SETTLEMENT CALCULATION AND BACK-ANALYSIS OF SOIL PROPERTIES FOR A TEST EMBANKMENT ON A SOFT CLAY GROUND IMPROVED BY PVD AND VACUUM-ASSISTED PRELOADING AT A SITE IN VUNG TAU, VIETNAM

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ABSTRACT: Application of vacuum assisted preloading is helpful when a considerable load is required to meet the desired rate of settlement and an increase in the undrained shear strength in a relative short time. To facilitate the vacuum propagation, vertical drains are usually employed in conjunction. The installation of vertical drains using a steel mandrel creates significant remoulding of the subsoil surrounding the drains thereby, reducing soil permeability and adversely affecting the soil consolidation process. In this research study, performance of a test embankment on a soft clay ground improved by vacuum combined with PVD and surcharge preloading at the site of Saigon International Terminals Vietnam (SITV) in Ba Ria-Vung Tau Province is presented and analyzed. The calculated settlement results are compared with the available observation data. Besides, soil parameters were back– calculated and compared with those obtained from soil investigation.

Keywords: soil improvement, vacuum-assisted preloading, negative pore pressure, surcharge load, SITV project.

INTRODUCTION

The prefabricated vertical drain preloading with embankment was modified by combining with vacuum pressure to decrease the associated instability. Vacuum consolidation preloads the soil by reducing the pore pressure while maintaining constant total stress instead of increasing the total stress. The effective stress is increased due to the reduced pore pressure in the soil mass. The net effect is an additional surcharge ensuring early attainment of the required settlement and an increased shear strength resulting in increased embankment stability. Hence, the vacuum preloading technique can diminish large quantity of fill material as well as minimize instability problem.

The SITV Terminal occupies an area of 33.7 hectares, which consists of a container terminal with three berths along 730 metres of quay at Thi Vai-Cai Mep area in Ba Ria-Vung Tau Province in Southern Vietnam. The Saigon International Terminals Vietnam (SITV) is located approximately 75 km from the Ho Chi Minh City.

The full-scale field test confirmed the effectiveness of the prediction and monitoring methods, such as the comparison of the settlement and pore water pressure between predicted values and measured values, comparison of the degree of consolidation using pore-pressure measurement versus settlement measurement, comparison of the actual water content reduction with computed values, and comparison of the actual increase in shear strength with predicted values. The full-scale test embankments was constructed in stages on a subsoil improved by PVDs combined with vacuum surcharge preloading at the SITV project site, Southern of Viet Nam. PVDs were installed to a depth of 16 ~ 20 m, at spacing of 1.2 m, in a triangular pattern.

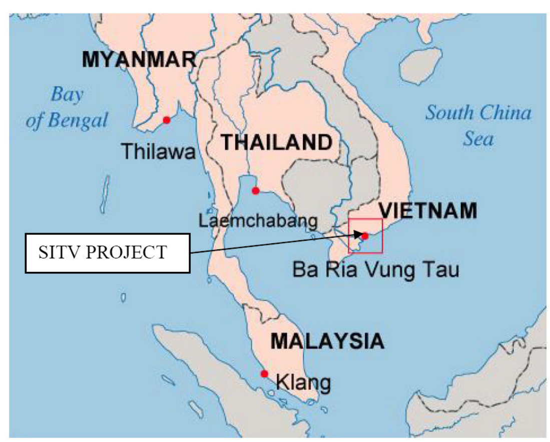

Figure 1 Location map

SOIL CONDITIONS AND PROPERTIES

Plasticity chart of the soil profile is shown in Fig. 2. Most of Atterberg limit values lie above the "A" line in the plasticity chart, confirming the high plasticity of the marine soft clay. The groundwater table is at the ground surface.

The generalized soil profile and soil properties are shown in Fig. 3. The soil profile is relatively uniform, consisting of a 2 m thick weathered crust overlying very soft to soft clay approximately 10 m thick. Underlying the soft clay is a medium clay layer about 7 m thick followed by a sand layer which is in turn 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 of 0 m and -17m. The profiles of soil strength and compressibility parameters determined by laboratory and field tests are also shown in Fig. 4.

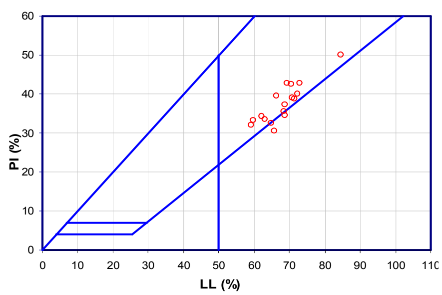

Figure 2 Plasticity chart

TEST EMBANKMENT CONSTRUCTION AND INSTRUMENTATION

The test embankment of trapezoidal shape has the size of 85 x 73 x 251m in plan dimensions and a final height of 4.1m (Fig. 5a). In the area of the test embankment, the original ground was cleared of grass roots and excavated to 0.5m below mean sea level. Organic soil was removed at +2.5m Chart Datum, then backfilled sand at +3.5m CD and drainage fill at +4.1m CD, where the PVDs were installed. The final platform elevation was +5.7m CD. The design load on sand cushion consist of a vacuum pressure of 80kPa, and a height of surcharge of 2.5m. The duration of vacuum preloading was about 4 months.

At the location of shared clay sealing wall, the clay bags were backfilled into the wall and two layer of geomembrane were placed. Geomembrane extends to two adjacent treatment zones not less than 2.0m, and it is bonded with the geomembrane of the adjacent treatment zones by glue to ensure the seal effect in treatment area and make sure that the soil in clay sealing wall is improved by vacuum pressure simultaneously.

After the vacuum pressure of 80kPa under geomembrane is achieved in the treatment area, cofferdam was 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 established to monitor surface and subsurface settlements, lateral movements, and excess 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 are placed along the boundary of the soft ground treatment project, the bottom of which was embedded 3.0m bellow the top of firm ground during consolidation of foundation. The groundwater level is measured by observation well, which is placed in the center of each vacuum and surcharge combined preloading zone. The plan showing the embankment instrumentation is plotted in Fig. 5b.

A slope indicator, labeled I15 was installed in the boundary of test embankment. Views of section of instrumentation are shown in Fig. 6 and Fig. 7. The PVDs were installed to a depth of 16 ~ 20m on a triangular pattern with 1.2m spacing. The size of PVD is 100mm length and 5mm width. The mandrel was retangular in cross section with a thickness of 6mm and outside dimensions of 150mm by 45mm. Rectangular-shaped anchoring shoes with dimensions of 150mm by 45mm were utilized. Construction commenced in October 2008 and was completed 6 months later.

ESTMATION OF SETTLEMENTS

Asaoka's Method (Asaoka, 1978)

The Asaoka's method (Asaoka, 1978) is based on settlement observation, in which earlier observations are used to predict the ultimate primary settlement. Asaoka showed that one-dimensional consolidation settlements at certain time intervals (Δt) could be described as a first order approximation:

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

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

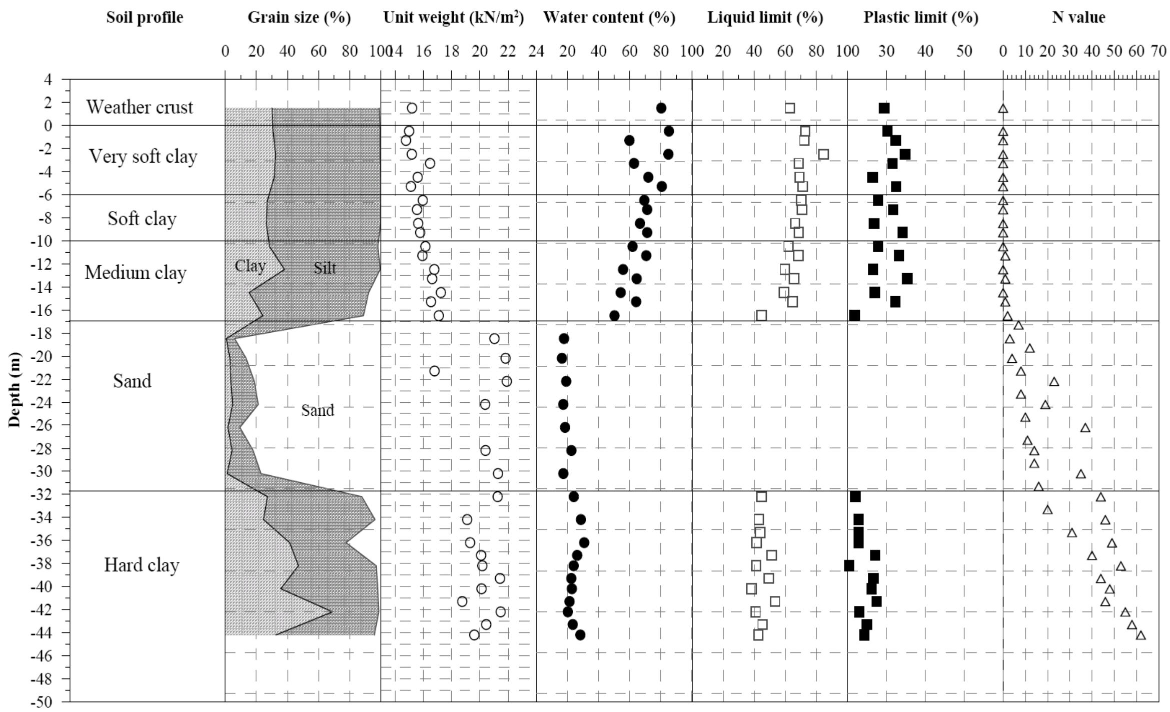

Figure 3 Physical properties

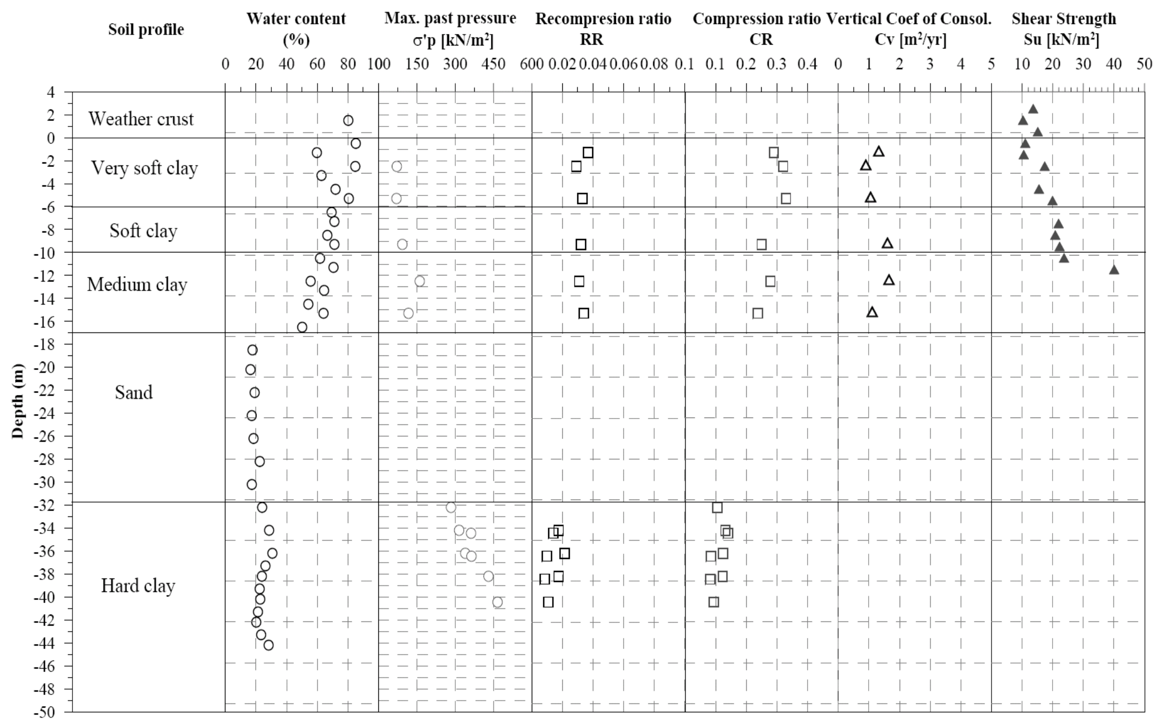

Figure 4 Summary of consolidation test results and strength profile

Table 1 Summary of soil parameter used in calculation of settlements

Physical & mechanical properties		Units	Layer			
			Weathered crust	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	0.28
RR		-	0.023	0.034	0.034	0.032
OCR		-	5.2	2.2	1.3	1.3
C_v90	OC	m^2/yr	5.5	5.9	5.7	5.7
	NC	m^2/yr	2.3	1.1	1.1	1.2
C_h	OC	m^2/yr	25.6	11.3	8.9	8.2
	NC	m^2/yr	10.6	2.1	1.7	1.8

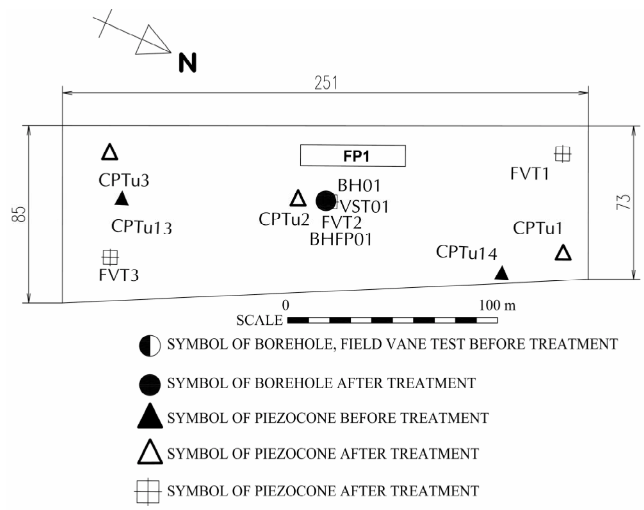

Figure 5a Locations of boreholes

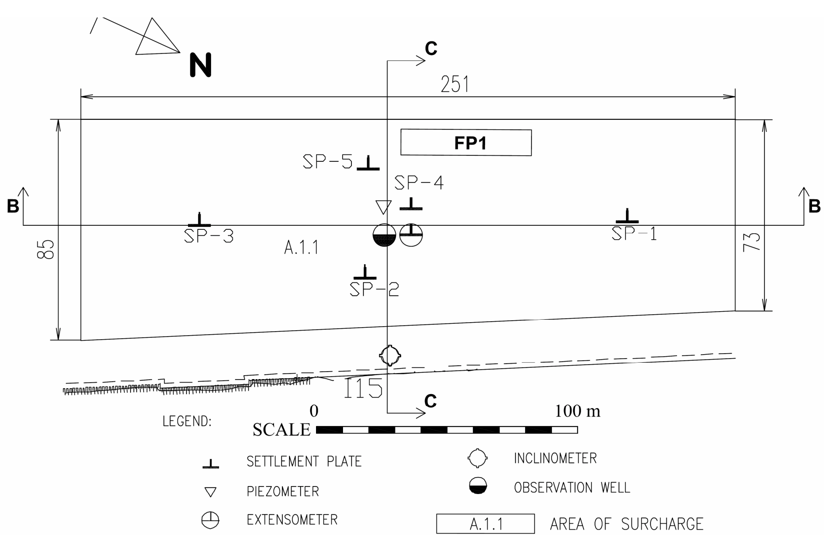

Figure 5b Layout of installation of field instrumentations

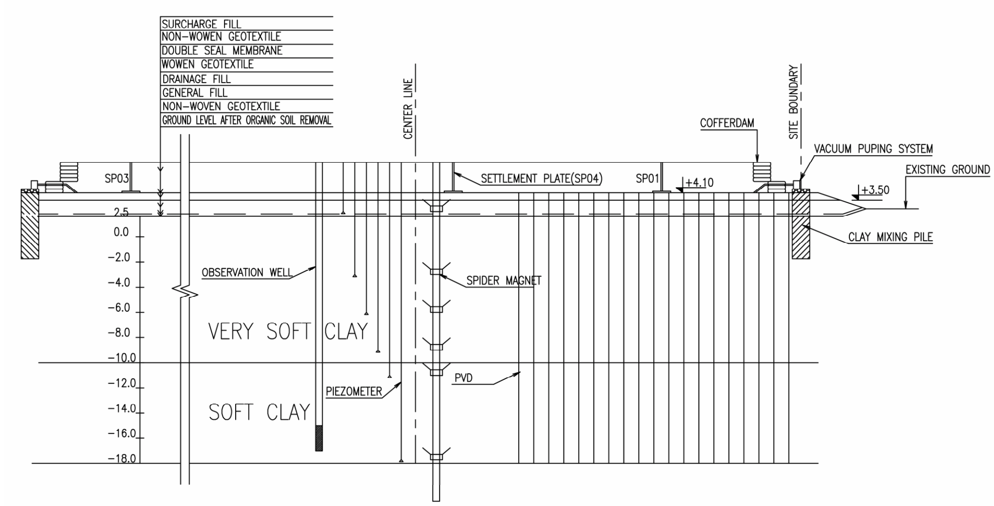

Figure 6 Section views of instrumentation (Section B-B)

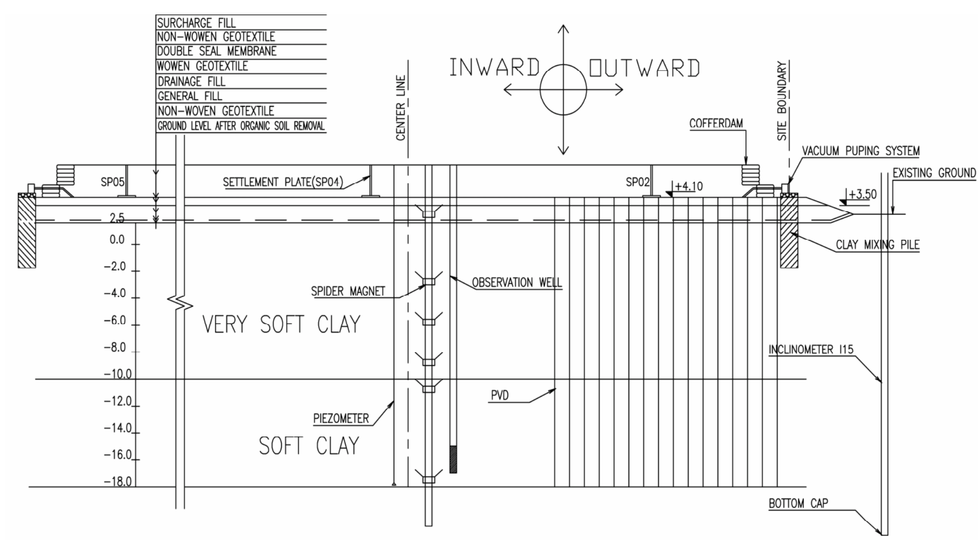

Figure 7 Section views of instrumentation (Section C-C)

From Eq. 1 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:

$$S_{ult} = \frac{\beta_0}{1-\beta_1} \tag{2}$$

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. 8.

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. The ultimate settlement calculated based on the field records at the settlement plate SP04 using Asaoka's method are shown in Fig. 9.

Pore pressure-based method (Chu and Yan, 2005)

Another possibility of assessing the degree of consolidation is based on pore water pressure measurements (Chu and Yan, 2005). To estimate an average degree of consolidation, the pore water distribution over the entire soil depth needs to be established. As a schematic illustration serves Fig. 10, where a combined fill surcharge and vacuum load is considered. The field pore-pressure dissipation with depth is presented in Fig. 11. The average degree of consolidation can be calculated as.

$$U_{avg} = 1 - \frac{\int [u_t(z) - u_s] dz}{\int [u_0(z) + \Delta\sigma - u_s] dz} \qquad (3)$$

$$u_s(z) = \gamma_w z - s \qquad (4)$$

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 applied.

The primary settlement and the time-dependent settlement are calculated using the following equation:

$$S_c = H \cdot \left[RR \cdot \log \frac{\sigma'_{vm}}{\sigma'_{vo}} + CR \cdot \log \frac{\sigma'_{vo} + \Delta\sigma_v}{\sigma'_{vm}} \right] \qquad (5a)$$

$$S_t = U * S_c \qquad (5b)$$

with

$$U = 1 - (1 - U_h)(1 - U_v) \qquad (6)$$

and

$$U_h = 1 - \exp\left(\frac{-8T_h}{\mu}\right) \qquad (7)$$

Where S_t is the settlement versus time, S_c is the primary settlement, U_h and U_v is the horizontal and vertical degree of consolidation.

The calculated settlement are plotted in comparison with the measured values as shown in Fig. 12. The data used in the settlement calculation are tabulated in Table 2.

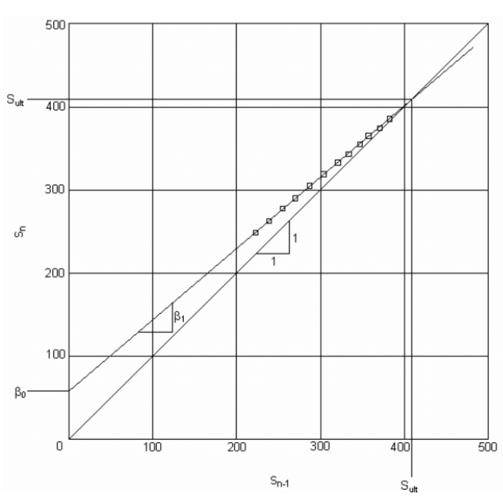

Figure 8 Graphical illustration of Asaoka's method

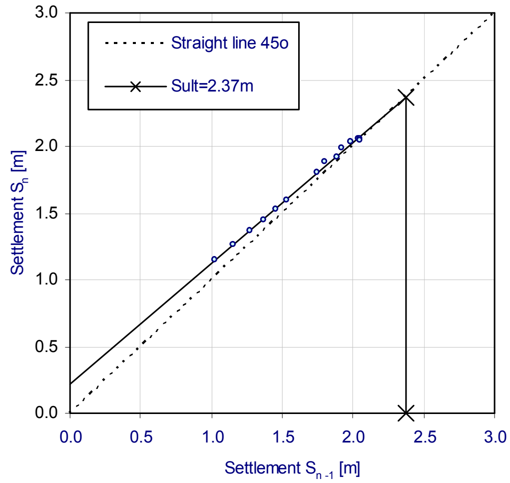

Figure 9 Field settlements, Asaoka's method plots for settlement plate SP04

The degree of consolidation of the clay layers below the test embankments was calculated both from pore-pressure dissipation and from the settlements of the test embankments. 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 03 compares the calculated degrees of consolidation. The degrees of consolidation obtained from settlement measurements were confirmed by the corresponding values from excess pore pressure measurements.

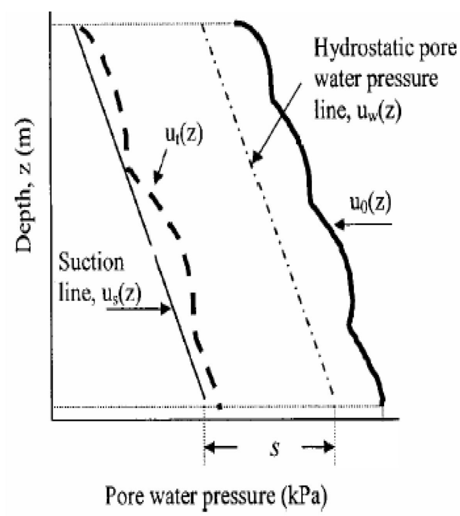

Figure 10 Pore water pressure distribution under combined surcharge and vacuum preloading

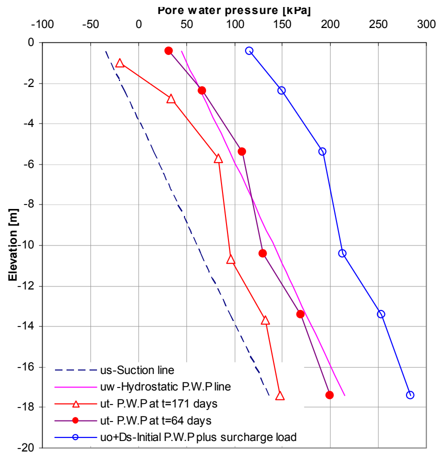

Figure 11 Pore water pressure versus depths

The degree of consolidation obtained from pore pressures (UP) is consistently less than that from settlements (US). A similar observation was reported earlier by Holtz and Broms (1972). These problems have also been observed by Hansbo (1997), Bo (1999) and Bergado (2002). The possible reasons for these differences may be as follows:

- Measurements were conducted at specific points only. Thus, the data may not be representative of the average values for the whole layer.
- Involved uncertainties in the prediction of ultimate settlement, such as measurements of initial settlements or effect of measurements by secondary compression.
- Piezometers were installed between two vertical drains in soil layers. Pore water pressure at this location will be the maximum and will gradually decrease towards vertical drain. Any misalignment in the piezometer vertically will lead to different measurements.

Table 2 Compression ratio (CR), recompression ratio (RR), effective overburden stress (σ'_{vo}), precompression stress (σ'_{vm}), increment loading ($\Delta\sigma_v$) used in analysis of primary consolidation settlement

Depth (m)	CR	RR	σ'_{vo} kPa	σ'_{vm} kPa	$\Delta\sigma_v$ kPa	Settlement m
0 - 2	0.31	0.034	5	12	138	0.70
2 - 4.5	0.23	0.034	18	40	133	0.36
4.5 - 7	0.23	0.034	33	43	128	0.34
7 - 9.5	0.23	0.034	48	63	123	0.26
9.5 - 12	0.28	0.034	63	83	118	0.25
12 - 15.4	0.28	0.034	81	106	113	0.26
					Sum(m)	2.17

Table 3 Comparison of degree of consolidation from settlement and pore pressure data

Degree of consolidation	Field measurements	
	From settlement	From pore pressure
	86.5%	74.6%

Table 4 Back analysis of C_h from settlements and pore pressure data

Location	PVD parameters			Settlement					Pore water pressure		
	S	D_e	d_w	μ	β_1	Δt	C_h	C_h(Ave.)	U_{avg}	t	C_h
	m	m	m			day	m^2/yr	m^2/yr	%	days	m^2/yr
E01	1.2	1.26	0.067	2.20	0.92	7	1.95	2.06			
SP04	1.2	1.26	0.067	2.20	0.91	7	2.17				
P01-P06	1.2	1.26	0.067						75	107	2.0

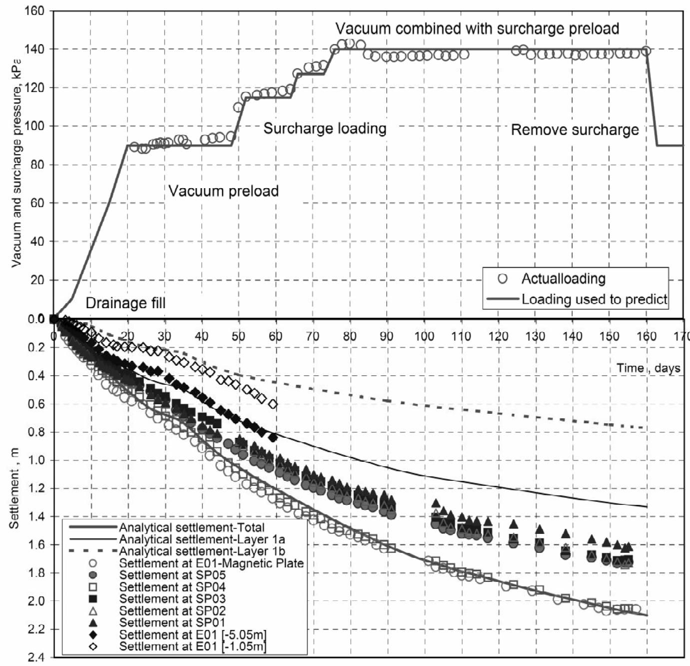

Figure 12 Comparison of settlement between analytical results and monitoring data

BACK-ANALYSIS OF SOIL PARAMETER

Reduction of water content

Changes in water content can also be estimated from the equation based on field settlement data (Stamatopoulos and Kotzias, 1985) as follows:

$$\Delta w_n = -\left(w_n + \frac{1}{G} \right) \frac{\delta}{h} \tag{8}$$

Where 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

Figure 13 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. 13 for test embankment and are in agreement with the measured water content data.

Increase undrained shear strength

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

$$\left(\frac{S_u}{\sigma'_{vo}} \right)_{OC} = \left(\frac{S_u}{\sigma'_{vo}} \right)_{NC} OCR^m \tag{9}$$

where OCR is the overconsolidation 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):

$$\Delta S_u = \left(\frac{1 + w_n G}{0.434 C_c} \right) S_u \frac{\delta}{h} \tag{10}$$

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:

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}} \tag{11}$$

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).

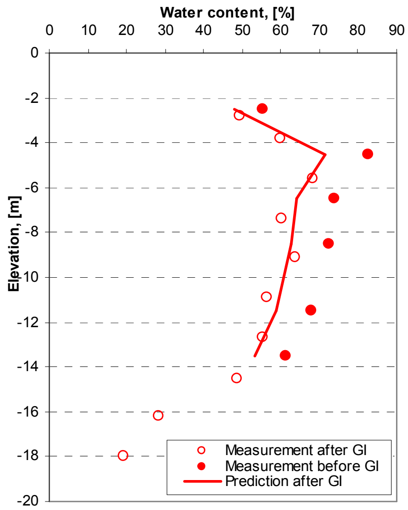

Figure 13 Back-calculated water contents from settlements

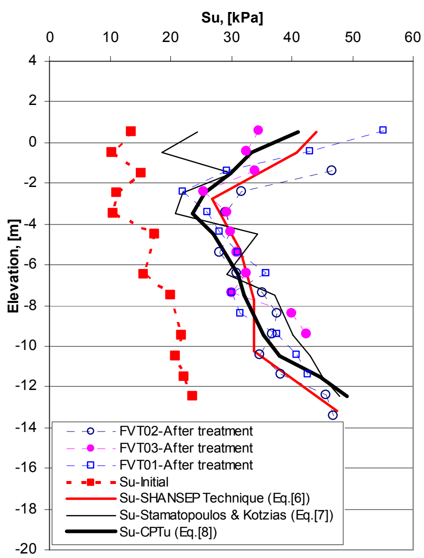

Figure 14 Undrained shear strength before and after treatment

The increase of shear strength can be estimated from the SHANSEP technique (Eq. 9). In this project, the SHANSEP equation can be obtained from field vane shear tests, oedometer tests and Constant Rate of Strain (CRS) tests as follows:

$$\frac{S_u}{\sigma'_{vo}} = 0.215 * OCR^{0.805} \qquad (12)$$

The predicted increases in undrained shear strengths are indicated by "solid lines" in Fig. 14. 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. 14, 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 ~ -2m, the predicted shear strength from field settlement data (Stamatopoulos and Kotzias, 1985) (Eq. 10) does not agree well with direct measurements. Besides, cone resistance, q_c measured by piezocone tests in before and after treatment are also plotted for comparison (Fig. 15). The results indicates that the shear strength and the cone resistance increase with 75% and from 34% (0 ~ -2m) to 72% (-2 ~ -12m), respectively.

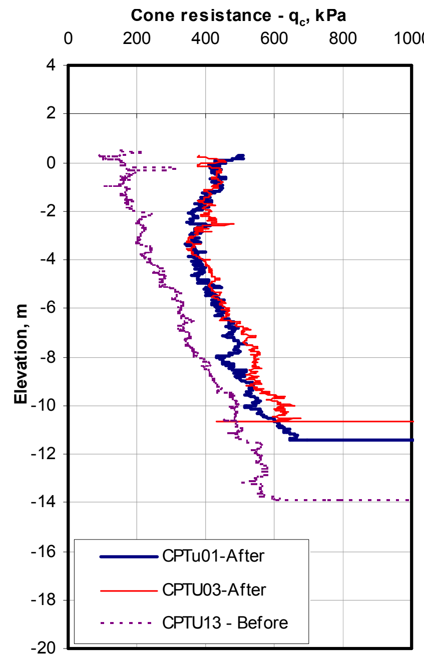

Figure 15 Cone resistance before and after treatment

Back-calculation of c_h values from pore water pressure measurements

In case of radial drainage consolidation, Barron's solution (1948) in perfect drain condition (without considering the effect of smear and well resistance) which is given by:

$$U_h = 1 - \exp\left[\frac{-8T_h}{\mu}\right] \qquad (13)$$

where U_h is average horizontal coefficient of consolidation, T_h is time factor , μ is factor for the effect of drain spacing

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. 13), which results in the following expression:

$$T_h = \frac{-\mu \ln(1-U_h)}{8} \qquad (14)$$

where T_h is time factor:

$$T_h = \frac{C_h t}{D_e^2} \tag{15}$$

By combining Eqs. 15 and 16, the coefficient of radial consolidation C_h can be calculated as follows:

$$C_h = \frac{-D_e^2 \mu \ln(1-U_h)}{8t} \tag{16}$$

Back-calculation of c_h values from settlement measurements

For estimating the in situ coefficient of consolidation, Magnan and Deroy (1980) determined that for radial drainage only, in situ C_h can be estimated by:

$$\frac{\ln \beta_1}{\Delta t} = \frac{8c_h}{\mu D_e^2} \tag{17}$$

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

$$\mu = \frac{n^2}{n^2-1}\ln(n) - \frac{3n^2-1}{4n^2} \tag{18}$$

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

Back-calculation of C_h value based on field settlements and pore pressure measurements were shown in Table 4.

CONCLUSIONS

The ultimate settlement was predicted using Asaoka's method, while the time-depdent settlement was estimated based on Barron's solution. The predicted and measured settlement are within an error of 3 ~ 6%, which are considered to be exceptional.

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 non-linearity of the soil.

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 with 75%.

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%

There is a good agreement between the horizontal coefficient of consolidation back-calculated from field measurements and from soil investigation before treatment (refer Table 1&4).

REFERENCES

Asaoka, A., (1978). Observational Procedure of Settlement Prediction, *Soils and Foundations*, Vol. 18, No. 4: 87-101.

Barron, R. A., (1948). Consolidation of Fined-Grained Soils by Vertical Drain Wells, Transaction, *American Society of Civil Engineers*, Vol. 124: 109-139.

Bergado, D. T., A.S. Balasubramaniam, R. Jonathan Fannin, and Robert D. Holtz, (2002). Prefabricated Vertical Drains in Soft Bangkok Clay: A case study of the new Bangkok International Airport project, *Canadian Geotechincal Journal*, Vol. 18, No. 39: 304-315

Chai, J.C., Carter, J.P. and Hayashi, S. (2006), Vacuum Consolidation and its Combination with Embankment Loading, *Canadian Geotechnical Jou*rnal, Vol.43, pp.985-996.

Charles C. Ladd, Hon. M.. (2003), Recommended Practice for Soft Ground Site Characterization: Arthur Casagrande Lecture, *12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering Massachusetts Institute of Technology Cambridge, MA USA*

Chu, J., Yan, W., and Yang, H (2000). Soil improvement by the vacuum preloading method for an oil storage station. *Geotechnique*, 50(6): 625–632.

Hansbo, S., (2005). Experience of Consolidation Process from Test Areas with and without Vertical Drains, *Ground Improvement — Case Histories, elsevier geo-engineering book series,* Vol. 3

Hansbo, S. (1979), Consolidation of Clay by Band Shaped Prefabricated Drains, *Ground Engineering*, Vol. 12, No. 5, pp. 16-25.

Kjellman, W., (1952). Consolidation of Clay Soil By Means of Atmospheric Pressure, *Proceeding of the Conference on Soil Stabilization*, Massachusetts Institute of Technology: 258-263.

T. Stapelfeldt (Henlsinki University of Technology). Preloading and vertical drains.

Za-Chieh Moh and Penchi Lin (MAA Group Consulting Engineers). Case Study of Ground Improvement Work at the Suvarnabhumi Airport Of Thailand.