical coefficient of consolidation (C_v) and undrained shear strength (s_u) with depth. The values of σ'_p and C_v were determined from the oedometer and CRS tests using Casagrande's procedure, and the values of s_u were obtained from field vane tests.

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
w		%	67.8	79.7	70.2	60.9
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
OCR		-	5.2	2.2	1.3	1.3
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

* $C_h(NC) = (RR/CR).C_h(piezocone)$

3. CONSTRUCTION AND INSTRUMENTATION OF TEST EMBANKMENT

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 CD, then backfilled with sand at +3.5 m CD and drainage fill at +4.1 m CD, where the PVDs were installed. The final platform elevation was +5.7 m CD. The design load on sand cushion consists of a vacuum pressure of 80kPa, and a 2.5 m surcharge. The duration of vacuum preloading was about 5 months.

At the location of clay sealing wall, the clay bags were backfilled into the wall and two layers of geomembrane were placed. Geomembrane extends to two adjacent treatment zones at least by 2.0 m, and it is bonded with the geomembrane of the adjacent treatment zones by glue to ensure the seal and make sure that the soil in clay sealing wall is improved by vacuum pressure simultaneously.

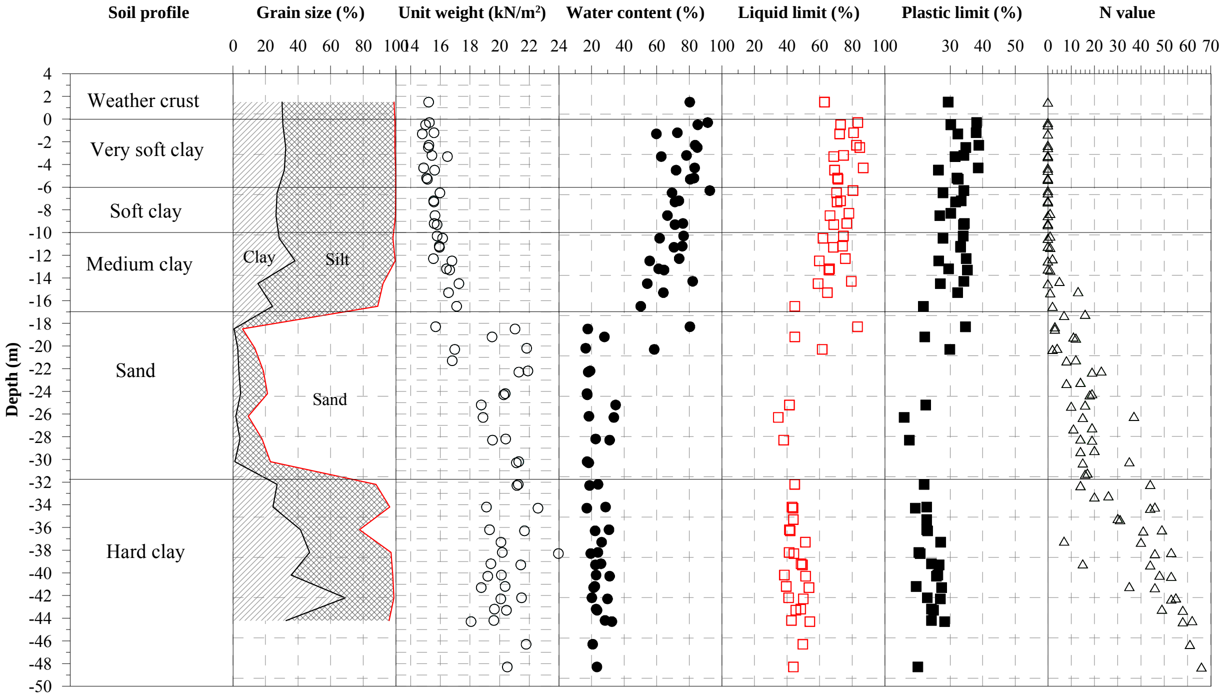


Fig. 4 Physical properties at the SITV site

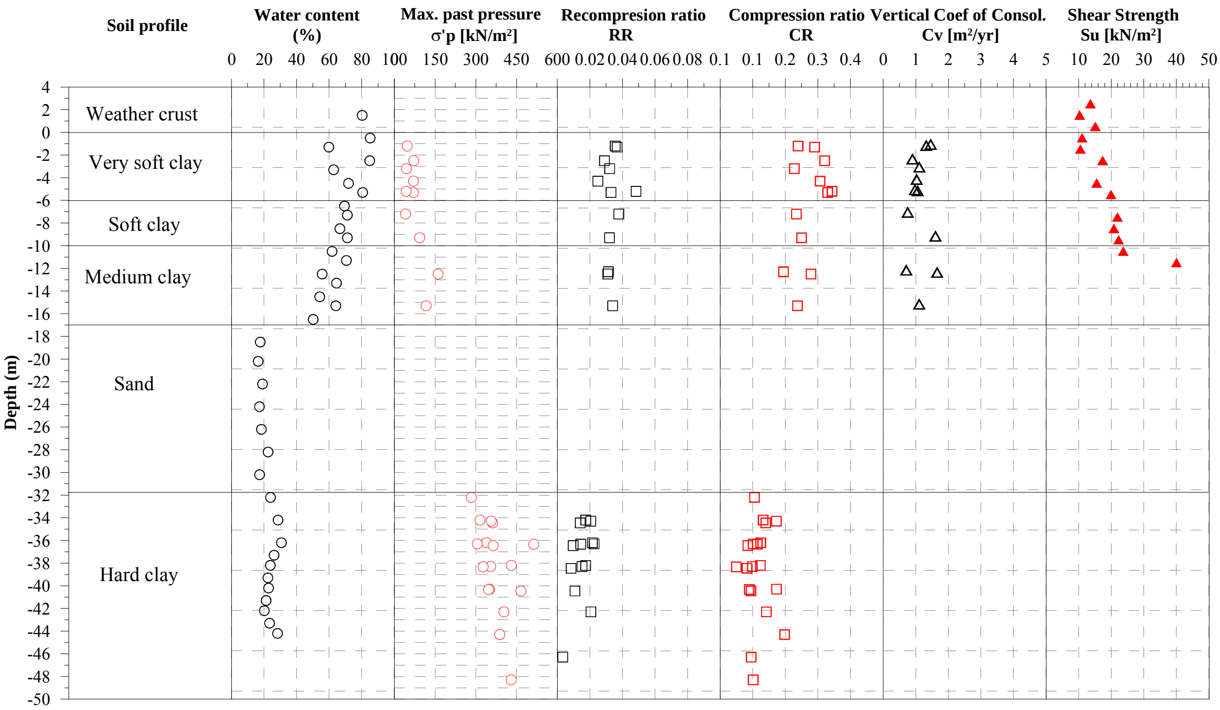


Fig. 5 Consolidation test and strength properties at the SITV site

After the vacuum pressure under geomembrane reached 80kPa in the treatment area, cofferdams were constructed along the borderline of every vacuum preloading zone. Cofferdam is made by woven geotextile bags filled with sand. Cofferdam section is rectangular and its construction is divided into two stages. The cofferdam is heightened with the increase of surcharge height.

A field monitoring program was setup to monitor surface and subsurface settlements, lateral movements, and pore pressures. Several settlement plates, one inclinometer, and piezometers were installed. The extensometers were installed in the center of the test embankment. The extensometers and the piezometers were installed at every 3 m vertical interval. The piezometers were installed between the PVDs. Inclinometers were placed along the boundary of the soft ground treatment project, the bottom of which was embedded 3.0 m bellow the top of firm ground during consolidation of foundation. The groundwater level was measured by observation well, which was placed in the center of each vacuum and surcharge combined preloading zone. The instrumentation plan was shown in Fig. 6.

Views of section of instrumentation are shown in Figs. 7a&b. PVDs were installed at depth from 16 to 20 m on a triangular pattern with 1.2 m spacing. The size of PVD is 100 mm length and 5 mm width. The mandrel was rectangular in cross section with a thickness of 6 mm and outside dimensions of 150 mm by 45 mm. Rectangular-shaped anchoring shoes with dimensions of 150 mm by 45 mm were utilized. Construction commenced in October 2008 and was completed 6 months later.

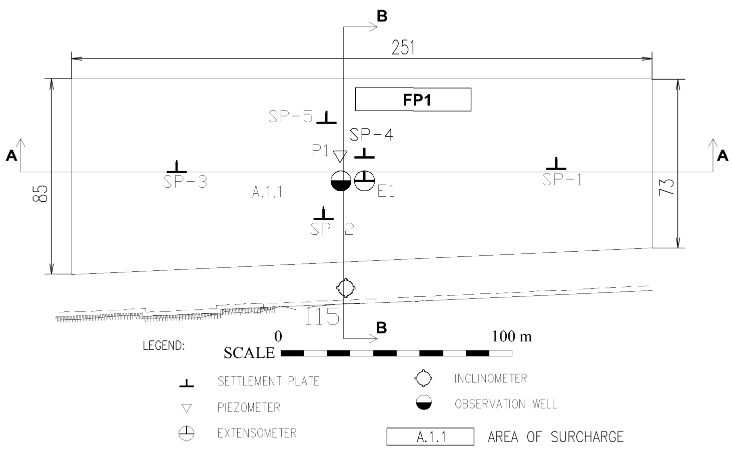
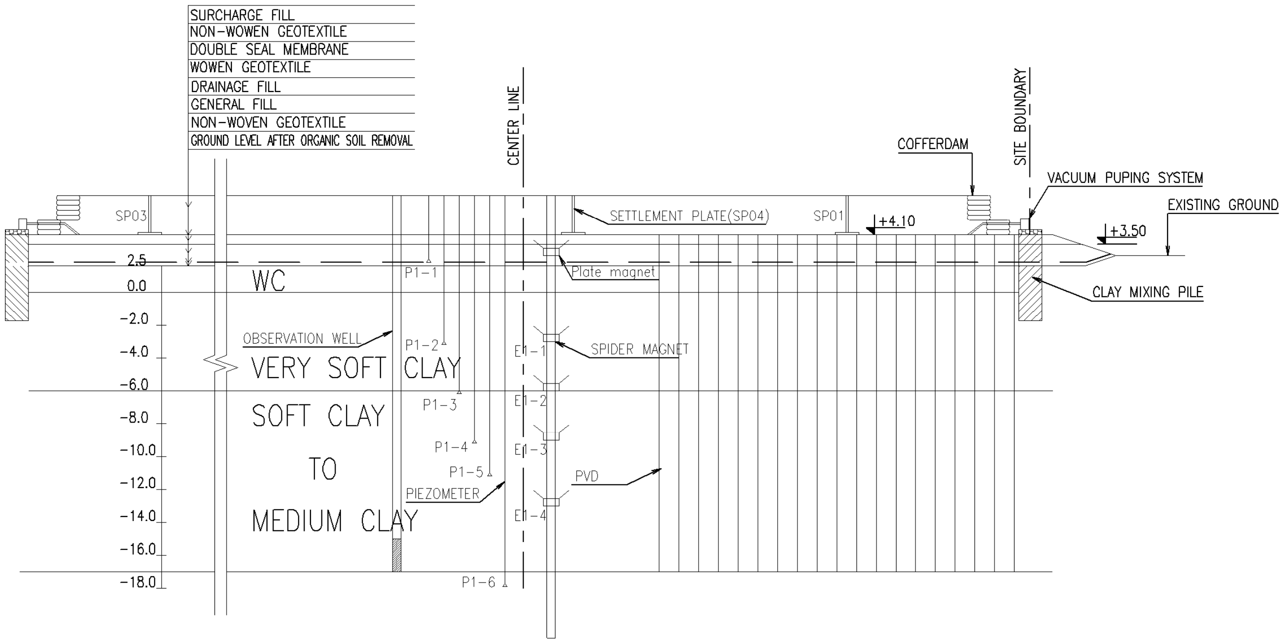
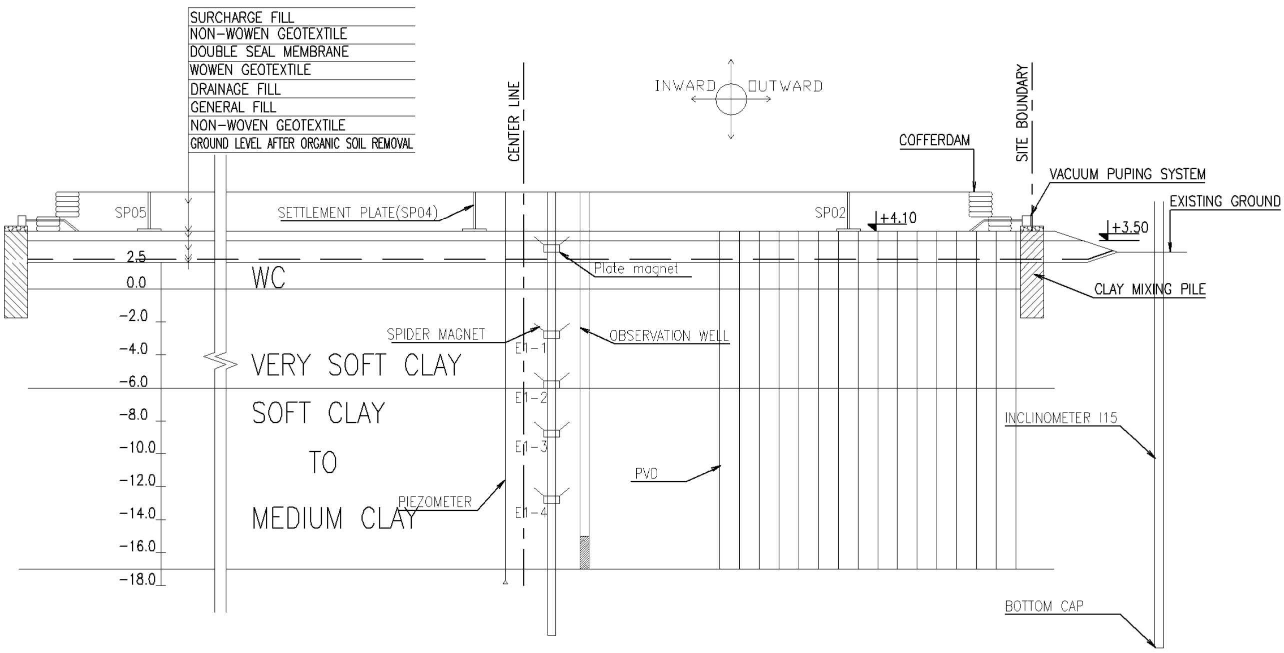


Fig. 6 Layout of installation of field instrumentations

As seen in Figs. 7a&b, 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. Extensometers were installed at various depths, e.g. plate magnet was installed at +3.44 m CD to record surface settlement and 04 spider magnets were installed at -2.05, -6.05, -8.76 and -12.73 m CD, respectively. To record pore-pressure, 06 piezometers were also installed at -0.4, -2.4, -5.4, -10.4 and -17.4 m CD. Settlements and pore-pressure obtained from field measurements observation were shown in Figs. 8&9, respectively. As shown in Fig. 8, field measurement data of plate magnet were fully recorded about 160 days and spider magnets installed at -2.05, 6.05 m CD were partly recorded for 60 days. The spider magnets installed at -8.76 and -12.73 m CD were damaged due to accidental reason. Thus they were not used in comparing with the estimated settlements.



(a) Section A-A



(b) Section B-B

Fig. 7 Views of instrumentation sections

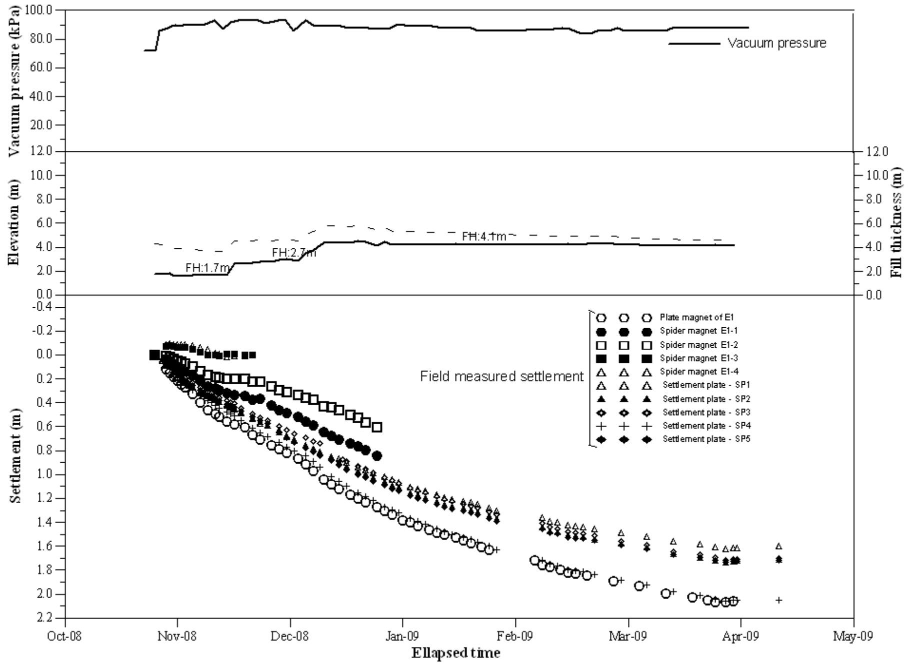


Fig. 8 Field settlement measurements

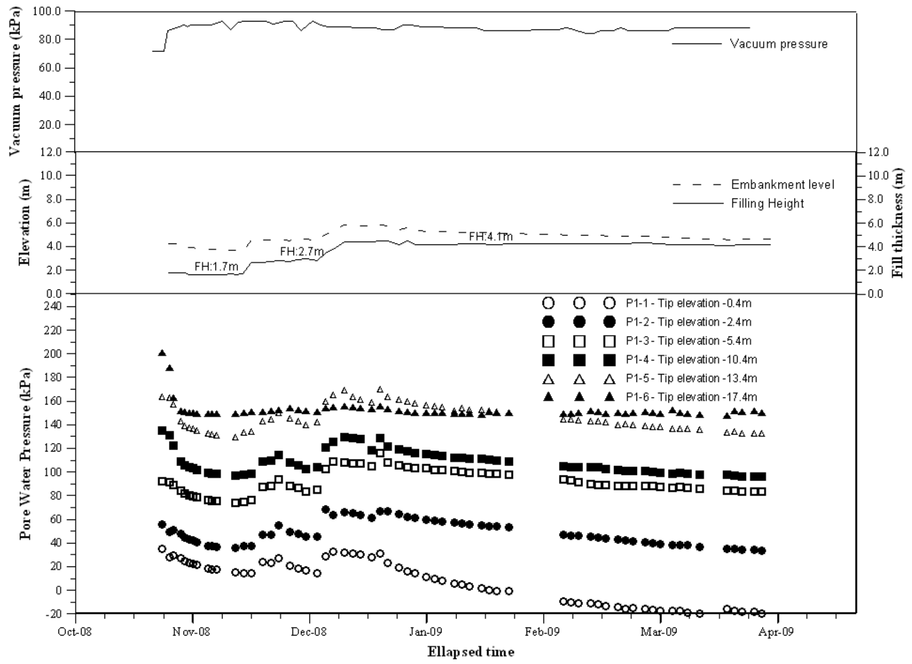


Fig. 9 Field pore-pressure measurements

4. CALCULATION OF CONSOLIDATION SETTLMENT 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:

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

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

C_h = horizontal coefficient of consolidation
D_e = diameter of equivalent soil cylinder

Hansbo (1979) suggested that the factor (F) consists of the following the component with consideration of the effect of smear zone and well resistance:

$$F = F(n) + Fs + Fr \quad \text{where} \quad F(n) = \ln \frac{d_e}{d_w} - 0.75 \tag{4}$$

d_w = equivalent diameter of the drain

$$Fs = \left[\frac{k_h}{k_s} - 1 \right] \ln \frac{d_s}{d_w} \tag{5}$$

(k_h/k_s ~ 2 from consolidation test and piezocone test data, and d_s/d_w=2 based on recommendation by Hansbo (1979).

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

$$U = 1 - (1 - U_h)(1 - U_v) \tag{6}$$

where 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 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:

$$S_n = \beta_0 + \beta_1 . S_{n-1} \quad (7)$$

where: S_1, S_2, …, S_n are settlements observations, S_n denotes the settlement at time t_n, $\Delta t = (t_n - t_{n-1})$ is time interval. The first order approximation should represent a straight line on a (S_n vs. S_{n-1})-co-ordinate. From Eq. (7) 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:

$$S_{ult} = \frac{\beta_0}{1-\beta_1} \quad (8)$$

S_{ult} is the very intersection between the S_n-S_{n-1} graph and the 45°-line (because $S_n=S_{n-1}$) as shown in Fig. 10.

In case of staged construction and when a large increment of surcharge load is applied, there is normally an obvious increase in the gradient of the settlement-time curve. In order to determine the ultimate settlement under these conditions, data obtained from the final stage of loading should be used. 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:

$$\frac{\ln \beta_1}{\Delta t} = \frac{8c_h}{\mu D_e^2} \quad (9)$$

where, D_e is diameter of an equivalent soil cylinder, S is drain spacing, μ is factor for the effect of drain spacing

$$\mu = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2} \quad (10)$$

where, n is drain spacing ratio n= D_e /d, d is equivalent diameter of prefabricated drain, Δt is time increment.

5.2. Calculation of ultimate consolidation settlement using the hyperbolic method (Tan, 1996)

The hyperbolic method as proposed by Tan (1996) had its origins in the rectangular hyperbola fitting method proposed by Sridharan et al (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. When hypothetical settlement data is plotted in a form of t/δ vs. t as in Fig. 11, that is time/settlement vs. time, the same features as the theoretical plotted observed, namely an initial concave segment, follow by a linear segment between the 60% and 90% consolidation state. The ultimate primary settlement can be obtained from any of the three equations:

$$\delta_{ult} = \frac{\alpha_i}{S_i} = \frac{\delta_{60}}{0.6} = \frac{\delta_{90}}{0.9} \tag{11}$$

The proposed method is based on theory with instantaneous loading. Since the method attempt to fit field settlement data beyond the 60% consolidation stage, it is not sensitive to the nature of the initial loading condition as long as the loading period is small compared to the time required for 60% consolidation to occur (Tan, 1996).

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. (6), 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:

$$C_h = \frac{-D_e^2 \mu \ln(1-U_h)}{8t} \qquad \text{where } t = t_{60} \text{ or } t_{90} \tag{12}$$

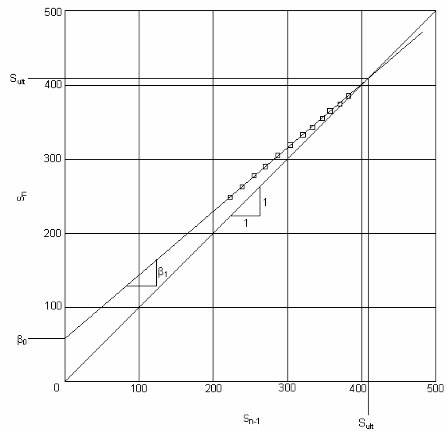


Fig. 10 Graphical illustration of Asaoka's method (Asaoka, 1978)

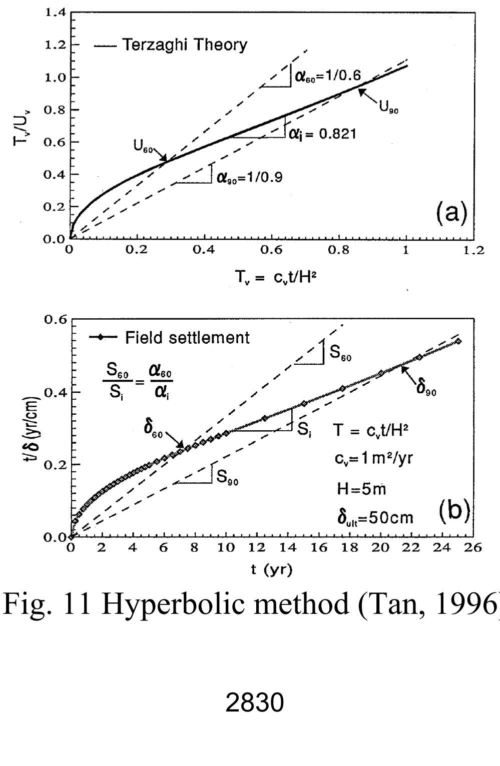


Fig. 11 Hyperbolic method (Tan, 1996)

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). As shown in Fig. 12, where a combined fill surcharge and vacuum load is considered. The average degree of consolidation at the end of preloading can be calculated as follows:

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s] dz}{\int [u_0(z) + \Delta\sigma - u_s] dz} \tag{13}$$

$$u_s(z) = \gamma_w z - s \tag{14}$$

where $u_0(z)$ is initial pore water pressure at depth z; Δσ 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 applied.

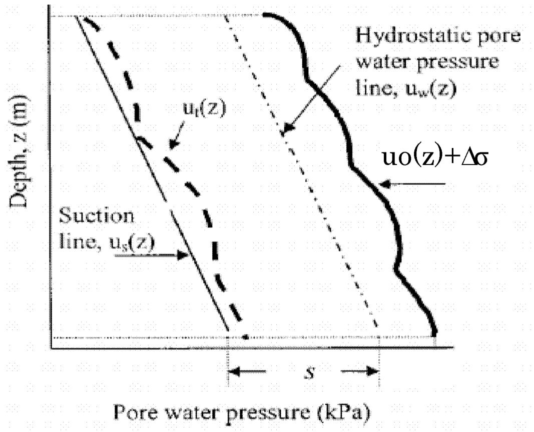


Fig. 12 Pore water pressure distribution under combined surcharge and vacuum preloading (after Chu & Yan, 2005)

As seen in Fig. 13, 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 time dependent pore-pressure profile shown in Fig. 13 was corrected due to settlement of piezometers.

The estimated time-dependent settlements were plotted in comparison with the field measured values as shown in Fig. 14. 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. (13) and from the measured settlements at given time divided by ultimate settlement using Eq. (8)&(11). 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. Table 3 compares the calculated degrees of consolidation.

As 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). A similar observation was reported earlier by Holtz and Broms (1972). These problems have also been observed by Crawford et al, Hansbo (1997), Bo (1999) and Bergado (2002). The delay in calculated degree of consolidation from pore-pressure observations 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.

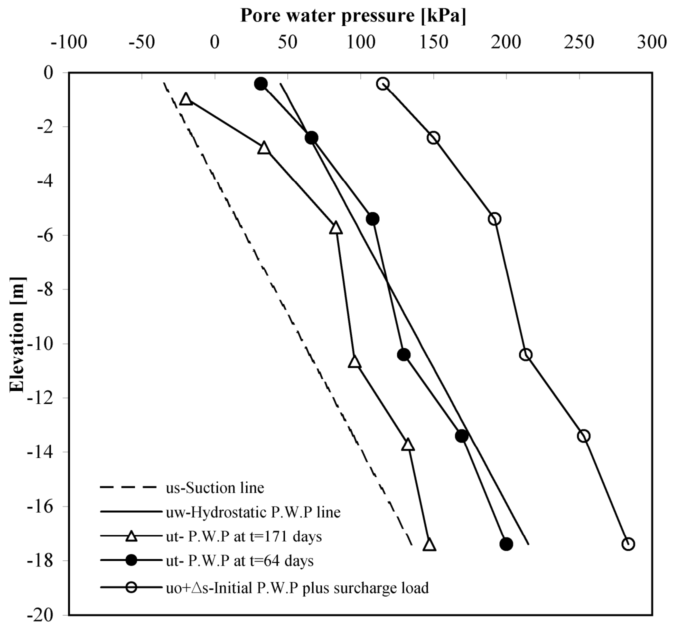


Fig. 13 Pore water pressure versus depths

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

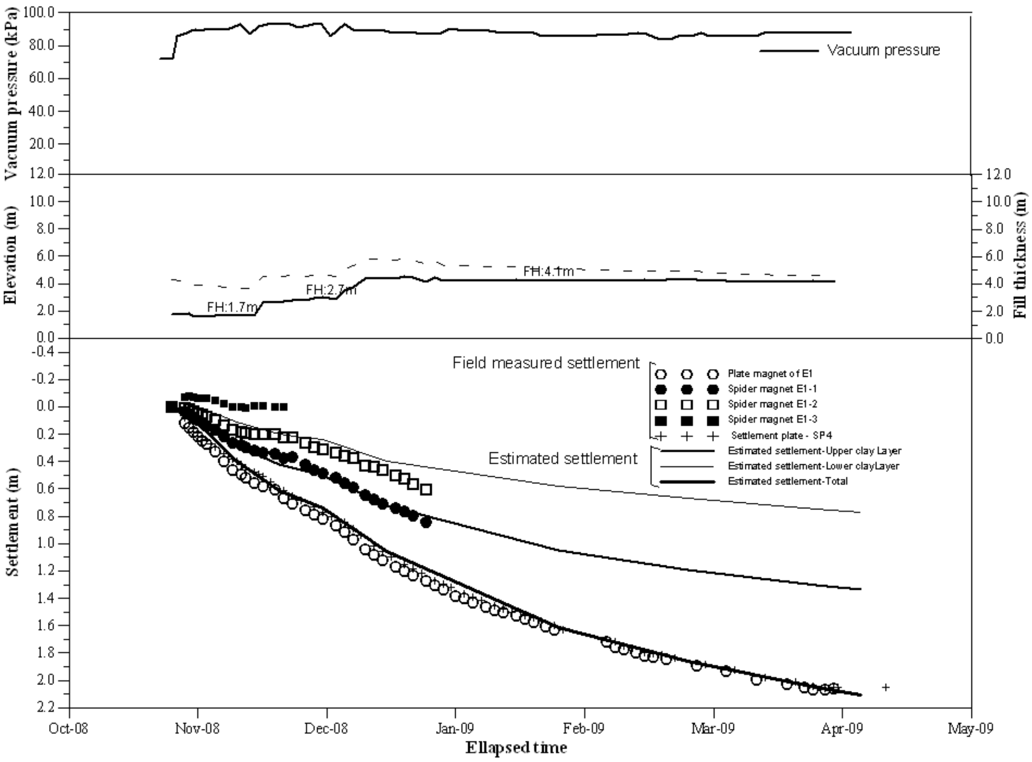


Fig. 14 Comparison of settlement between analytical results and monitoring data

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:

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

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. 15 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. 15 for test embankment and are good agreement with the measured water content data.

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:

$$\left(\frac{S_u}{\sigma'_{vo}} \right)_{OC} = \left(\frac{S_u}{\sigma'_{vo}} \right)_{NC} OCR^m \tag{16}$$

where OCR is the over consolidation ratio; σ_{vo} is the effective overburden pressure; and NC and OC denote normally consolidated and overconsolidated, respectively.

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

$$\Delta S_u = \left(\frac{1 + w_n G}{0.434 C_c} \right) S_u \frac{\delta}{h} \tag{17}$$

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

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \tag{18}$$

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 increase of shear strength can be estimated from the SHANSEP technique (Eq. 19). 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:

$$\frac{S_u}{\sigma'_{vo}} = 0.215 * OCR^{0.805} \tag{19}$$

The predicted increases in undrained shear strengths are indicated by "solid lines" in Fig. 16. The corrected undrained shear strengths measured by field vane shear tests before and after treatment are also plotted by "dotted lines". As seen in Fig. 16, there is an excellent agreement between the measured and predicted data with regards to the increase in undrained shear strength due to preconsolidation and drainage. At depths of 0 ~ -2 m, the predicted shear strength from field settlement data using Eq. (17) does not agree well with field measurements. Furthermore, cone resistance, q_c measured by piezocone tests in before and after treatment are also plotted for comparison (Fig. 17). The results indicates that the shear strength and the cone resistance increase with 75% and from 34% (0 ~ -2 m) to 72% (-2 ~ -12 m), respectively.

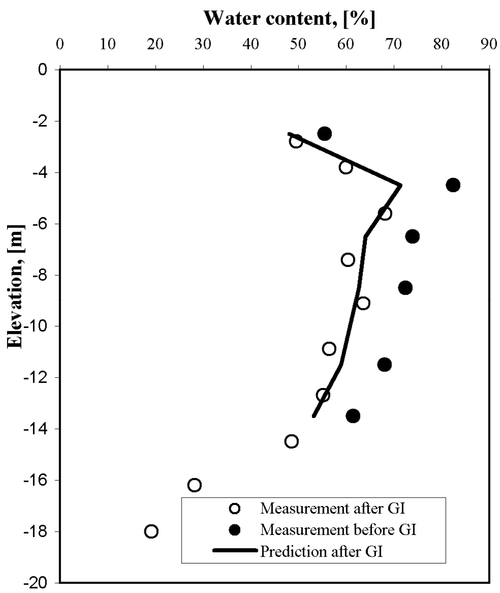


Fig. 15 Back-calculated water content from settlements

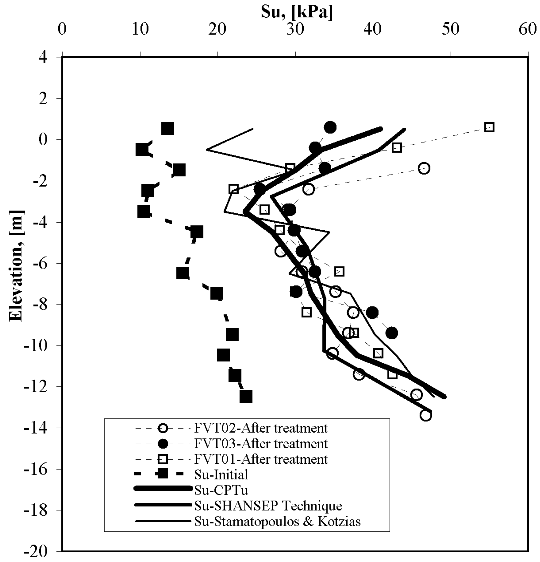


Fig. 16 Undrained shear strength before and after treatment

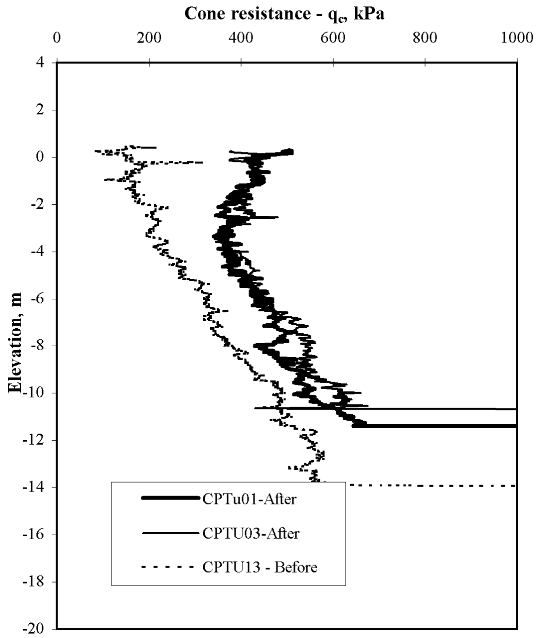


Fig. 17 Cone resistance before and after treatment

6.3. Determination of c_h 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, 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.

Asaoka's Method (Asaoka, 1978) The value of c_h can be back-calculated from settlement measurements using Eq. 9. 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. 18, c_h values derived from settlement of settlement plate (SP04) and extensometer (E01) are 2.17 and 1.95 m^2/yrs, respectively.

Hyperbolic's Method (Tan, 1996) The value of c_h based on hyperbolic's method can be back-calculated from settlement measurements using Eq. (12). As shown in Fig. 19, 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%.

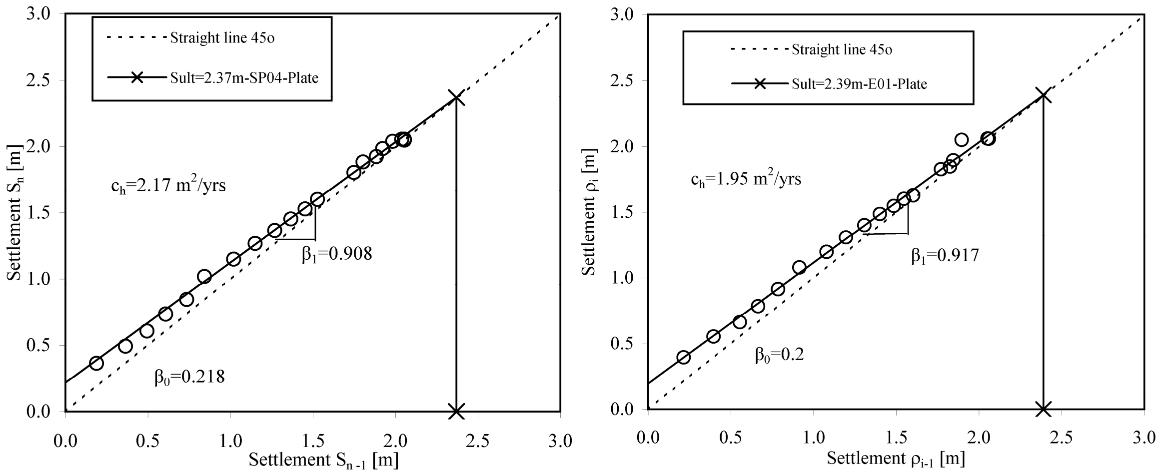


Fig. 18 Field settlement, Asaoka's plot for case study

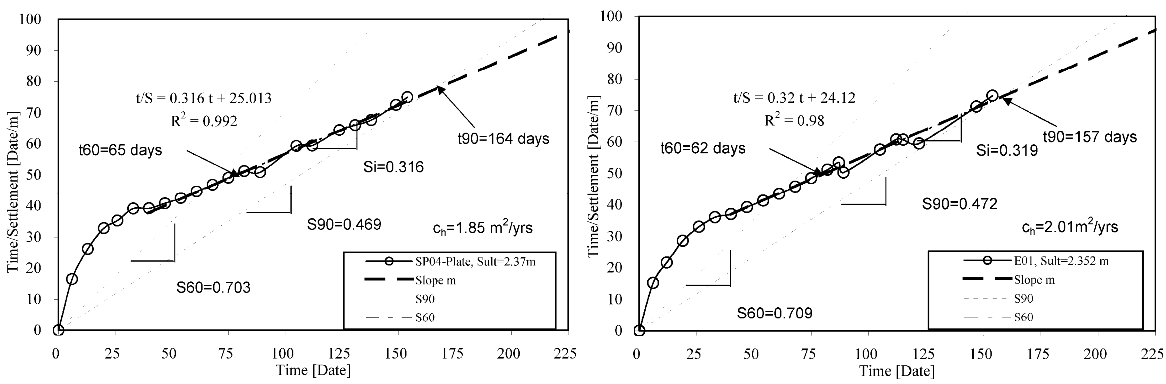


Fig. 19 Field settlement, hyperbolic's plot for case study

6.4. Determination of c_h based on field pore water pressure observation

Aboshi and Monden (1963) presented a curve fitting method using logU 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:

$$T_h = \frac{-\mu \ln(1-U_h)}{8} \tag{20}$$

where T_h is time factor:

$$T_h = \frac{C_h t}{D_e^2} \tag{21}$$

By combining Eq. (20)&(21), the coefficient of radial consolidation C_h can be calculated as follows:

$$C_h = \frac{-D_e^2 \mu \ln(1-U_h)}{8t} \tag{22}$$

Back-calculation of c_h values based on field settlements and pore pressure measurements were shown in Table 3. As seen in Table 3, the c_h values deduced from settlement were slightly higher than those estimated from pore-pressure back-analysis.

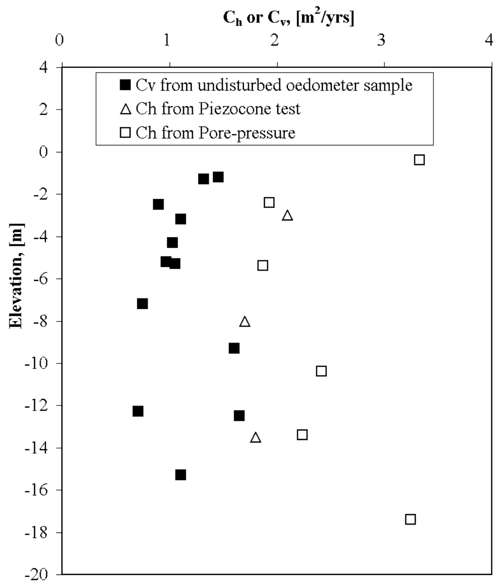


Fig. 20 Comparison of results of coefficient of consolidation from field measurements and laboratory oedometer test

Furthermore, as seen in Fig. 20, the values of c_h derived from pore-pressure is a good agreement with those obtained from the corrected result of dissipation test of piezocone test based on Baligh and Levadoux's equation (1986). The c_h values deduced from pore-pressure measurement at -0.4 mCD is slightly higher since it is within weathered crust clay layer that has high over consolidated ratio. This implies that the c_h values decreased consistently with the increase in effective stress (with the progress of consolidation). In addition, the c_h values is over 3 m^2/yr at level of -17.4 mCD due to adjacent to the lower sand layer.

RESULT DISCUSSIONS

Based on the results of calculation of consolidation settlement and back-analysis of soil properties such as reduction of water content, increase of undrained shear strength and horizontal coefficient of consolidation, some of conclusions can be drawn as follows:

The ultimate settlement was predicted using Asaoka's method (Asaoka, 1978) and hyperbolic's method (Tan, 1996), while the time-dependent settlements were estimated based on Barron's solution (Barron, 1948). The estimated and measured settlements are within an error of 3 ~ 6%, which are considered to be acceptable.

The average degree of consolidation was assessed based on both settlement and pore pressure data. The results indicated that the average degree of consolidation estimated from the settlement data was higher than that estimated from the pore water pressure data 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 undrained shear strength profiles after preloading based on SHANSEP technique and results from the piezocone penetration tests. The results indicates that the shear strength increase approximately 75% after treatment.

The measured water contents of the treated soil after preloading agreed well with values computed from the consolidation settlements. The results indicated that the reduction of water content is about 13%

The c_h values deduced from field settlements were slightly higher than those estimated from field pore-pressures back-analysis. The c_h values derived from field pore-pressure measurements decreased consistently with the increase in effective stress.

There is a good agreement between the horizontal coefficient of consolidation back-calculated from field pore-pressures and from results of ground investigation.

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