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Design and Performance of Soft Ground Improvement Using PVD with and without Vacuum Consolidation

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ABSTRACT: This paper presents the soft ground improvement using Prefabricated Vertical Drains (PVD) including PVD installation and preloading techniques, settlement and stability design calculations, observational methods and back analyses of monitoring data and performance of conventional preloading with surcharge fill and preloading using vacuum consolidation method (VCM) in combination with fill embankment. Several case histories were studied. The monitored data illustrated that the effectiveness of VCM is dependent on the method of applying vacuum pressure to the PVDs. Measured pore pressure in the PVDs at different depths indicated that the effective vacuum pressure inside the PVDs is distributed uniformly along the PVD depth with a magnitude of over 80 kPa for VCM using airtight membrane. Back-calculated c_h values from measured settlement data using Asaoka method confirmed that with the assumed values of $d_s/d_m = 2$ and $k_h/k_s = 2$, the corresponding value of $c_h/c_{v,oed} = 3$ to 5 were obtained for both soft Bangkok (BKK) clay and soft clays in Mekong River Delta (MRD). Also, the linear relationship between compression index and water content for soft clays in MRD is similar to that of BKK clay. The settlements versus time calculated by 1-D method are in very good comparison with measured data for both conventional preloading and VCM considering the vacuum pressure as an induced vertical stress distributed uniformly in the PVD zone. From the results presented in this paper, simple procedures can be made for selection of soil parameters and design calculations of embankments on PVD improved soft ground using conventional preloading and vacuum consolidation.

KEYWORDS : Settlement, stability, soft clay, ground improvement, PVD

1. INTRODUCTION

Most of coastal plains in Southeast Asia (SE) region are covered with soft clay deposits (Figure 1). The soft Bangkok (BKK) clay in Thailand and Muar flat clays in Malaysia are marine clays and extend to great depths of about 10 m to 25 m (Balasubramaniam et al., 2010). In Vietnam, the soft clays in Mekong river delta (MRD) and in Saigon-Dong Nai river lower plain (SDR) including Ho Chi Minh city (HCMC), Ba Ria – Vung Tau and Dong Nai provinces are also typically characteristic of marine deposits. Generalized soil profile of MRD in Figure 2 (Giao et. al, 2008) indicated that the soft soil thickness varies from 10m to more than 40 m. The distribution of soft clay in HCMC and soil profile along the right bank of Saigon River is presented in Figures 3 and 4, respectively. The average thickness of soft clay in this area is more than 20 m (Figure 4).

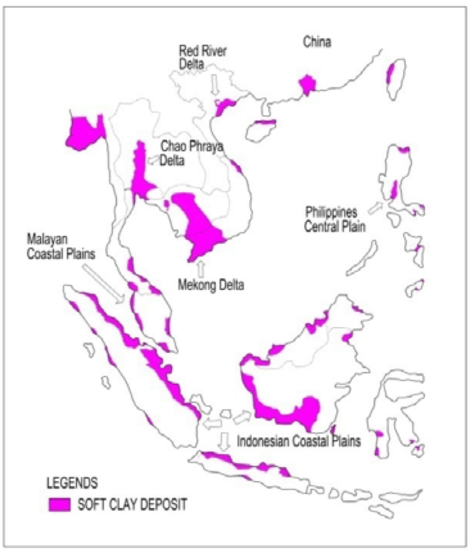

Figure 1 Distribution of soft clay in SE region

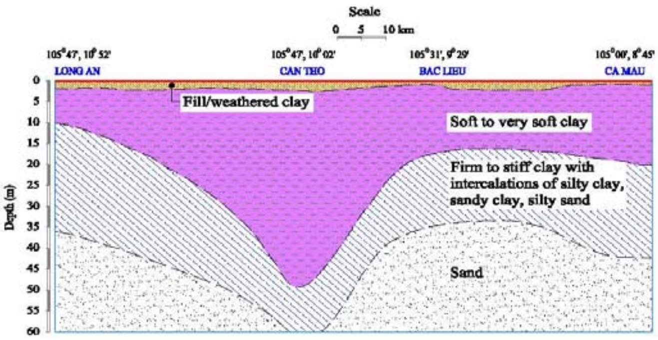

Figure 2 Generalized soil profile in MRD

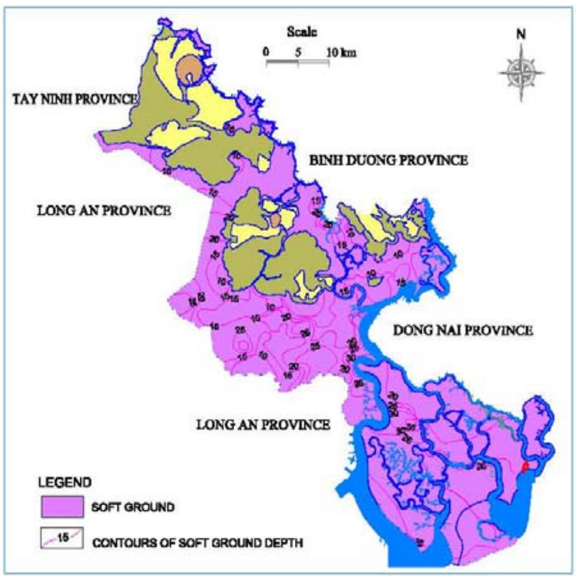

Figure 3 Distribution of soft ground in HCMC

The low shear strength and high compressibility of these soft clays have challenged the wit of the geotechnical design engineer in solving problems related to the stability condition during construction as well as to the residual settlement including differential settlement during operation. Typical failure of embankment slope constructed on soft clay can be seen in Figure 5. Typical landslide of river bank as seen in Figure 6 has occurred very often along Saigon river and Mekong river. Differential settlement (Figure 7) is the major problem of most highway projects constructed on soft ground in MRD and SDR regions.

Preloading using PVD with and without vacuum pumping is the most favorable soft ground improvement method that has been applied in SE Asia but significant differences between the field performance and the design expectations has still existed particularly for the residual settlement after construction. Moreover, the c_h value and the smear effects are still a matter of argument even for soft BKK clay that has been extensively studied in many decades. For example, at the site of the Second Bangkok International Airport (SBIA), the c_h value of 0.75 m^2/year was back-calculated by Seah et al. (2004) while the values of higher than 3 m^2/year were obtained by Balasubramaniam et al. (1995) and Bergado et al. (2002).

This paper presents: i) current pre-loading techniques with PVD; ii) settlement calculation; iii) stability analysis; iv) observational methods; v) case histories; and vi) conclusions and recommendations on practical design calculations of PVD improved soft ground. In addition, back-calculations were analyazed from performance data obtained from test embankments on soft clay improved.

2. PRE-LOADING WITH PVD

Preloading with PVD is a commonly used method for improving soft clay deposits. The function of PVD is to shorten the drainage path for accelerating the consolidation rate. Earliest contributions on the use of preloading with PVD have been made by Hansbo (1960, 1979, 1981 and 1987). The effective surcharge pressure for preloading can be from either the weight of the imposed fill materials and/or the application of a vacuum pressure applied to the soft soil.

2.1 Conventional Preloading Method (CPM)

The soft ground is preloaded by the weight of embankment fills. In this method, excess pore pressure in the soft soil is designed to be dissipated through the horizontal drainage system on the top of PVDs that can be either the sand blanket or prefabricated band drains (PBD). Conventional preloading method using sand blanket (CPM-S) has been successfully constructed with sand blanket thickness of not less than 0.5 m and the embedded length of PVDs in the sand blanket of not smaller than 0.3m. In order to increase the effective preloading pressure, it is necessary to have additional drainage system in the sand blanket that may include horizontal sub-drains and longitudinal main drain connected to a drainage sum with pumping for controlling the water table in embankment as assumed in design calculations. In recent years, the conventional sand blanket has been replaced by prefabricated band drains (PBD), in which, one or two rows of PVDs can be connected to one PBD. The main advantages of conventional preloading method with horizontal band drains (CPM-B) against CPM-S are shorter construction time and lesser or no need of clean sand for sand blanket.

2.2 Vacuum consolidation method (VCM)

The main advantages of preloading with vacuum pumping are lower surcharge, less lateral displacement, no need or smaller counterweight berm, and shorter construction time. There are several methods for applying vacuum pressure to soft ground such as VCM using direct tubing (VCM-DT), VCM using membrane and sand blanket (VCM-MS), VCM using membrane and horizontal band drains (VCM-MB), VCM using membrane and flexible perforated tube (VCM-MT).

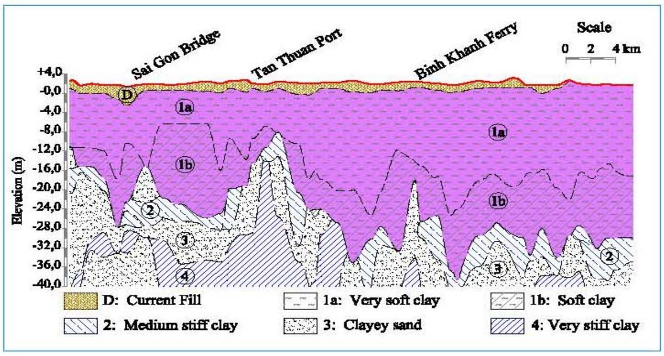

Figure 4 Soil profile along the right bank of Saigon River

Figure 5 Slope failure of embankment on soft soil

Figure 6 Landslide of Saigon river bank on soft ground

Figure 7 Differential settlement at approach to bridge

2.2.1 VCM using direct tubing (VCM-DT)

In this method, PVDs are connected to the vacuum pump through a flexible tubing system using a tubing cap for each individual PVD as seen in Figure 8a and Figure 8b. The tubing cap is located at the depth of about one meter from the soft ground surface for air-tight sealing by bentonite slurry or in-situ clay. The advantage of this method is no need of clean sand and airtight geomembrane.

Figure 8a Connection of PVD and flexible tube

Figure 8b Tubing system of VCM-DT

2.2.2 VCM using geomembrane with sand blanket (VCM-MS)

Sand blanket and embedment of PVDs in the sand blanket are similar to that of CPM-S. The sand blanket is covered with an airtight membrane enable to transfer vacuum pressure from sand blanket to soft soil through PVDs. Vacuum pressure can be applied in the sand blanket through a sub-drain system using flexible perforated pipes placed at spacing of several rows of PVDs.

2.2.3 VCM using geomembrane with band drain (VCM-MB)

Typical construction of this method is presented in Figure 9. Vacuum pressure can be applied to PVDs through longitudinal drain using perforated pipe and horizontal band drains.

Figure 9 VCM using membrane and PBDs

2.2.4 VCM using membrane and perforated tube (VCM-MT)

For this method, horizontal drainage system under airtight membrane consisted of horizontal drains using flexible, corrugated, perforated tube and longitudinal main drain of PVC or HDPE pipe. PVD was connected to horizontal pipe by winding with string tie (Figure 10a) and using 4-way or T-shaped connector for connection between horizontal drainage tube and longitudinal pipe (Figure 10b).

Figure 10a Connection of PVDs to perforated-horizontal pipe

Figure 10b Connection between horizontal and longitudinal pipes

2.2.5 Specifications of PVD

Typical specifications of PVD have been applied in design practice given by Federal Highway Administration – USA (FHWA), Department of Highway – Thailand (DOH), and Ministry of Transportation – Vietnam (MOT) are summarized in Table 1.

Table 1 Specifications of PVD

Properties and Testing Standard	**DOH**	**FHWA**	**MOT**
Apparent Opening Size, µm ASTM D4751-87	≤ 90	-	≤ 75
Grab Tensile Strength, kN (whole PVD) ASTM D4632-91	≥ 0.35	≥ 0.35	-
Tensile strength, kN (whole PVD) ASTM D4595	-	-	≥ 1.00
Puncture Resistance, kN (filter only) ASTM D4833-88	≥ 0.20	≥ 0.22	-
Discharge Capacity @7days, 200 kPa at i = 1, m^3/year ASTM D4716-87	≥ 500	≥ 500	-
Discharge Capacity 350 kPa at i = 0.5, m^3/year ASTM D4716-87	-	-	≥ 1890

3. SETTLEMENT CALCULATION

3.1 Final Primary Settlement

From 1-D conventional oedometer test, final primary settlement, S_{oed}, can be calculated as follows:

$$S_{oed} = \sum h[RR.\log(\sigma'_p/\sigma'_{vo})+CR.\log(\sigma'_{vf}/\sigma'_p)] \quad (1)$$

where h is thickness of the calculated sub-soil layer, CR and RR are compression and re-compression ratio, σ'_{v0} is existing overburden, σ'_p is pre-consolidation pressure, and σ'_{vf} is the calculated final effective vertical stress.

The final primary settlement of soft ground may consist of final primary consolidation settlement due to effective stress increase, S_{cf}, and immediate settlement due to undrained deformation, S_i. For most non-sensitive soft marine clays, following expressions have been used in design practice:

$$S_{cf} = \mu_c S_{oed} \text{ and } S_i = (1-\mu_c)S_{oed} \quad (2)$$

where μ_c can be taken as 1 to 0.8 depending on the ratio of soft ground thickness to embankment width, OCR of the soft soil, and preloading technique.

The final primary consolidation settlement under long-term service loads of embankment on soft ground can be calculated using Eq (1) and Eq. (2) with the value of σ'_{vf} in Eq. (1) should be determined as below:

$$\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma_v + (u_0 - u_f) \quad (3)$$

where $\Delta\sigma_v$ is increase of total vertical stress due to dead load of embankment materials and permanent imposed loads acting on the embankment surafce, u_0 is initial pore pressure (just before embankment construction), and u_f is the final pore pressure. It can be seen that u_f can be smaller than u_0 for the case of pore pressure draw-down due to ground water pumping.

3.2 Consolidation with PVD

3.2.1 Degree of consolidation

The degree of consolidation, U, can be estimated as below:

$$U = 1-(1-U_h)(1-U_v) \quad (4)$$

where U_h and U_v is degrees of consolidation in horizontal and vertical direction, respectively. For PVD improved zone, U_v can be neglected. For underlying soil layers without PVD, $U_h = 0$ and the upper drainage boundary for vertical consolidation can be set at the bottom of PVDs.

Hansbo (1979) presented the solution for calculating the degree of horizontal consolidation, U_h, of soft ground improved by PVD as follows:

$$U_h = 1-\exp(-8T_h/F) \quad (5)$$

$$T_h = c_h\, t / d_e^2 \quad (6)$$

$$F = F_n + F_s + F_r \quad (7)$$

$$F_n = \frac{n^2}{n-1}\log_e(n) - \frac{3n^2-1}{4n^2} \quad (8)$$

$$F_s = (k_h/k_s - 1)\log_e(d_s/d_w) \quad (9)$$

$$F_r = \pi z\,(2L - z)k_h/q_w \quad (10)$$

$$d_w = (a+b)/2 \quad (11)$$

where c_h is the coefficient of horizontal consolidation, d_e is the equivalent diameter of a unit PVD influence zone, k_h is the horizontal permeability of the soft soil, k_s is the horizontal permeability of soft soil in smear zone, z is the distance from the drainage end of the drain, L is the length PVD for one way drainage and is half of PVD length for drainage boundary at both ends of PVD, q_w is the in-situ discharge capacity of the PVD, d_w is the equivalent diameter of PVD, a and b are thickness and width of PVD, and d_s is the diameter of smear zone due to PVD installation that can be related to the equivalent diameter of the mandrel, d_m, recommended by Hansbo (1987) as follows:

$$d_s = 2\,d_m \quad (12)$$

$$d_m = 2(w.l/\pi)^{0.5} \quad (13)$$

where *w* and *l* are the width and thickness of the mandrel.

The value of k_h/q_w in Eq. (10) is often smaller than 0.0001 for most practical cases. Thus, value of the well resistance F_r becomes negligible in comparison with the values of F_n and F_s. Balasubramaniam et al. (1995), Bergado et al (1996, 2002) and Long et al (2006) also indicated that the well resistance has very little effect when the in-situ discharge capacity of PVD greater than 50 m^3/year. Therefore, with a known value of PVD spacing, the main parameters influencing on the calculated consolidation rate are the values of c_h, $R_s = k_h/k_s$, and d_s/d_m that have to be assumed in design practice.

3.2.2 Consolidation settlement during preloading

Consolidation settlement at time t during preloading stage can be estimated from the corresponding degree of consolidation, U_t, and the final consolidation settlement, S_{cf}, under peloading load.

$$S_{ct} = U_t.\, S_{cf} \quad (14)$$

The value of S_{cf} can be calculated using Eqs. (1) and (2), in which, the value of σ'_{vf} can be estimated as follows:

-For PVD with conventional preloading (without vacuum pumping):

$$\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma_v \quad (15)$$

where $\Delta\sigma_v$ is induced vertical stress caused by the embankment pressure, p_{fill}, acting on the ground surface, p_{fill}.

$$p_{fill} = \sum \gamma_{fill}.h_{fill} - \gamma_w.h_w \quad (16)$$

where γ_{fill} is total unit weight of fill materials, h_{fill} is thickness of fill layer, γ_w is unit weight of water, and h_w is the thickness of embankment fill below water table during preloading period.

-For PVD with vacuum pumping using direct tubing system (without air-tight membrane):

$$\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma_v + p_{vac} \quad (17)$$

where $\Delta\sigma_v$ is induced vertical stress due to embankment fill, p_{fill}, as given in Eq. (15), and p_{vac} is effective vacuum pressure.

For PVD with vacuum pumping using air-tight membrane: the value of σ'_{vf} can also be estimated by Eq. (17) but the induced vertical stress $\Delta\sigma_v$ should be determined with embankment pressure of $p_{fill} = \sum \gamma_{fill}.h_{fill}$ considering that porepressure in embankment fill under the airtight membrane is negative.

3.3 Secondary Compression Settlement

Secondary compression is the slow compression of soil that occurs under constant effective stress after the excess pore pressures in the soil fully dissipated. From conventional oedometer test, the secondary compression settlement, S_s, at time t can be expressed as below:

$$S_s = h.C_{\alpha\varepsilon}\log(t/t_p) \quad (18)$$

where $C_{\alpha\varepsilon}$ is secondary compression ratio, and t_p is the time at the end of primary consolidation. For embankment on soft clay without ground improvement, secondary settlement can be neglected because the time to complete primary consolidation settlement would be greater than the life time of the project. However, for soft ground improved by PVD, the time to reach 90% of consolidation can be of about one year. Thus, the value of t_p = 1 year has been widely used in design practice for evaluating the residual settlement of PVD improved soft ground. The value of $C_{\alpha\varepsilon}$ in Eq. (18) should be taken in normally consolidated (NC) state, $C_{\alpha\varepsilon(NC)}$, or in over-consolidated (OC) state, $C_{\alpha\varepsilon(OC)}$, depending on stress state of the soil under service load in operation period.

3.4 Residual Settlement

Residual settlement or post-construction settlement, RS, from the end of construction (EOC) to a design time, t_D, for soft ground improved by PVD can be expressed as follows:

$$RS = \Delta S_{c\,(I)} + S_{s\,(I)} + \Delta S_{c\,(II)} \quad (19)$$

where $\Delta S_{c\,(I)}$ is the remaining primary consolidation settlement under service load of PVD improved soil layers, $S_{s(I)}$ is secondary compression of PVD improved soil layers, and $\Delta S_{c(II)}$ is the remaining primary consolidation under service load of underlying soil layers without PVD improvement.

The maximum value of RS shall be smaller than the specified value as given in the design criteria of the project. Some residual settlement criteria for embankment on soft ground are summarized in Table 2.

Table 2 Residual settlement criteria

Country	Settlement criteria
USA	(a) A slope of 1/200 is typically accepted. (b) 0.5 in (12.5 mm) differential settlement at the interface with bridge likely require maintenance but not intolerable [*]
Singapore	Post construction settlement for road embankment should not be more than 50 mm and differential not more than 1/200.
Malaysia	<u>North-South Highway Concessionaire's design criteria</u> (a) Total settlement in the first 7 years shall nowhere exceed 400 mm. (b) Differential settlement in the first 7 years shall not exceed 100 mm within a length of 100 m for transition zones.
	<u>JKR (PWD) design criteria</u> Total post construction settlement shall be smaller than 250 mm except for approach embankment. For embankment within 10 m from bridge abutment, the above settlement criteria should be reduced to 15%.
Vietnam	<u>MOT (22TCN-262:2000)</u> Post construction primary consolidation settlement for expressway and highway embankment with design speed of 80 km/hr shall be smaller than 100 mm, 200 mm, and 300 mm corresponding to embankment approach to bridge, near the culvert, and other areas remote from the structures, respectively.
	<u>CMIT Container Terminal project</u> a) Total post construction settlement in 50 years shall be smaller than 1.25m. b) Maximum differential settlement within a length of 50 m shall be smaller than 100 mm.
	<u>PM3- Ca Mau project</u> a) Total post construction settlement in 30 years smaller than 30 cm. b) The maximum rate of residual settlement smaller than 2 cm/year.

The secondary compression rate in NC state $C_{\alpha\varepsilon(NC)}$ of soft Bangkok clay and MRD marine clays is in the range of about 1% to 1.5%. With the PVD improved depth of about 20 m, the secondary compression settlement in 20 years would be about 26 cm to 34 cm. Thus, for satisfying the residual settlement criteria of smaller than 20 cm in 20 years, the soft clay must be preloaded to be lightly over-consolidated under service load. As such, the degree of consolidation corresponding to service load at the time of removing surcharge should be greater than 100%.

3.5 Selection of Design Parameters

Design parameters needed for settlement and degree of consolidation (DOC) analyses are tabulated in Table 3.

Table 3 Design parameters for settlement analyses

Analysis	Design parameters
Final consolidation settlement, S_{cf}	Initial overburden, σ'_{v0} Final consolidation stress, σ'_{vf} Pre-consolidation stress, σ'_p Virgin compression ratio, CR Re-compression ratio, RR
Secondary compression settlement, S_s	Secondary compression ratio in NC state, $C_{\alpha\varepsilon(NC)}$ Secondary compression ratio in OC state, $C_{\alpha\varepsilon(OC)}$
DOC in vertical drainage, U_v	Coefficient of consolidation, $c_v = k_v/(m_v\gamma_w)$
DOC in horizontal drainage with PVD, U_h	Horizontal coef. of consolidation in NC state, $c_{h(NC)}$. Smear zone diameter, d_s Permeability ratio, $R_s = k_h/k_s$ In-situ discharge capacity of PVD, q_w

Compressibility and flow parameters are often determined by conventional oedometer test (ASTM D 2435-96). Fundamentals of 1-D consolidation behavior given by Ladd & DeGroot (2003) as seen in Figure 11 that illustrates the significant changes in compressibility and flow properties when a clay is loaded beyond the pre-consolidation stress.

From Figure 11, as the loading changes from OC state to NC state, c_v and $C_{\alpha\varepsilon}$ also undergo marked changes. The rate of secondary compression increases as σ'_v approaches σ'_p and often reaches a peak just beyond σ'_p. The values of $C_{\alpha\varepsilon(NC)}$ / $C_{\alpha\varepsilon(OC)}$ and $c_{v(OC)}$ / $c_{v(NC)}$ of undisturbed clay (solid line) are greater than that of the disturbed one (dotted line). For undisturbed clay, the value of $c_{v(OC)}$ is typically of 5 to 10 times the value of $c_{v(NC)}$. The value of $C_{\alpha\varepsilon(NC)}$ / $C_{\alpha\varepsilon(OC)}$ is essentially equal to the value of CR/RR (Mesri and Castro, 1987).

When consolidation tests are few and also, the results are strongly affected by sample disturbance, following empirical correlations have been often used for estimating compressibility and flow parameters.

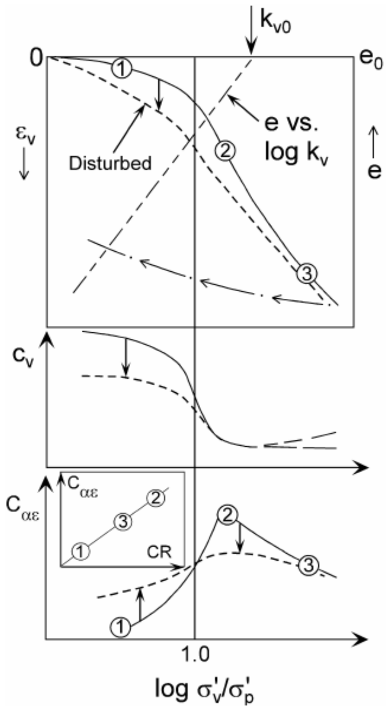

Figure 11 Fundamentals of 1-D consolidation behavior (Ladd & DeGroot, 2003)

3.5.1 Compressibility parameters

Some empirical relations of compressibility with natural water content, ω, are summarized in Table 4.

Table 4 Correlations for RR and C_c versus ω

Authors	Formula	Remark
Simon and Menzies (1975)	CR = 0.06ω - 0.03	20 < ω < 140
Willkes (1974)	CR=0.26 ln(ω) – 0.83	30 < ω < 90
Lamb and Whitman (1969)	CR=0.12 ln(ω) – 0.28	10 < ω < 100
Bergado et al (1996)	C_c = 0.016ω - 0.295	Bangkok clay

For secondary compression, the value $C_{\alpha\varepsilon}$ obtained from oedometer test at NC state is smaller than actual due to sample disturbance as seen in Figure 11. Back-calculated value of $C_{\alpha\varepsilon(NC)}$ is about 1.5 times greater than that from laboratory oedometer test (Long et al., 2006). Mesri et al. (1994) provided the relation between $C_{\alpha\varepsilon}$ and CR as in Table 5. Mesri & Castro (1987) also demonstrated that $C_{\alpha\varepsilon(NC)} / C_{\alpha\varepsilon(OC)}$ = CR/RR with typical value of CR/RR = 5 to 10.

Table 5 Correlations for $C_{\alpha\varepsilon}$ (Mesri et al., 1994)

Soil	$C_{\alpha\varepsilon(NC)}$/CR
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat	0.06 ± 0.01

3.5.2 Stress history

The over-consolidated ratio, OCR, can be interpreted from laboratory oedometer test and field tests. Typical correlation for OCR from field vane test given by Chandler (1988) as follows:

$$OCR = [(s_{u(FV)} / \sigma'_{v0})/S_{FV}]^{1.05} \tag{20}$$

where $s_{u(FV)}$ is field vane shear strength, σ'_{v0} is existing overburden, and S_{FV} is normalized field vane shear strength at OCR = 1 as given in Figure 12 Chandler (1988).

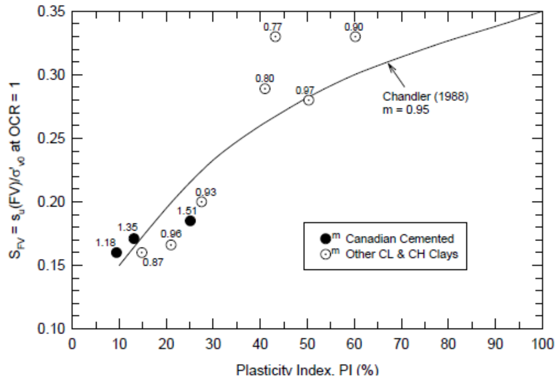

Figure 12 S_{FV} versus plasticity index

3.5.3 Flow parameters and Smear effect

During preloading, the PVD improved soft soil is mainly in NC state. Thus, the value of horizontal coefficient of consolidation in NC state, $c_{h(NC)}$, should be used in estimating the rate of consolidation. The value of c_h can be estimated from the results of laboratory consolidation tests, field CPTu tests, or back analysis data. However, it is really difficult for selecting the reliable value of $c_{h(NC)}$ at NC state. The laboratory tests normally under-estimate the c_v and c_h values while the c_h value obtained from field CPTu test at in-situ stress is at lightly OC state needed to be scaled down to NC state. In design practice, the c_h value is often estimated from laboratory c_v value with assumed value of k_h/k_s and using the relation of $c_h = (k_h/k_v)c_v$. Back-calculated c_h values from measured settlement using Asaoka method are dependent on the assumed values of smear zone diameter, d_s, the ratio of $R_s = k_h/k_s$, and the field value of discharge capacity of the drain, q_w. As discussed above, for the in-situ value of q_w greater than 50 m^3/year, the well resistance becomes negligible. The smear effects include the smear zone diameter, d_m, and the ratio of k_h/k_s The smear zone is directly related to the size of the mandrel with the value of ds/dm = 2 to 3 is commonly accepted (Hansbo, 1987; Jamiolkowski et al., 1983; Bergado et al. 1991; Indraratna and Redana, 1998) but the value of kh/ks has been assumed in a wide range from 1.4 to higher than 10 (Indraratna et al. 2005; Seah et al. 2004; Bergado and Long, 1994; Bergado et al. 1992, 1993). For soft Bangkok clay at the Second Bangkok International Airport site (SBIA), based on the measured settlement from three full scale test embankments, with assumed values of d_s/d_m = 2 and k_h/k_s = 5, Balasubramaniam et al. (1995) and Bergado et al. (2002) obtained the minimum value of c_h of about 3 m^2/year (Figure 13) for the weakest soil layer from 4 m to 8 m. Also from measured settlement data of another embankment at SBIA site, with assumptions of d_s/d_w = 2 and k_h/k_s = 1.4, Seah et al. (2004) obtained the value of c_h = 0.75 m^2/year that is even smaller than the c_v value from laboratory oedometer tests as seen in Figure 14.

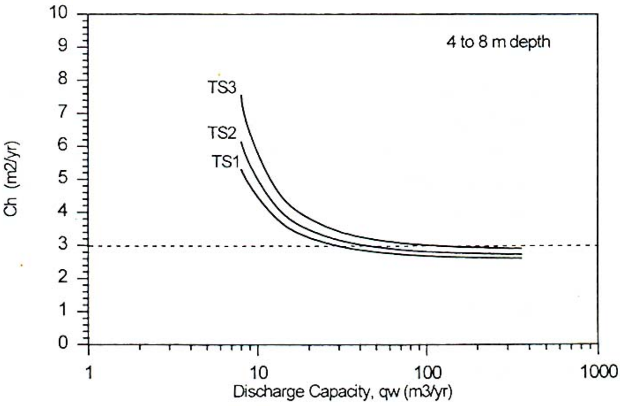

Figure 13 Back-calculated c_h value of TS1, TS2,TS3 embankments at SBIA site using $d_s/d_m = 2$ and $k_h/k_s = 5$ (Bergado et al., 2002)

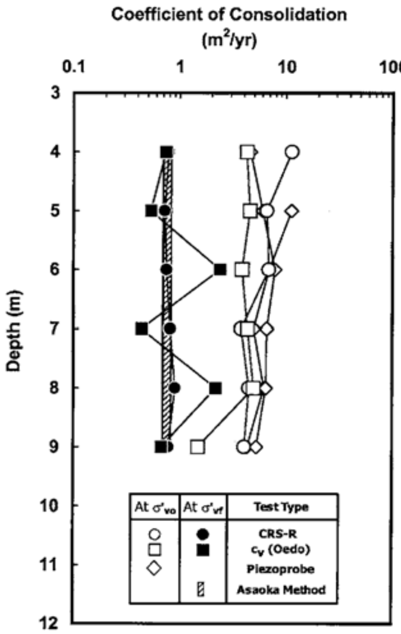

Figure 14 Coefficients of consolidation for soft clay at SBIA site using $d_s/d_w = 2$ and $k_h/k_s = 1.4$ for back calculation of c_h (Seah et al., 2004)

4. STABILITY ANALYSIS

Undrained strength analysis for limit equilibrium method using computer program with circular or non-circular failure surface has been used widely for evaluating the overall stability during construction of embankment on PVD improved soft ground. The increase of undrained shear strength of improved soft soil and settlement of embankment should be taken into account. Initial undrained shear strength of soft clay, s_{u0}, should be selected based on field vane shear test results. The increase of undrained shear strength, Δs_u, due to consolidation can be obtained from the corresponding increase of effective stress, $\Delta\sigma'_v$, that can be estimated from calculated consolidation settlement during design stage and can be re-checked with measured settlement and pore pressure during construction.

Using SHANSEP formula, $s_u = S.\sigma'_{v0}.(OCR)^m$, following expressions can be derived:

- For $\sigma'_v < \sigma'_p$:

$$\Delta s_u = S(1\text{-}m)\Delta\sigma'_v\,(\sigma'_p/\sigma'_v)^m \tag{21}$$

- For $\sigma'_v \geq \sigma'_p$:

$$\Delta s_u = \Delta s_{up} + S(\sigma'_v - \sigma'_p) \tag{22}$$

$$\Delta s_{up} = S(1\text{-}m)(\sigma'_p - \sigma'_{v0}) \tag{23}$$

where m = 0.8 has been applied successfully for marine soft clays and the normalized strength at NC state, $S = s_u/\sigma'_{vc}$, can be estimated from Figure 15.

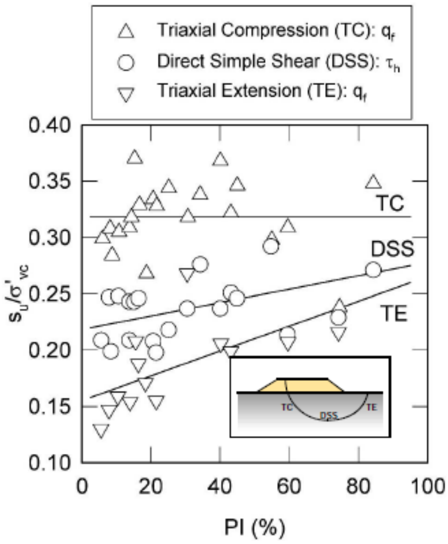

Figure 15 Undrained strength anisotropy from CK_0U tests on NC clays and silts (Ladd, 1991)

The most critical case of embankment construction on soft ground is at the end of filling (EOF) when maximum filling is achieved. For PVD improved soft ground, the settlement at EOF can be more than 50 % of the target settlement at EOP. This big amount of settlement should be included for updating the embankment geometry in stability analysis. The design value of factor of safety FS should be selected based on the reliability of soil parameters as well as on the monitoring plan during embankment construction. Generally, FS = 1.1 to 1.3 have been adopted in successfully constructed projects. For enhancement of FS and/or reduction of counterweight berm, geotextile basal reinforcement has been often used. The mobilized tensile strength in the reinforcement, T_{mob}, should be selected as:

$$T_{mob} \;\geq\; T_{ult} / FS_g$$

where T_{ult} is the characteristic short term tensile strength at elongation of not higher than 10% for strain compatibility and FS_g is the product of partial factors for creep rupture, construction damage, and environment effects. The value of FS_g has been often taken as 2 and 4 corresponding to polyester and polypropylene reinforcement, respectively.

For filling surcharge with vacuum consolidation using membrane technique (VCM-MS or VCM-MT), the shear strength of sand fill material under the air-tight membrane can be increased significantly due to suction during vacuum pumping. However, this effect should not be considered in practical design for preventing the risk of unexpectedly stopping the vacuum pumping.

5. OBSERVATIONAL METHOD

5.1 Settlement Control

Asaoka method (Asaoka, 1978) and Hyperbolic method (Tan et al., 1991) are commonly used methods for prediction of settlement with time based on the measured settlement data.

5.1.1 Asaoka method

The 1-D consolidation equation can be expressed in a first-order approximation as below:

$$S_t = S_f\,[\,1 - \exp(\lambda t)\,] \tag{24}$$

where S_t is consolidation settlement at time t under a constant load, S_f is final consolidation settlement, and λ is a constant. Asaoka (1978) pointed out that Equation 24 is a solution of following differential equation:

$$dS/dt - \lambda S = f \tag{25}$$

where f is an unkmown constant.

From Eq. (25), the time is evenly discritized in Δt interval, and following expression can be obtained:

$$S_k = \beta_0 + \beta_1 S_{k-1} \tag{26}$$

$$\beta_1 = 1/(1-\lambda\Delta t) \tag{27}$$

$$\beta_0 = f\,\beta_1 \tag{28}$$

where S_k is settlement at time $t = t_k$ and S_{k-1} is settlement at time $t = t_k - \Delta t$.

From Eq. (26), the values of β_0 and β_1 can be obtained as the intercept and the slope of the best fitted straight line of ($S_k \sim S_{k-1}$) plot. As time approaches infinity, $S_k = S_{k-1} = S_f$, then:

$$S_f = \beta_0 / (1-\beta_1) \tag{29}$$

From Eqs. (5), (24), and (27), the following expression can be derived for c_h value that is often used for back analysis of monitored settlement data.

$$c_h = \frac{(1-\beta_1)\, d_e^2\, F}{8\,\beta_1\,\Delta t} \tag{30}$$

It is noted that Eq. (24) is an approximation for primary consolidation settlement. Thus, the final primary settlement S_f from the straight line of ($S_k \sim S_{k-1}$) plot should be drawn through the measured points with S_k smaller than 0.9 S_f. Otherwises, secondary settlement would be included in the value of S_f as seen in Figure 16.

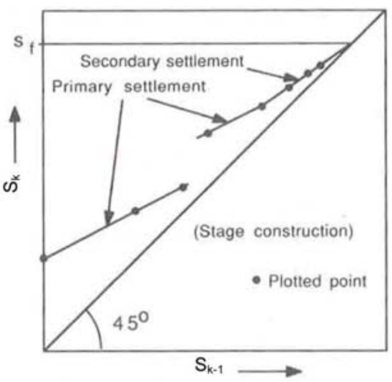

Figure 16 Asaoka plot for final settlement (Hausmann, 1990)

5.1.2 Hyperbolic method

Tan et al. (1991) suggested that the time-settlement can be approximated by following hyperbolic equation:

$$S_t = S_1 + t/(\alpha + \beta t) \tag{31}$$

where S_1 is immediate settlement, and $t/(\alpha + \beta t)$ is consolidation settlement at time t, when $t \rightarrow \infty$:

$$S_\infty = S_f = S_1 + 1/\beta \tag{32}$$

From Eq. (31), the values of α and β can be obtained as the intercept and the slope of the best fitted straight line of $t/(S_t - S_1) \sim t$ plot.

5.1.3 Target settlement at removing surcharge

The target settlement is the required settlement, S_{req}, to be achieved at the time of removing surcharge for satisfying the criteria of residual settlement. The value of S_{req} is often specified using the degree of settlement under preloading load, DOS = S_{req}/S_f, where S_f is final settlement under design preloading load. Depending on the requirements of construction time and surcharge conditions, the design value of DOS of 80% to 90% has been commonly adopted in practice.

If the residual settlement in Eq. 19 is calculated with $C_{\alpha\varepsilon(OC)}$ for the PVD improved zone (zone I), then it should be preloaded to lightly OC state under service load and the target settlement should be determined as:

$$S_{req} \geq S_{i\,(I+II)} + S_{c(II)} + 1.1\, S_{fc\,(I)} \tag{33}$$

where $S_{i(I+II)}$ one is immediate settlement under preloading load of zone I and underlying unimproved soil layers (zone II), $S_{c(II)}$ is consolidation settlement of zone II under preloading load at the end of preloading (EOP), and $S_{fc(I)}$ is final primary consolidation settlement under long term service load of zone I.

If the residual settlement satisfying the setttlement criteria with secondary settlement of zone I calculated with $C_{\alpha\varepsilon(NC)}$, then the target settlement can be as below:

$$S_{req} \geq S_{i\,(I+II)} + S_{c(II)} + U.S_{fc\,(I)} \tag{34}$$

$$U \geq [\,S_{fc(I)} - \Delta S_{c(I)}\,] / S_{fc(I)} \tag{35}$$

where U is the degree of consolidation under service load, $\Delta S_{c(I)}$ is the remainning consolidation settlement of zone I under service load that can be calculated using Eq. 19 with RS is the allowable residual settlement.

5.2 Stability Control

The most common method that has been applied for stability control during construction of embankment on soft ground was suggested by Wakita and Matsuo (1994) as re-presented in Figure 17, in which, δ is settlement at the center of embankment, h is maximum lateral displacement at the toe of embankment, q is embankment pressure at the considered time, and q_f is embankment pressure at failure. Expereiences from actual projects indicated that the stability control chart in Figure 17 is conservative for the case of PVD improved soft ground with large consolidation settlement.

Other methods have been employed for stability evaluation based on the measured data including the rate of settlement, rate of lateral displacement, ratio of lateral displacement to settlement. Some warning values for stability control have been applied sucessfully for embankment construction on PVD improved soft ground in Viet Nam as summarized in Table 6. When level 1 is reached, cautioned work is required with reducing filling rate and when level 2 is approached, stop filling with increasing of monitoring frequency.

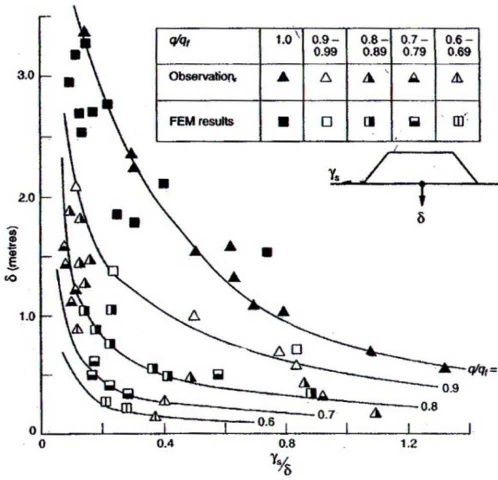

Figure 17 Embankment stability control chart

Table 6 Warning values for stability control

Method	Level 1	Level 2
Wakita and Matsuo method q/q_f	0.9	1.0
Rate of lateral displacement (mm/day)	15	20
Rate of settlement (mm/day)	30	40
Ratio of lateral displacement to settlement	0.5	0.7

6. CASE HISTORIES

6.1 CMIT Project

Cai Mep International Terminal project (CMIT) was constructed in Baria-Vung Tau province (Figure 18). The basic properties of soft soil are presented in Figure 19a and Figure 19b. The original ground surface is at elevation of about EL. +2.00 m to +3.00 m CD, the top of sand blanket is at EL. +5.5 m CD. PVDs were installed down to the bottom of soft soil layer at 34 to 35 m.

Figure 18 Location of the project sites

The mandrel with cross section of 70mm by 140 mm was employed. Vacuum consolidation using airtight geomembrane with sand blanket (VCM-MS) and PVDs spacing of 0.9 m in square pattern was applied for Section I of 50 m width along the river bank. In the remainning area (Section II), conventional preloading with surchage fill was adopted using PVDs spacing of 1.2 m in square pattern.

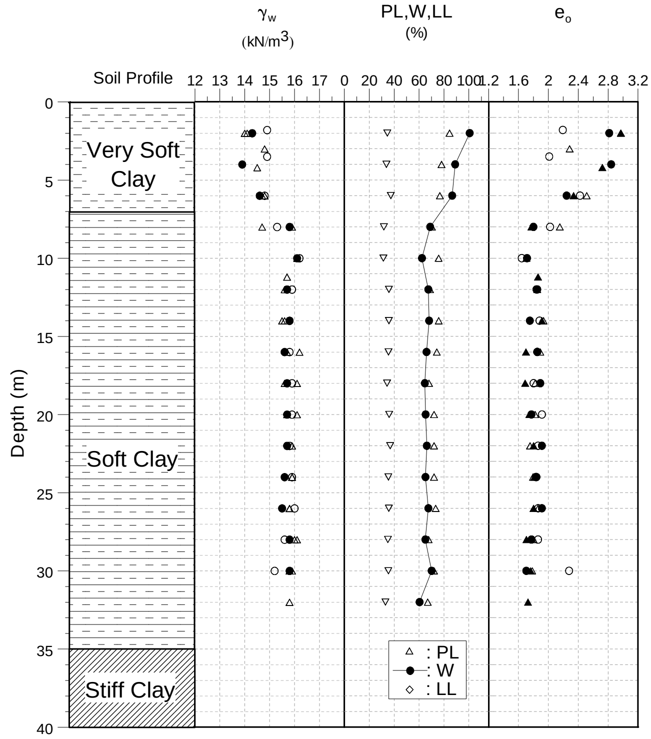

Figure 19a Basic properties of soft clay at CMIT site

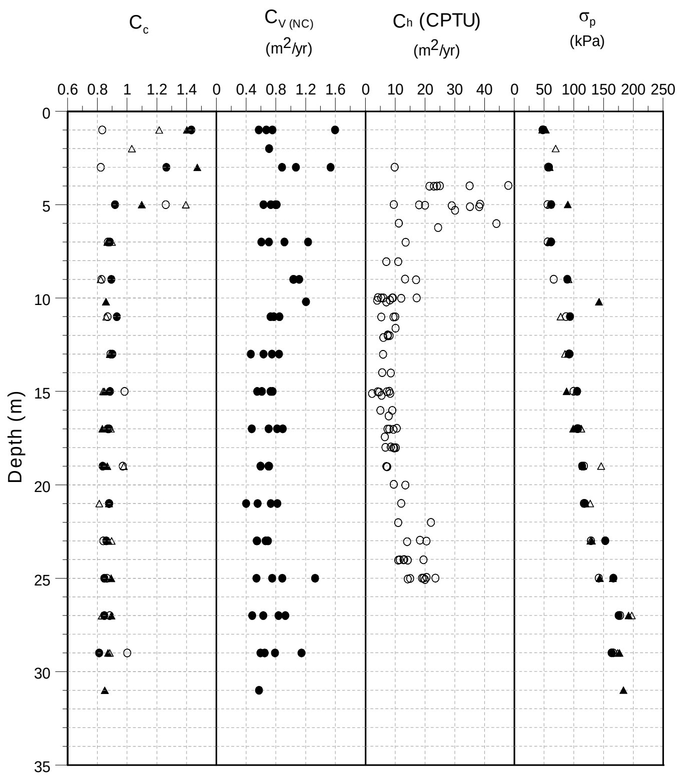

Figure 19b Deformation patameters at CMIT site

For Section I, the measured data of vacuum pressure in the sand blanket at various locations indicated that the effective vacuum pressure gradually reducing from 70 kPa at beginning to 55 kPa at EOP as seen in Figure 20. For Section II, the ground water table in the embankment during banking and waiting period was at the original elevation of the sand blanket. The settlement and rate of settlement versus time are plotted in Figure 21 and Figure 22 for Section I and II, respectively. The measured lateral displacements at toe of embankments are presented in Figure 23.

From measured settlement, S, and corresponding maximum lateral displacement, h, the stability control chart, S ~ h/S, was constructed for Section II in Figure 24. As seen in this figure, after reaching the maximum fill height, the values of q/q_f should decrease as increasing consolidation during waiting period but it continued to increase to unsafe side. Moreover, we have also found that the value of q/q_f approach 1 without any instability for many other actual cases. Thus, it seems that the Wakita and Matsuo plot is conservative for the case of embankment on PVD improved soft ground.

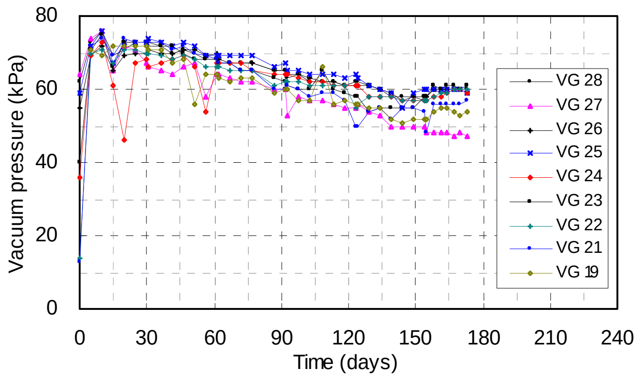

Figure 20 Measured vacuum pressure in sand blanket (CMIT project using VCM-MS method)

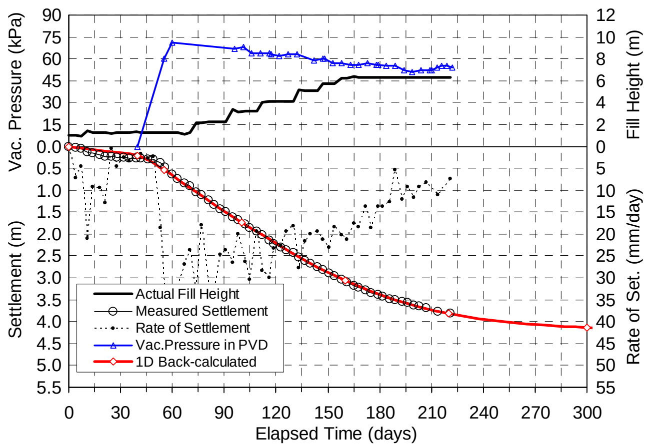

Figure 21 Measured settlement for PVD improved soft soil layer from 0 m to 35 m of Section I

6.1.1 Back calculation of compression index, C_c

From the above measured settlement data, Asaoka plots were obtained as seen in Figure 25 and Figure 26 for Section I and II, respectively. Assuming C_r = 0.2C_c and the linear relationship of compression index and natural water content, C_c = 0.016ω - C, as suggested by Bergado et al. (1996), based on the profile of ω versus depth, divide the soft ground to be many sub-layers with corresponding values of ω, then by trial and error for the value of C until matching the calculated final settlement by 1-D conventinal method with the S_f value from Asaoka plot. The back-calculated relation of C_c versus ω was obtained as presented by Eq. 36 that is plotted together with all oedometer test results as seen in Figure 27.

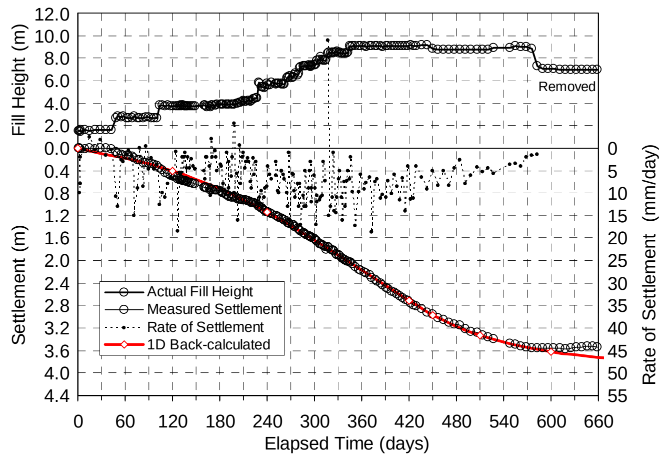

Figure 22 Measured settlement for PVD improved soft soil layer from 0 m to 34 m of Section II

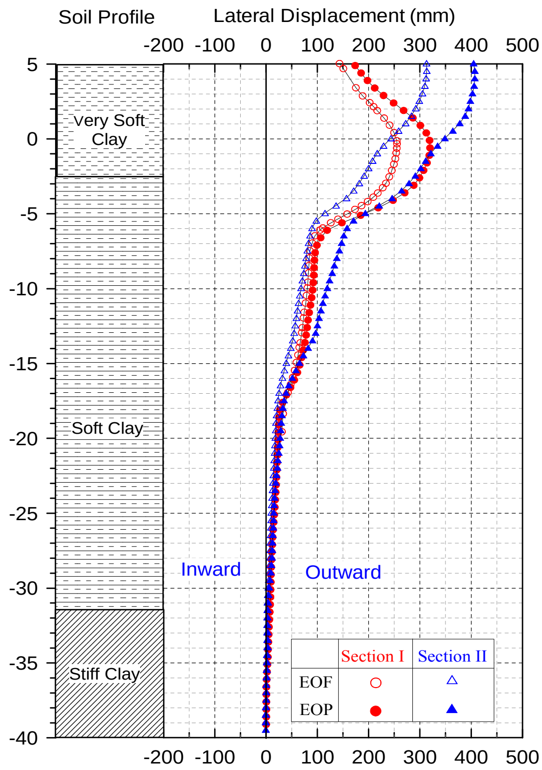

Figure 23 Measured lateral displacements of Section I and Section II at maximum fill height

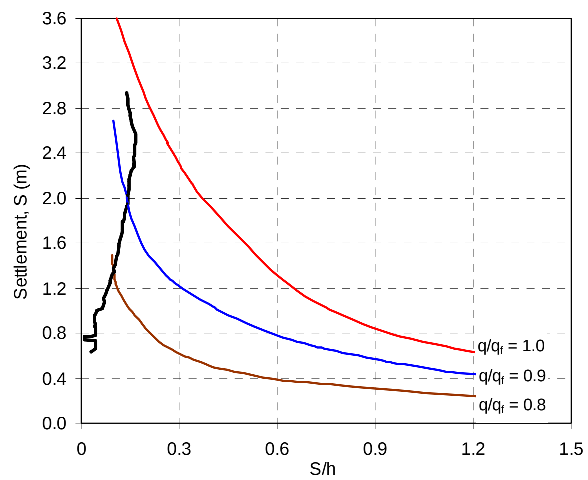

Figure 24 Stability control chart for Section II

$$C_c = 0.016\omega - 0.27 \quad (36)$$

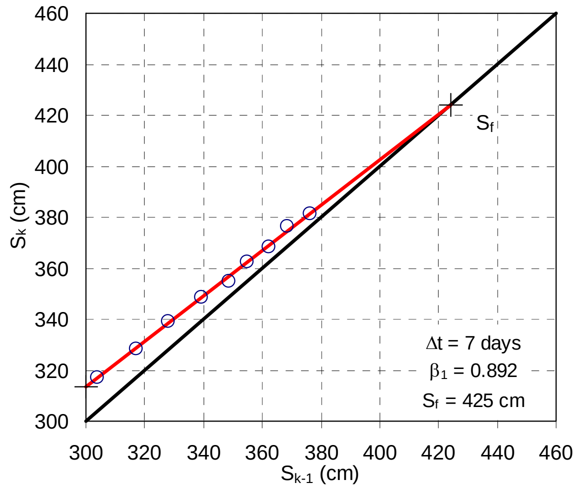

Figure 25 Asaoka plot for Section I

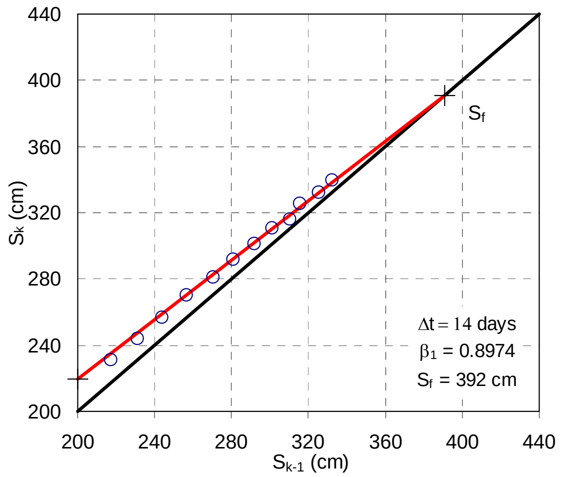

Figure 26 Asaoka plot for Section II

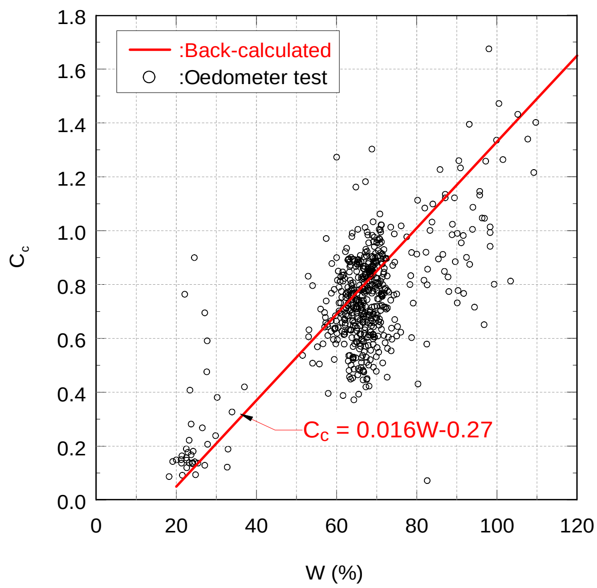

Figure 27 Relation of C_c versus ω for CMIT site

6.1.2 Back calculation of c_h value

From Eq. 30, with β_1 values from above Asaoka plots and taking the assumed field value of discharge capacity q_w = 100 m^3/year, back-calculated results for c_h values are presented in Figure 28 and Figure 29. Corresponding to d_s/d_m = 2 and $R_s = k_h/k_s$ = 2, the values of c_h are 4 m^2/year and 3.5 m^2/year for Section I and II, respectively. Using these c_h values with d_s/d_m = 2, R_s = 2, and C_c from Eq. (35), the calculated settlements versus time were plotted as the solid lines in Figure 21 and Figure 22 that are in excellent agreement with the measured data.

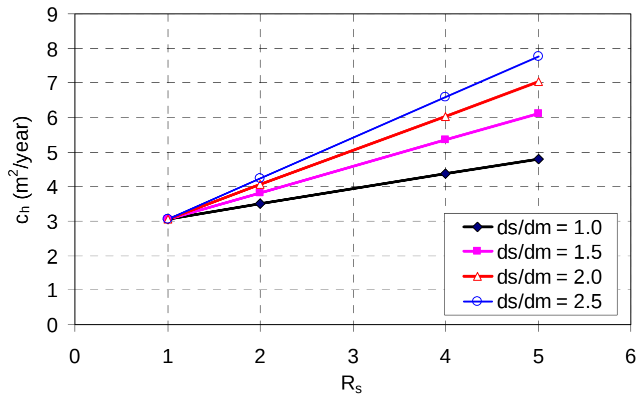

Figure 28 Back-calculated c_h for Section I - CMIT

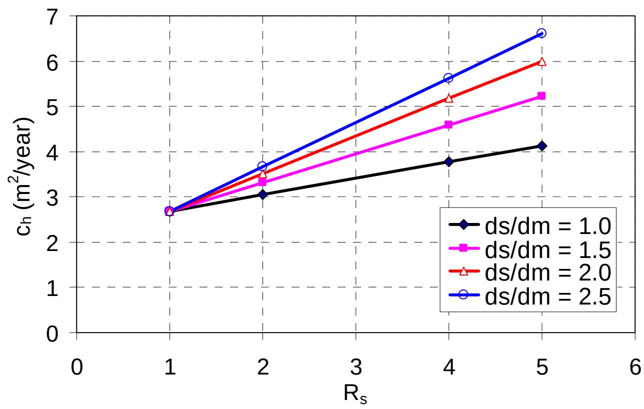

Figure 29 Back-calculated c_h values for Section II

6.2 North-South Expressway Project (NSEW)

NSEW was constructed in Dong Nai province (Figure 18). Four full scale Trial Sections using VCM with embankment fill preloading namely C1, C2, D1, and D2 were conducted in order to evaluate the suitable technique for mass construction. Soil profile and soil properties at the site are presented in Figures 30 and 31. PVDs were installed at spacing of 0.9 m in triangular pattern for all Sections using the mandrel with cross section of 70 mm x 140 mm. Direct tubing method without airtight membrane (VCM-DT) was used in C1 and C2 while airtight membrane with horizontal band drains (VCM-MB) was applied for D1 and D2. Surface settlement plates, extensometers, piezometers in soil and in PVD (at the mid depth) were installed for all Sections but inclinometers were installed for C1 and D2 only. For Section C1 and C2, measured vacuum pressure in PVD was lost at the measured settlement of about 1.8 m as seen in Figure 32. Measured vacuum pressure in PVD of Section D2 was also dramatically decreased while the vacuum pressure in D1 Section was stable at about 75 kPa until the end of preloading as plotted in Figure 33. From the results of these Trial Sections, VCM using membrane and flexible perforated tube (VCM-MT) was applied for mass construction of NSEW. Measured vacuum pressures at various locations during mass construction of NSEW are presented in Figure 34 indicating that the effective vacuum pressure in PVDs can be as high as about 80 kPa. Moreover, the effective vacuum pressure in PVDs during preloading period can be assumed as constant with depth and time.

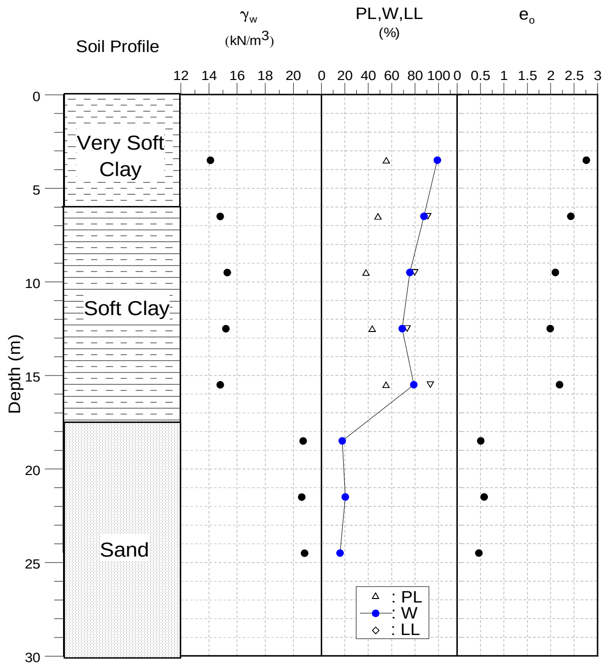

Figure 30 Basic properties at D1 Section – NSEW

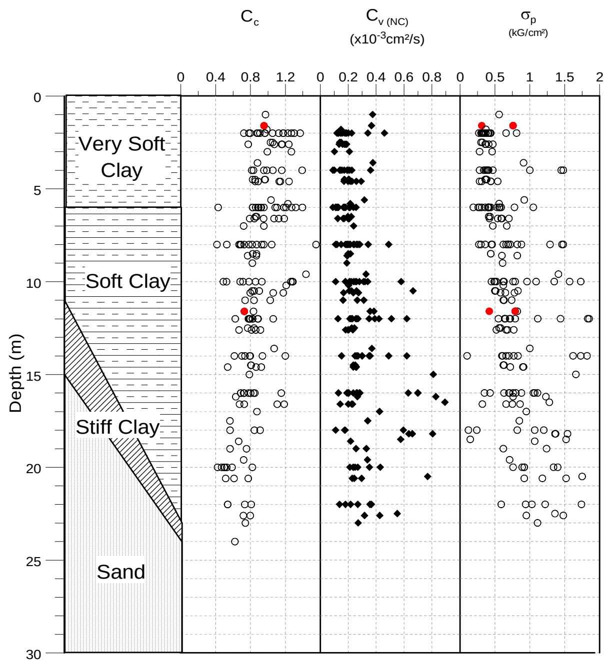

Figure 31 Compressibility properties of GU6-NSEW including D1 Section

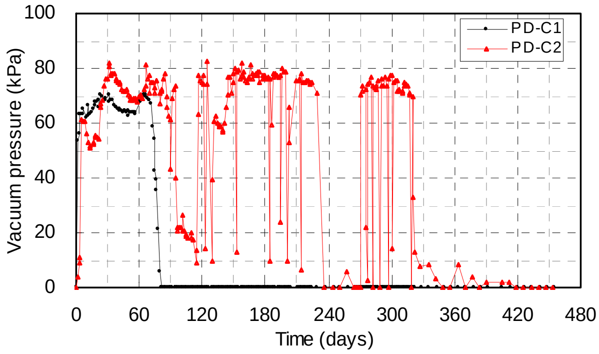

Figure 32 Measured vacuum pressure in PVD (C1 and C2 Sections using VCM-DT method)

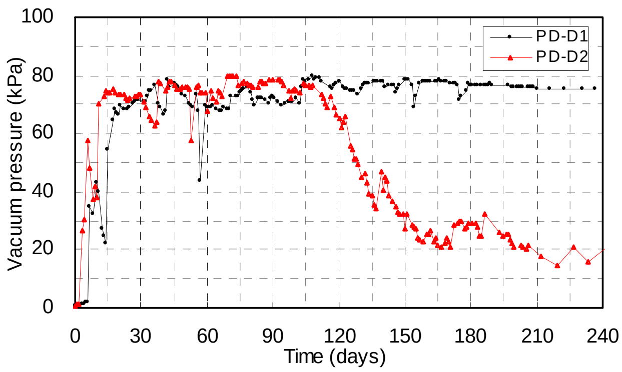

Figure 33 Measured vacuum pressure in PVD (D1 and D2 Sections using VCM-MB method)

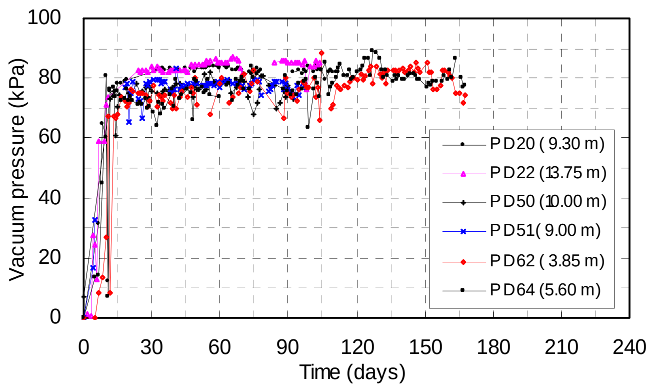

Figure 34 Measured vacuum pressure in PVDs during mass construction using VCM-MT technique

Lateral displacement profiles under the toe of embankment at the maximum fill height are presented in Figure 35 for C1 Section. From the figure, outward lateral dispacement can be observed. Measured settlement and rate of settlement at the center of Section D1 are plotted in Figure 36.

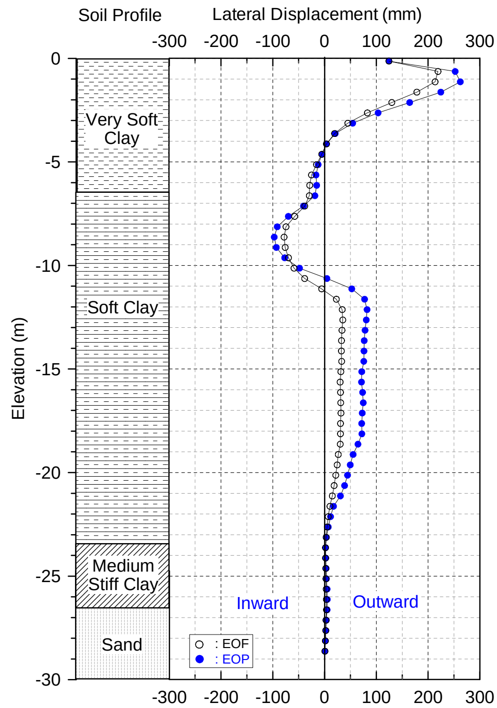

Figure 35 Lateral displacement profiles in Section C1 (membranless type using direct tubing)

Asaoka plot for final settlement and β_1 value of Section D1 are presented in Figure 37. Following the procedures as explained in Section 6.1, the back-calulated relation of $C_c \sim \omega$ for this site is $C_c = 0.016 - 0.413$ that is plotted together with oedometer test result in Figure 38. The back-analysis of c_h values are presented in Figure 39. Using the back-calculated values of c_h coresponding to $d_s/d_{.m} = 2$ and $k_h/k_s = 2$, the calculated settlement versus time is in very good comparison with observed data as seen in Figure 36.

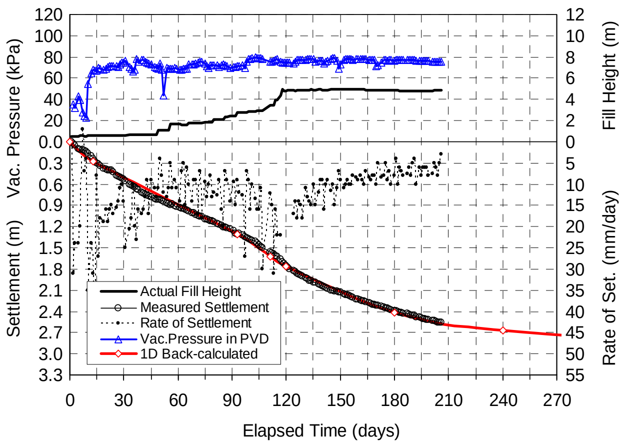

Figure 36 Settlement versus time of Section D1

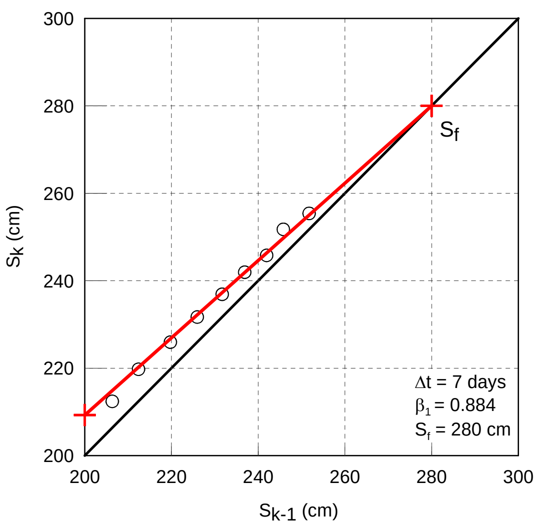

Figure 37 Asaoka plot for Section D1

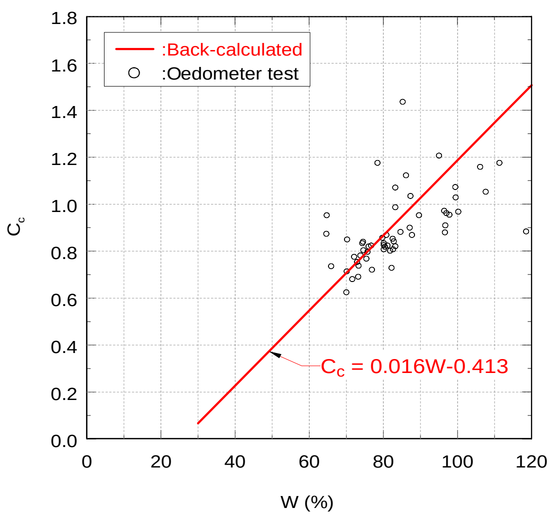

Figure 38 Relation of C_c versus ω for NSEW site

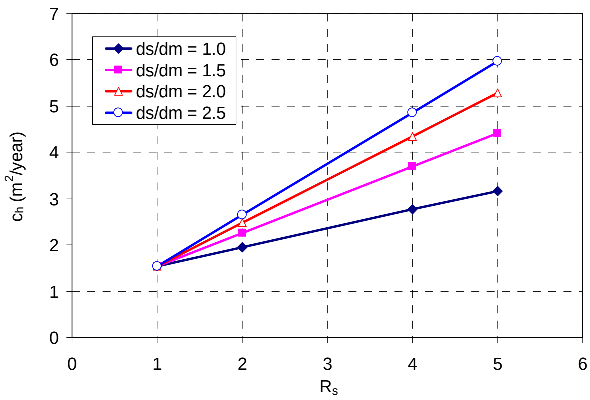

Figure 39 Back-calculated c_h value for NSEW site

6.3 PM3 Ca Mau Project (PM3-CM)

The project site is in Ca Mau province, about 300km from HCMC (Figure 18). Soil profile and soil properties are presented in Figures 40 and 41. The mandrel size of 70 mm x 140 mm was used to install the PVDs down to 15 m depth at spacing of 1.2 m in square pattern. High strength geotextile reinforcement with ultimate tensile strength of 200 kN/m was employed for increasing the factor safety during construction. Measured settlements at the center of embankment are plotted with time and embankment fill height as seen in Figure 42. Asaoka plot for final settlement and β_1 values is presented in Figure 43. Back-calculated results indicated the relation of $C_c = 0.016\omega - 0.27$ that is plotted together with oedometer test values as seen in Figure 44. Back-analyzed results of c_h are presented in Figure 45. Using the above back-calculated values of C_c and c_h value corresponding to $d_s/d_m = 2$ and $k_h/k_s = 2$, the calculated settlements versus time in comparison with measured data are presented in Figure 42.

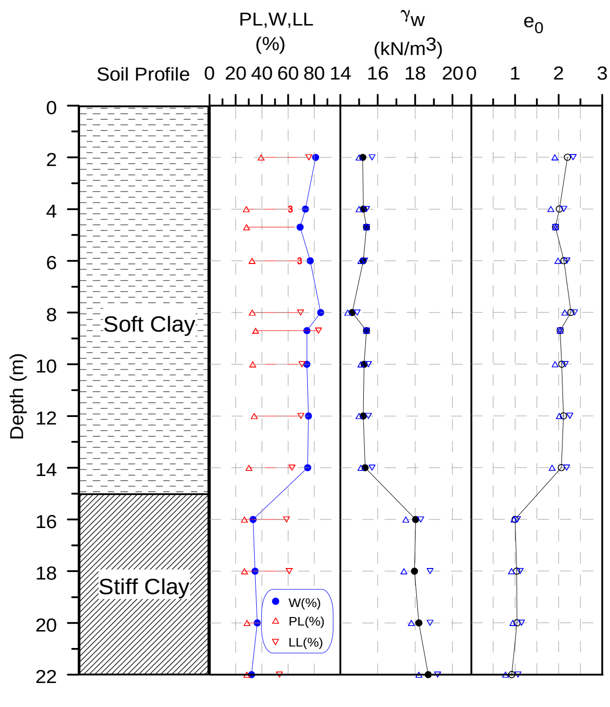

Figure 40 Basic properties of soft soil at PM3-CM site

6.4 Test Embankments at SBIA Site

Three full scale test embankments, namely: TS1, TS2 and TS3 were constructed on soft Bangkok clay using PVD with surcharge preloading at the site of the Second Bangkok International Airport (SBIA) in Thailand (Balasubramaniam et al., 1995). PVDs were

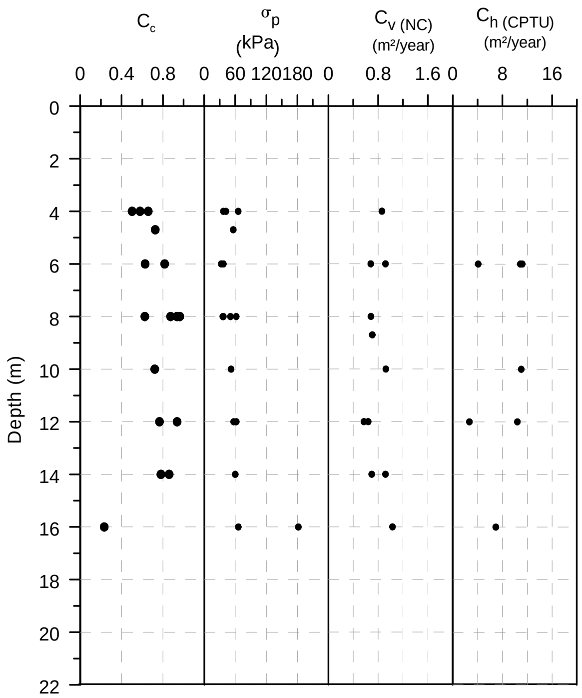

Figure 41 Compressibility and flow parameters of soft soils at PM3-CM site

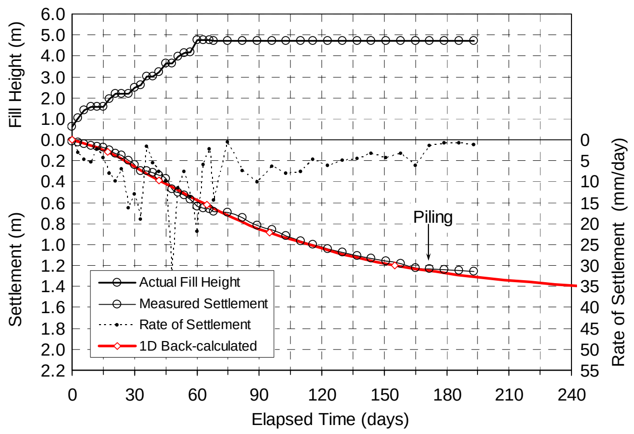

Figure 42 Settlements versus time for PM3-CM

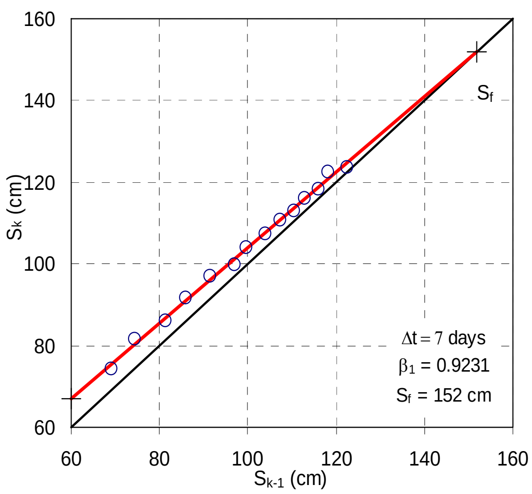

Figure 43 Asaoka plot for PM3-CM

installed down to the depth of 12 m in square pattern at the spacing of 1.5 m, 1.2 m, and 1.0 m for TS1, TS2, and TS3, respectively. Soil profile and soil properties are presented in Figures 46 and 47. Back-analyzed results by Bergado et al. (2002) indicated that the minimum c_h value of the weakest soil layer from 4m to 8 m depth is about 2.8 m^2/year with assumptions of d_s/d_m = 2 and k_h/k_s = 5 as seen in Figure 13.

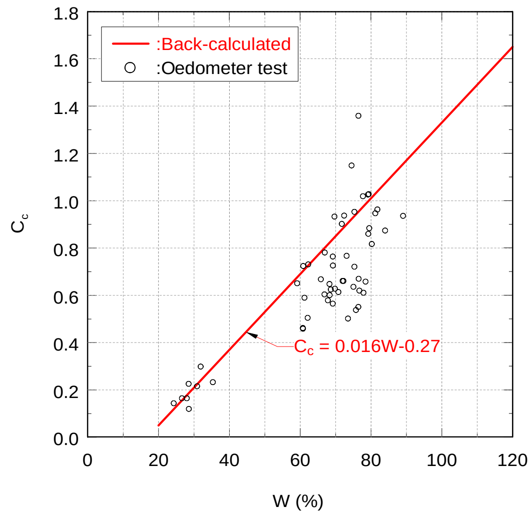

Figure 44 Relation between C_c versus ω for PM3-CM

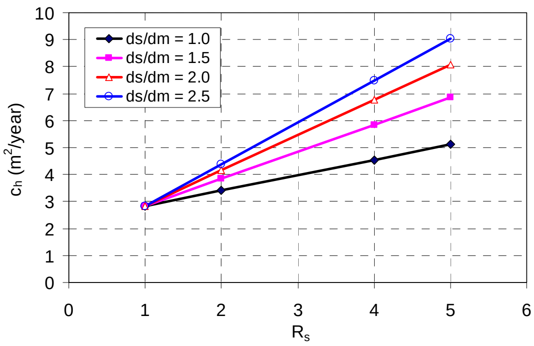

Figure 45 Back-calculated c_h values for PM3-CM

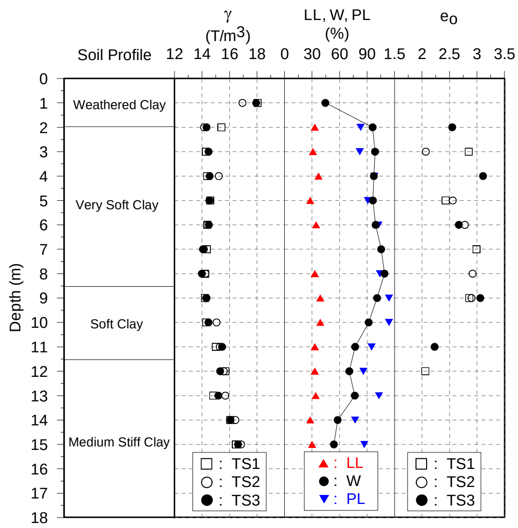

Figure 46 Basic properties of soft soils at SBIA

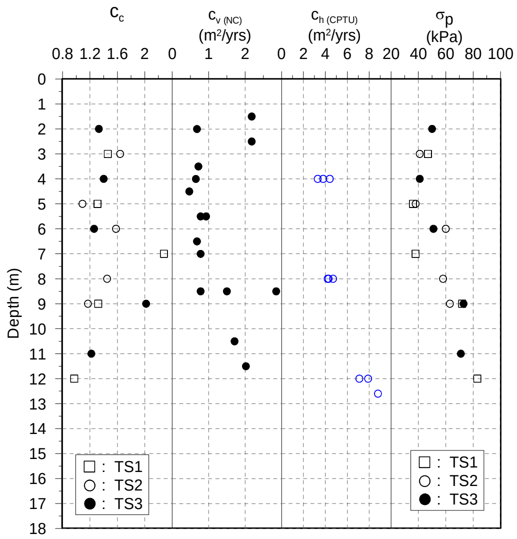

Figure 47 Compressibility parameters at SBIA site

Measured settlements at the centerline of TS3 (Figure 48) are re-analyzed for the average c_h value of the whole PVD improved soft soil layer from 0 m to 12 m. Asaoka plotted in Figure 49 gives the value of $\beta_1 = 0.852$. The back-calculated c_h values are presented in Figure 50 that is in accordance with the results obtained by Balasubramaniam et al. (1995) and Bergado et al. (2002).

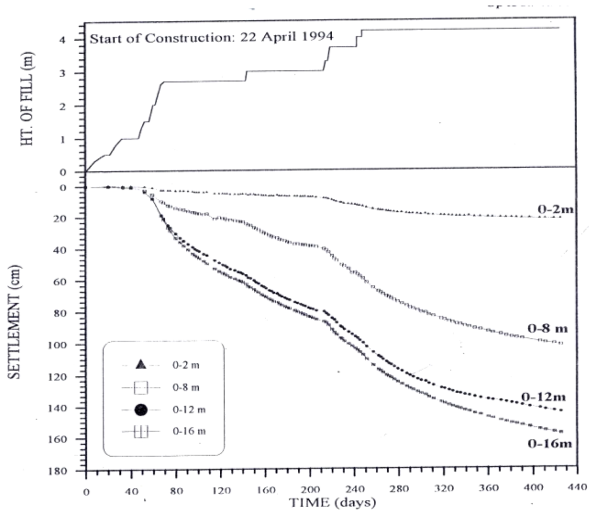

Figure 48 Measured settlements in TS3 – SBIA

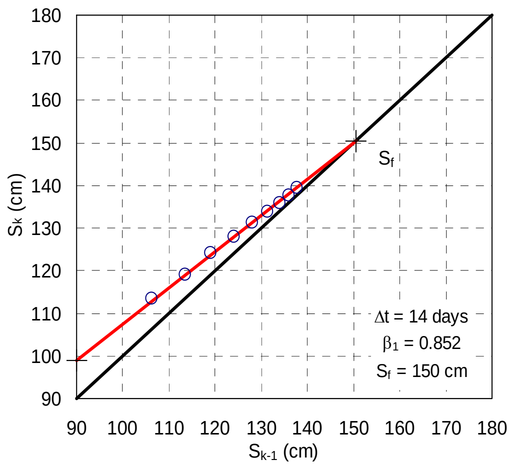

Figure 49 Asaoka plot for soil layer from 0 m to 12 m of TS3 – SBIA

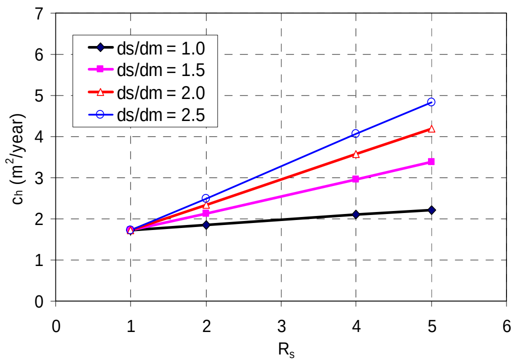

Figure 50 Back-calculated c_h values for SBIA site

6.5 Remarks on Back-calculated Values of C_c and c_h

6.5.1 Compression index, C_c

Back-calculated results from several case histories presented in above Sections indicated that the relationship between C_c and natural water content ω of soft clays in MRD and SDR can be expressed in Eq. 37 that is similar to that of soft BKK clays as introduced by Bergado et al. (1996).

$$C_c = 0.016\omega - 0.32 \ \pm 0.10 \tag{37}$$

6.5.2 Coefficient of consolidation, c_h

As disccussed in previous Sections, it is really difficult to obtain the field value of c_h at NC state. The back-anlyzed results are dependent on the assumed values d_s/d_m and k_h/k_s. Using $d_s/d_m = 2$ as commonly accepted, the relations between c_h and R_s from several case histories were presented. It can be seen that the back-calculated value of c_h is direct proportion with the assumed value of $R_s = k_h/k_s$. The values of c_h and $c_h/c_{v,oed}$ with $d_s/d_m = 2$ and $k_h/k_s = 1$, 2, and 5 are given in Table 7, 8, and 9 .

Table 7 Back-calculated c_h values with $k_h/k_s = 1$

Location	$c_{v,oed(NC)}$ (m²/year)	c_h (m²/year)	$c_h/c_{v,oed(NC)}$
CMIT	0.80	2.69	3.4
NSEW	0.66	1.54	2.3
PM3-CM	0.80	2.84	3.6
SBIA	0.70	1.63	2.3
Average	0.74	2.18	2.9

Table 8 Back-calculated c_h values with $k_h/k_s = 2$

Location	$c_{v,oed(NC)}$ (m²/year)	c_h (m²/year)	$c_h/c_{v,oed(NC)}$
CMIT	0.80	3.52	4.4
NSEW	0.66	2.48	3.8
PM3-CM	0.80	4.10	5.1
SBIA	0.70	2.53	3.6
Average	0.74	3.16	4.2

Table 9 Back-calculated c_h values with $k_h/k_s = 5$

Location	$c_{v,oed(NC)}$ (m²/year)	c_h (m²/year)	$c_h/c_{v,oed(NC)}$
CMIT	0.80	6.00	7.5
NSEW	0.66	5.29	8.0
PM3-CM	0.80	8.08	10.1
SBIA	0.70	5.24	7.5
Average	0.74	6.15	8.3

7. CONCLUSIONS

Soft ground improvement using prefabricated vertical drains including current practice of PVD installation and preloading techniques, settlement and stability calculations, observational methods and back analyses of monitoring data, and performance of several case of histories using conventional surcharge fill as well as combination of surcharge fill and vacuum pumping have been presented. Conclusions and recommendations can be made as follows:

i) For conventional preloading with embankment fill, horizontal drainage system with pumping wells is necessary for controlling the water table as expected in design calculations particularly for the case of preloading in rainy season and/or using hydraulically filling for large reclamation area.
ii) For VCM, the effective vacuum pressure of 60 kPa, 75 kPa, and 80 kPa have been observed corresponding to membrane type VCM methods VCM-MS, VCM-MB, and VCM-MT, respectively.
iii) Measured pore pressure inside PVDs indicated that the uniform distribution of effective vacuum pressure along the PVD depth can be assumed for practical design.
iv) Back-calculated results suggested that the values of $d_s/d_m = 2$, $k_h/k_s = 2$ and $c_h/c_{v,oed\,(NC)} = 3$ to 5 can be used for PVD improved soft Bangkok clay as well as for soft clays in MRD and SDR of Vietnam.
v) Back calculation of C_c from measured settlement data indicated that the linear relationship of $C_c = 0.016\omega - 0.32 \pm 0.1$ can be applied for soft clays in Southern Vietnam.
vi) The settlement during staged construction should be included in stability calculation of embankment on PVD improved soft ground. Moreover, it has been found that the stability control chart given by Wakita and Matsuo (1994) is conservative for PVD improved soft ground.
vii) 1-D conventional method can be applied well for design calculating the settlement of soft ground improved by PVD using VCM. The vacuum pressure can be taken into account as an induced stress, $\Delta\sigma_v$, distributed uniformly in the PVD zone.

8. REFERENCES

Asaoka, A., (1978) "Observational procedure of settlement prediction", Soil and Foundations, Vol. 18, No. 4, pp. 87–101.

Balasubramaniam, A.S., Bergado, D.T. and Phienwej, N. (1995) "The full scale field test of prefabricated vertical drains for the Second Bangkok International Airport (SBIA), Final Report", Div. of Geotech. and Trans. Eng. AIT, Bangkok, Thailand, pp 259.

Balasubramaniam, A.S., Cai, H., Zhu, D., Surarak, C., and Oh, E.Y.N. (2010) "Settlements of embankments in soft soils", Geotechnical Engineering Journal of the SEAGS & AGSSEA, Vol 41, No.2, pp 1-19.

Bergado D. T. , H. Asakami, M.C. Alfaro and A.S. Balasubramaniam, (1991) "Smear effects of vertical drains on soft bangkok clay". Journal of Geotechnical Engineering, ASCE, 117 No.10, pp1509–1530.

Bergado, D.T., Enriquez, A.S., Sampaco, C.L., Alfaro, M.C., and Balasubramaniam, A.S. (1992) "Inverse analysis of geotechnical parameters on improved soft Bangkok clay", J. Geotech. Eng'g. Div, ASCE, Vol. 118, No.7, pp1012-1030.

Bergado, D.T., Mukherjee, K., Alfaro, M.C., and Balasubramaniam, A.S. (1993) "Prediction of vertical band drain performance by finite element method" Geotextile and Geomembranes Journal., Vol. 12, No.6, pp567-586.

Bergado, D.T. and Long, P.V. (1994) "Numerical analysis of embankment on subsiding ground improved by vertical drains", Proc. Int'l. Conf. Soil Mech. and Found. Eng'g., New Delhi, pp1361-1366.

Bergado, D. T., Long, P. V. and Balasubramaniam, A. S. **(**1996**)** "Compressibility and flow parameters from PVD improved soft Bangkok clay", Geotechnical Engineering Journal, Vol. 27, No. 1, pp. 1-20.

Bergado, D. T., Balasubramaniam, A. S., Fannin, R. J. and Holta, R. D. (2002) "Prefabricated vertical drains (PVDs) in soft Bangkok clay: A Case Study of the new Bangkok International Airport Project", Canadian Geotechnical Journal. 39, pp304-315.

Chandler, R.J. (1988) "The in-situ measurement of the undrained shear strength of clays using the field vane: SOA paper. Vane Shear Strength Testing in Soils Field and Laboratory Studies", ASTM STP 1014, pp13-44.

Giao, P.H., Dung, N.T., Long, P.V. (2008) "An integrated geotechnical geophysical investigation of soft clay at a coastal site in the Mekong delta for oil and gas infrastructure development" Canadian Geotech. Journal, 45, pp1514–1524.

Hansbo, S. (1960) "Consolidation of clay with special reference to influence of vertical sand drains", Swedish Geotechnical Institute, Proc., Issue 18, pp160.

Hansbo, S. (1979) "Consolidation of clay by band-shaped prefabricated drains", Ground Eng., Volume 12, Issue 5, pp16-25.

Hansbo, S. (1981) "Consolidation of fine-grained soils by prefabricated drains", Proc. 10th Int. Conf. SMFE., Stockholm, Volume 3, 1981, pp677-682.

Hansbo, S. (1987) "Design aspects of vertical drains and lime column installation". Proc. 9th Southeast Asian Geotechnical Conference, Volume 2, Issue 8, pp1-12.

Hausmann, M.R. (1990) Engineering Principal of Ground Modification, McGraw-Hill Publishing Company, p631.

Indraratna, B., and Redana, I. W. (1998) "Laboratory determination of smear zone due to vertical drain installation". J. Geotech. Eng., ASCE, 125 No. 1, pp 96-99.

Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I. (2005) "Analytical and numerical solutions for a single vertical drain including the effects of vacuum preloading". Canadian Geotechnical Journal, 42, pp994-1014.

Jamiolkowski, M., Lancellotta, R., and Wolski, W. (1983) "Precompression and speeding up consolidation". Proc. 8th ECSMFE, pp1201-1206.

Ladd**,** C.C. and DeGroot D.J. (2003) **"**Recommended practice for soft ground site characterization: Arthur Casagrande Lecture", 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering**,** MIT, USA June 22 – 25, 2003**.**

Ladd, C. C. (1991) "Stability evaluation during staged construction", ASCE Journal of Geotechnical Engineering Division, Vol. 117, No. 4, pp540-615.

Long, P.V., Bergado, D.T., Giao, P.H., Balasubramaniam, A.S., and Quang, N.C. (2006) " Back analyses of compressibility and flow parameters of PVD improved soft ground in southern Vietnam". Proc. Of the 8th International Conference on Geosynthetics, Yokohama 2006, Vol. 2, pp465-468.

Mesri, G., Lo, D.O.K, and Feng, T.W., (1994) "Settlement of embankment on soft clay". Keynote Lecture, Geotechnical Special Publication, Issue 40, 1, Texas A and M University, College Station, Texas, 1994, pp8-56.

Mersri, G and Castro, A. (1987) "C_α/C_c concept and K_0 during secondary compression", Journal of Geotechnical Engineering, ASCE, 112 No.3, pp230-247.

Seah, T.H., Tangthansup B., and Wongsatian, P. (2004) "Horizontal coefficient of consolidation of soft Bangkok clay", Geotechnical Testing Journal, ASTM, Volume 27, Issue 5, pp430-440.

Tan, T.S., Inoue, T., and Lee, S.L. (1991) "Hyperbolic method for consolidation analysis", Journal of Geotechnical Engineering, Vol.117, No.11, pp1723-1737.

Wakita, E. and Matsuo, M. (1994) "Observation design method for earth structures constructed on soft ground, Geotechnique, 44 No.4, pp747-755.