

# Performance and Analyses of Thick Soft Clay Deposit Improved by PVD with Surcharge Preloading and Vacuum Consolidation – A Case Study at CMIT

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**ABSTRACT:** Ground improvement using PVD was applied successfully for increasing foundation stability and controlling residual settlements of the container yard constructed on 35 m thick soft clay deposit at CMIT, Vietnam. The treated area is about 40 ha including vacuum consolidation combined with 6.3 m embankment surcharge for a strip of 57 m along the river bank (VCA) and conventional surcharge preloading using 9.1 m sand fill embankment for the remaining area. The monitored data indicated that PVD thickness of 3 mm arranged in spacing of 0.9 m to 1.2 m can be used successfully for improvement of thick soft clay deposit in both methods of embankment preloading with and without vacuum pumping. Performance of reduced embankment combined with vacuum pumping is very much better than that of conventional embankment preloading in terms of shortening construction time, reducing lateral displacement, increasing stability, and minimizing residual settlement. Back calculated  $c_h$  value is dependent on the assumptions of smear effects including smear zone ratio,  $d_s/d_m$  and permeability ratio,  $R_s = k_h/k_s$ . For  $d_s/d_m = 2$  as commonly used, the back-calculated  $c_h$  value is directly proportional to  $R_s$  and the value of  $R_s$  in vacuum consolidation seems significantly smaller than that in embankment preloading. Using the back-calculated results of compressibility and flow parameters, the time-settlements re-calculated by 1-D method are in very good comparison with measured data for both conventional preloading and vacuum consolidation considering the vacuum pressure as an induced vertical stress distributed uniformly in the PVD zone. Analyses of factor of safety from observed pore pressures during embankment construction illustrated that the commonly used stability chart as given by Wakita & Matsuo (1994) is too conservative for PVD improved soft ground. Secondary compression behavior of thick soft ground improved by PVD including back calculation for coefficient of secondary compression and estimation of long term residual settlement have also been provided.

**KEYWORDS:** Settlement, Soft clay, Ground improvement, PVD, Vacuum consolidation

## 1. INTRODUCTION

Preloading soft clay deposits for increasing stability and controlling post construction settlements using PVD has been extensively applied (Balasubramaniam et al., 1995; Bergado et al., 1996, 1998, 2002; Chu et al., 2000; Seah et al., 2004; Yan and Chu, 2005; Kelly and Wong, 2009; Rujikiatkamjorn and Indraratna, 2007, 2009, 2013; Indraratna et al., 2005, 2011, 2012; Artidteang et al., 2011; Geng et al., 2012; Chai et al., 2008, 2013a, 2013b; Voottipruex et al., 2014; Long et al., 2006, 2013, 2015). However, further investigations on problems related to practical design such as assumptions on smear effects and flow parameters, distribution of vacuum pressure along PVD length, performance of PVD in thick soft clay deposit including thickness and spacing of PVDs and residual settlement of PVD improved ground are still needed. One of the largest port project, namely CMIT, has been constructed on thick soft clay deposit in Southern Vietnam. The project site is about 45 km from Ho Chi Minh City. The treated area is about 40 ha including vacuum consolidation combined with 6.3 m embankment surcharge for 58 m strip along the river bank and conventional surcharge preloading with 9.1 m sand fill for the remaining area. PVD thickness of 3 mm, PVD length up to 35 m, and PVD spacing of 0.9 m to 1.2 m were applied. Site conditions, construction and instrumentation, monitored results, and back analyses for smear effects, compressibility and flow parameters, time-settlements and stability during preloading, and secondary compression after construction are presented in following sections.

## 2. SOIL CONDITIONS

The project is located in low land area along the right bank of Thi Vai River. Natural ground surface of the site is at elevation of about +2.5 m to +3.5 m CDL (chart datum level) that is lower than the high tide level. The deepest elevation of river bed is about – 20.0 m CDL and the side slope of river bank is about 1V by 10 H. Tidal water level at this area varies from +4.5 m CDL to +2.5 m CDL corresponding to high tide and low tide, respectively.

Soil profile of the site is presented in Fig. 2, consisted of following sub-soil layers:

**Layer 1a:** From ground surface to the depth of about 6 m to 8 m, bluish/dark gray color, very soft to soft high plasticity organic clay (OH), occasionally parting of fine grained sand and mollusca shell debris, trace of some very stiff yellowish grey clay lumps. The average values of water content are of 92 %, plastic limit of 35%, liquid limit of 80%, unit weight of 14.5 kN/m<sup>3</sup>.



Fig. 1 Location of the project

**Layer 1b:** Underlying layer 1a to the depth of about 32m to 38 m, soft bluish grey high plasticity clay (CH-MH), with some lumps of very stiff yellowish grey clay. The average values of water content are of about 66 %, plastic limit of 35 %, liquid limit of 72 %, unit weight of 15.8 kN/m<sup>3</sup>, SPT-N values from 1 to 2.

**Layer 2 :** Underlying layer 1b to the depth of about 36 m to 44 m, stiff to very stiff sandy clay (CL), dark gray color, average values of water content of 24 % to 40 %, plastic limit of about 20%, liquid limit from 40% to 60%, unit weight of 18 to 20 kN/m<sup>3</sup>, SPT-N values ranging from 8 to 22.

**Layer 3 :** Underlying layer 2 to the depth of 40 m to 52 m, fine to medium sand with silt and some gravels (SM/SP), gray, whitish gray color, medium to dense state, SPT-N value from 12 to 39.

The engineering properties of foundation soils consisted of wet unit weight,  $\gamma_w$ , natural water content,  $\omega$ , initial void ratio,  $e_0$ , plasticity index, PI, compression index,  $C_c$ , pre-consolidation pressure,  $p_c$ , secondary compression ratio,  $C'_\alpha$ , coefficient of vertical consolidation at normally consolidated state from conventional oedometer tests,  $c_{v(NC)}$ , coefficient of horizontal consolidation from CPTu tests,  $c_h(CPTu)$ , undrained shear strength from field vane shear tests,  $S_u(FVT)$ , and SPT-N values in typical bore holes (CB4, CB5, CB7, CB8, BH 15) are plotted with depth in Fig. 3. The average ground water level is about +3.5 m CDL. All laboratory consolidation tests were performed following ASTM-D2435 standard test method for undisturbed samples taken by 91 mm outer diameter thin-walled tube sampler with piston head.

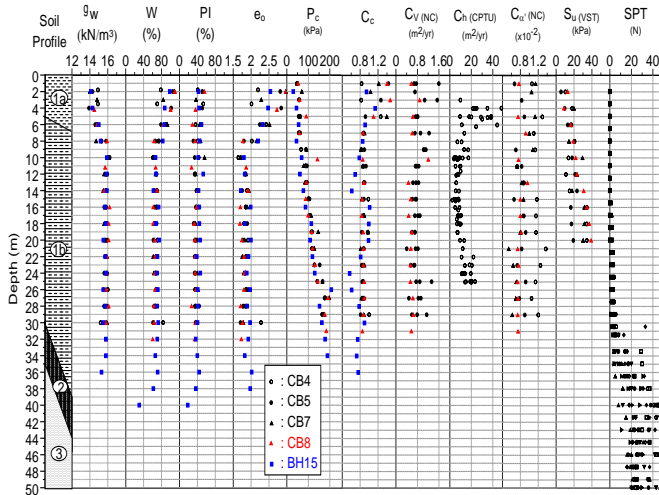


Fig. 3 Soil properties from boreholes CB4, CB5, CB7, CB8, BH15

From Fig. 3, the pre-consolidation pressure,  $p_c$ , can be expressed as function of depth as below:

$$p_c = 45 \text{ kPa} \quad \text{for } z \leq 5 \text{ m} \quad (1)$$

$$p_c = 5.3(z - 5) + 45 \quad \text{for } z > 5 \text{ m} \quad (2)$$

where  $z$  is the depth from ground surface in meter and  $p_c$  in kPa.

The value of coefficient of consolidation in horizontal direction,  $c_h$ , from CPTu tests varies from 10 m<sup>2</sup>/yr to 40 m<sup>2</sup>/yr for the upper most soft clay (layer 1a) and from 4 m<sup>2</sup>/yr to 20 m<sup>2</sup>/yr for sub-soil layer 1b. The average value of coefficient of consolidation in vertical direction at NC state from conventional oedometer test,  $c_{v(NC)}$ , is about 0.87 m<sup>2</sup>/year.

Typical consolidation curves from conventional oedometer test of soft clay at various depths are presented in Fig. 4. The values of compression index,  $C_c$ , from all available oedometer tests are plotted as function of natural water content in Fig. 5 together the empirical correlation given by Long et al. (2013). The large variation of  $C_c$

values can be explained as mainly due to specimen disturbance (Ladd and DeGroot, 2003). The values of secondary compression ratio  $C'_\alpha$  and coefficient of vertical consolidation  $c_v$  from oedometer tests are plotted with normalized loading pressures,  $p/p_c$ , in Fig. 6 and Fig. 7, respectively. From these figures, it can be seen that the value of secondary compression ratio in normally consolidated state,  $C'_\alpha(NC)$ , is about 5 to 10 times higher than that at over-consolidated state,  $C'_\alpha(OC)$  and the value of coefficient consolidation at OC state,  $c_v(OC)$ , is higher than that in NC state,  $c_v(NC)$  of more than 2.5 times.

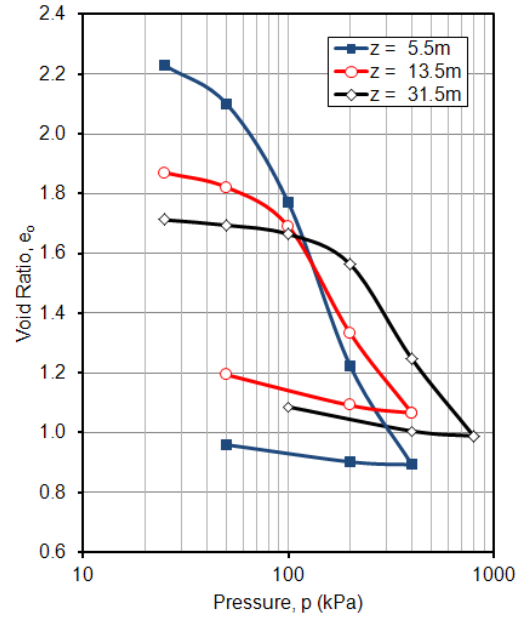


Fig. 4 Consolidation curves of soft clay

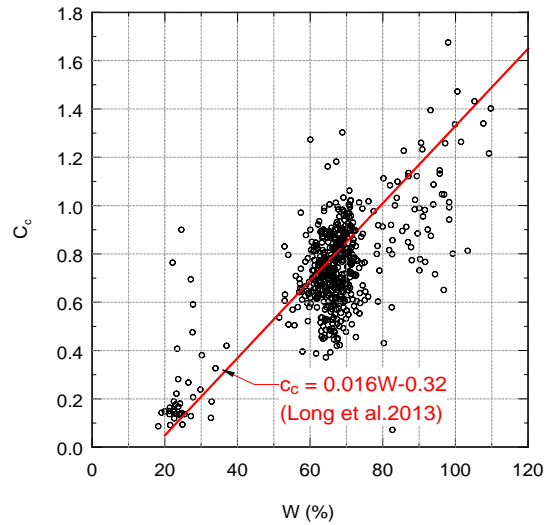


Fig. 5 Compression index  $C_c$  versus natural water content  $\omega$

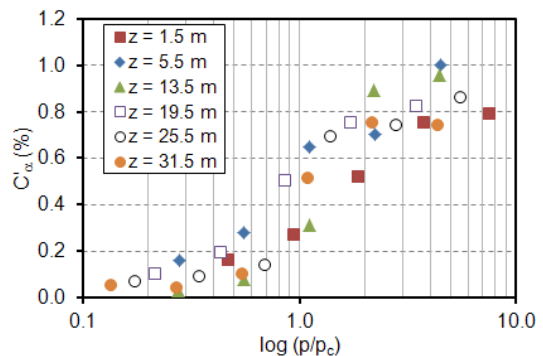


Fig. 6 Relation between  $C'_\alpha$  and normalized loading pressure  $p/p_c$

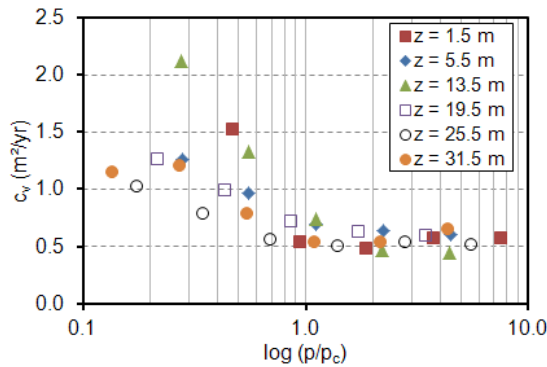


Fig. 7 Relation between  $c_v$  and normalized loading pressure  $p/p_c$

### 3. CONSTRUCTION AND INSTRUMENTATION

#### 3.1 Construction

In order to build a container yard on soft ground with finished elevation at +6.3 m CDL, reclamation with soft ground improvement on an area of 600 m x 660 m was conducted. Layout of the project is presented in Fig. 8 and typical cross sections are given in Fig. 9a,b. Due to heavy container load with considering the secondary compression of thick soft clay deposit, the design criteria were set out as follows: i) exposed load on pavement surface is 40 kPa in average; residual settlements shall be smaller than 50 cm in the first 3 years and 120 cm in 50 years after construction; and the slope of pavement surface due to differential settlements between any points must be smaller than 1.5%. Soft ground improvement using PVD consists of surcharge combined with vacuum pumping for a strip of 57 m along the river bank and embankment preloading for the remaining area. Conventional vacuum consolidation using airtight membrane and sand blanket was applied. Construction procedures can be summarized as follows:

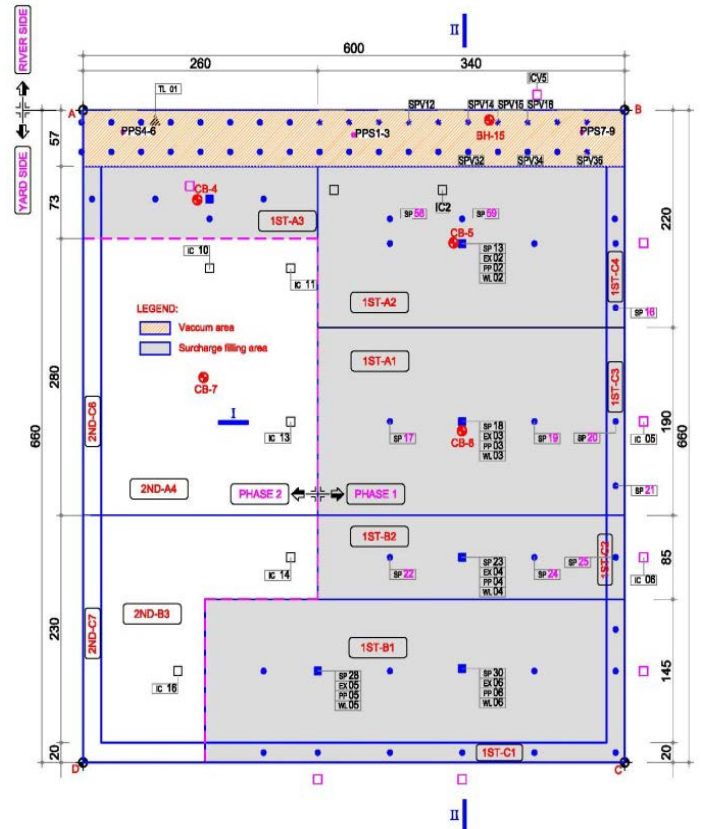
- Site clearance and filling up to elevation of +4.5 m CDL by fine silty sand.
- Filling with medium to coarse sand for sand blanket from +4.5 m CDL to +5.5 m CDL.
- Installation of PVDs in triangle pattern at spacing of 1.0 m for vacuum area, 0.9 m and 1.2 m for surcharge preloading areas (Fig. 9a, b).
- Installation of airtight membrane for vacuum area.
- Filling with fine silty sand for embankment preloading, the wet and saturated unit weight of embankment fill after compaction is about 18 kN/m<sup>3</sup> and 19 kN/m<sup>3</sup>, respectively.

PVD type of 3 mm thickness fully penetrated to the bottom of soft clay layer 1b using the mandrel of 70 mm x 140 mm was employed for all sections. Vacuum consolidation method using sand blanket covered by airtight membrane (VCM-MB) was applied for vacuum section.

#### 3.2 Instrumentation

Plan view and cross sections of monitoring devices can be seen in Fig. 8 and Fig. 9, consisted of:

- Settlement plates (SP) at ground surface
- Extensometers (EX) at various depths
- Inclinometers (IC) at the toe of embankment
- Piezometers (PP) at various depths, installed in between PVDs for measurement of total pore pressure in the treated soft ground.
- Vacuum gauges, VG, beneath the airtight membrane for monitoring effective vacuum pressure,  $p_{vac}$ , in the sand blanket of vacuum area.
- After removing surcharge and completion of pavement, settlement monitoring points were installed on the pavement surface for observation of post construction settlements.





removing surcharge ( $t = 570$  days) was about 3.9 m corresponding to degree of consolidation (DOC) of about 90 %.

Embankment fill height, vacuum pressure measured in sand blanket, and corresponding settlements at numerous settlement plates (SP-V15 to SP-V36) are presented in Fig. 11 for vacuum consolidation area (VCA). With 6.3 m fill height combined with vacuum pumping, the average settlement measured at the time of removing surcharge ( $t = 220$  days) was of about 4.2 m corresponding to DOC of about 90%. Thus, reduced embankment combined with vacuum pumping can reduce significantly the construction time.

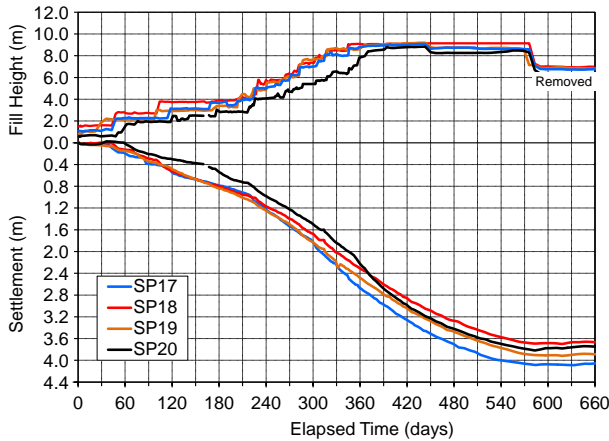


Fig. 10 Surface settlements of surcharge preloading area (SPA)

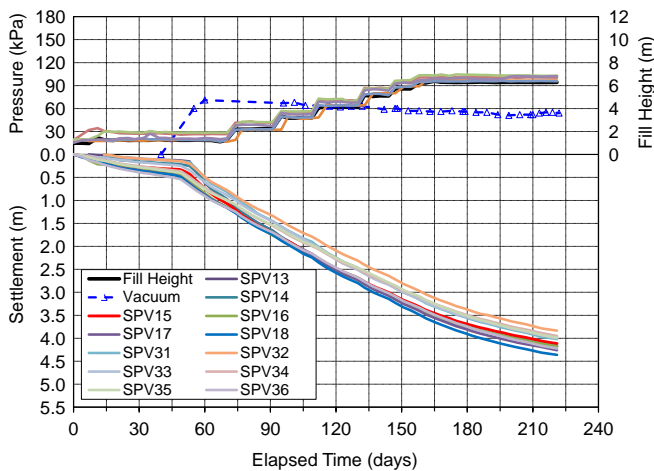


Fig. 11 Surface settlements of vacuum area (VCA)

Sub-surface settlements at various depths in SPA and VCA are presented in Fig. 12 and Fig. 13, respectively. From these figures, it can be seen that PVDs were used effectively to the depth of 35 m.

#### 4.2 Lateral displacement

Lateral displacements under the toe of embankment measured at IC-05 in SPA and IC-V05 in VCA are plotted with depths in Fig. 14. From this figure, it can be seen that the lateral displacements at the end of filling (EOF) and at the end of preloading (EOP) in VCA are smaller than that in SPA even though no counterweight berms in VCA. No inward lateral displacements were observed at the toe of vacuum area that can be explained as the combined influence of high embankment load in land side together with low overburden stress in river side of VCA.

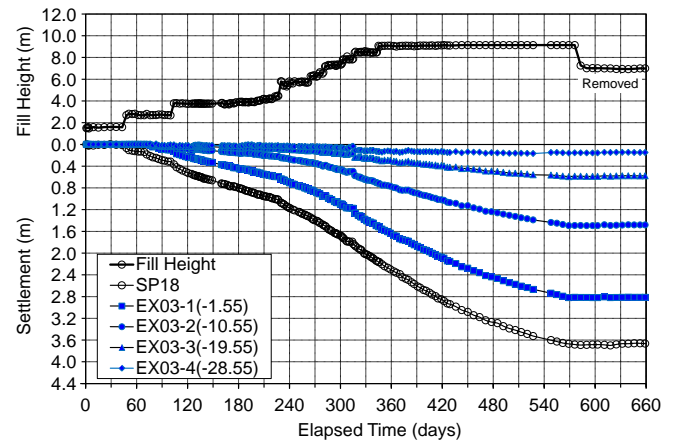


Fig. 12 Settlements at various depths of SPA

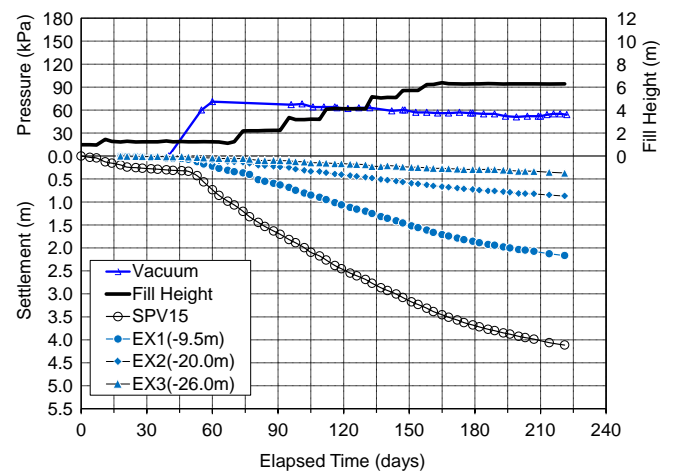


Fig. 13 Settlements at various depths of VCA

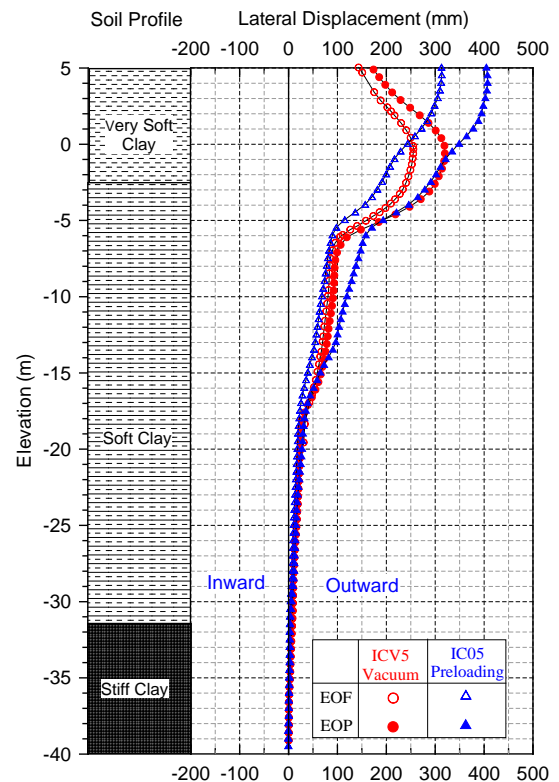


Fig. 14 Lateral displacements at the end of filling (EOF) and at the end of preloading (EOP) in VCA (IC-V5) and in SPA (IC-05)

Settlement,  $S$ , versus ratio of lateral displacement to settlement,  $d/S$ , at numerous locations were plotted in the stability chart (Wakita & Matsuo, 1994) in Fig. 15. In this chart, the shear stress ratio,  $q/q_f$ , can be considered as the inverse value of factor safety,  $FS$ . Some problems of this stability chart can be seen as follows:

- The values of  $q/q_f$  of about 0.94 to 1.0 (corresponding to  $FS = 1.06$  to 1.0) were obtained for all locations but no any signals of slope instability were observed during construction.
- During consolidation period after the end of filling, the lateral displacement ratio,  $d/S$ , decreased with increase of settlement but the values of factor of safety tend to decrease instead of increase with consolidation process.

Therefore, it might be concluded that this type of stability chart is too conservative for embankments on PVD improved soft ground.

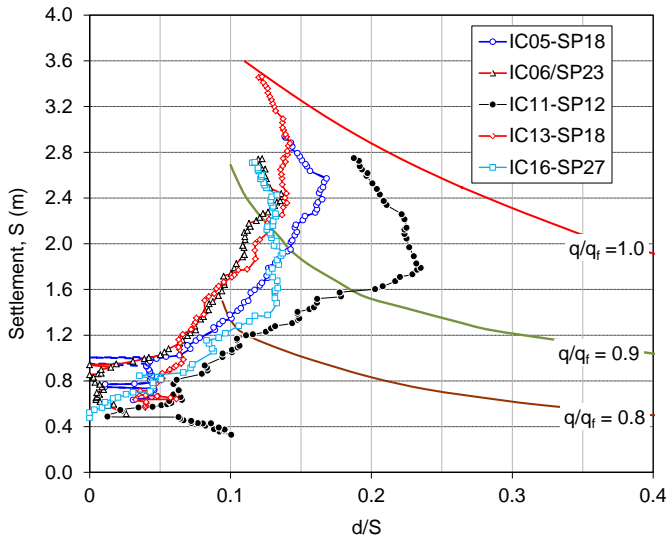


Fig. 15 Settlement,  $S$ , versus lateral displacement ratio,  $d/S$

### 4.3 Pore pressure and Vacuum pressure

Excess pore water pressures in soft ground of SPA at elevations of -1.55 m, -10.55 m, -19.55 m and -28.55 m are presented in Fig. 16. The higher rate of pore pressure dissipation at shallow depth (PP03-1) can be explained as the combined effects of horizontal and vertical drainage (into the sand blanket) while the rate of pore pressure dissipation at -10.5 m and at -28.5 m is almost the same indicated that PVD type of 3 mm thickness can be used effectively for thick soft ground deposits.

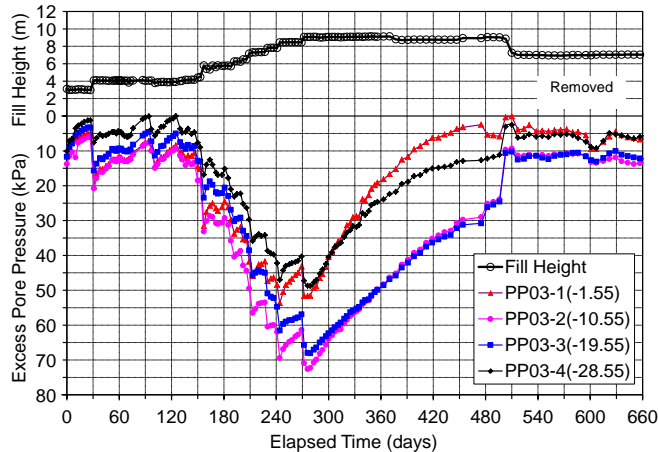


Fig. 16 Excess pore water pressures in SPA at various elevations of -1.55 m, -10.55 m, -19.55 m, and -28.55 m

Excess pore water pressures in soft ground at different depths of -3.0 m, -15.5 m, and -28.0 m in VCA are plotted with time in Fig. 17. Excess pore pressures from PP-S9 at -29.0 m were quickly dissipated that may be due to the factual distance between this piezometer tip to the adjacent PVD was too close and/or the connection of PVD with sand lenses. From the rate of pore pressure dissipation at various depths in this figure, the effectiveness of vacuum consolidation method using 3 mm thick PVDs for thick soft clay deposits can be confirmed.

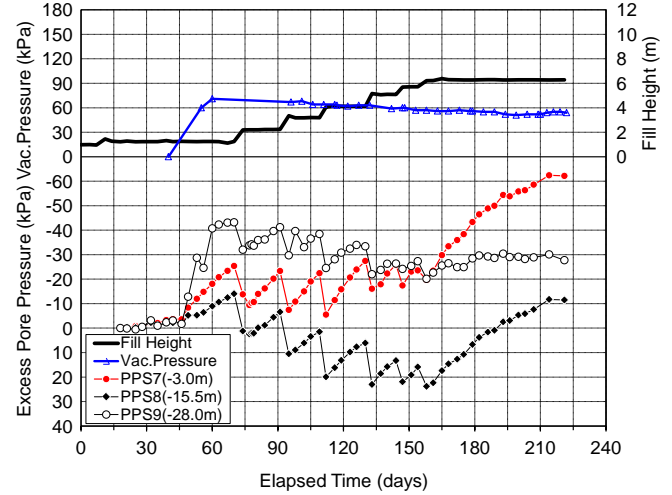


Fig. 17 Excess pore water pressures in VCA at various elevations of -3.0 m, -15.5 m, and -28.0 m

Vacuum pressures in sand blanket measured at numerous locations beneath the airtight membrane are presented in Fig. 18. It can be seen from this figure that the maximum vacuum pressure of about 75 kPa was achieved at early stage and then gradually decreased to about 55 kPa at the end of preloading.

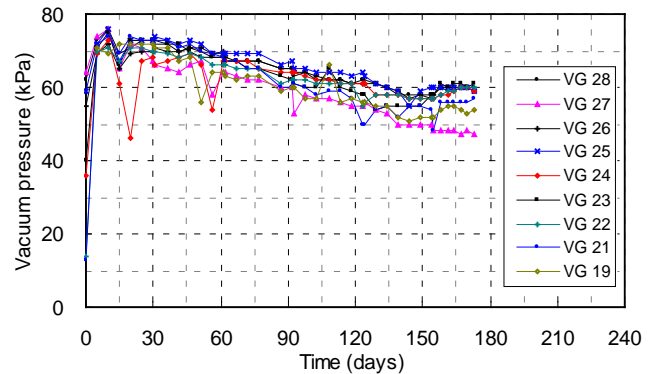


Fig. 18 Vacuum pressures measured from vacuum gauges (VG)

## 5. ANALYSES

### 5.1 Settlement analyses

#### 5.1.1 Final primary settlement

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

$$S_{oed} = \sum h [RR \cdot \log(\sigma'_p / \sigma'_{v0}) + CR \cdot \log(\sigma'_{vf} / \sigma'_p)] \quad (3)$$

where  $h$  is thickness of the calculated sub-soil layer,  $CR$  and  $RR$  are compression and re-compression ratio,  $\sigma'_{v0}$  is existing overburden,  $\sigma'_p$  is pre-consolidation stress, and  $\sigma'_{vf}$  is the 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:

$$S_{cf} = \mu_c S_{oed} \text{ and } S_i = (1 - \mu_c) S_{oed} \quad (4)$$

where  $\mu_c$  can be taken as 1 to 0.8 for soft marine deposits 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. (3) and Eq. (4) with the value of  $\sigma'_{vf}$  in Eq. (3) should be determined as below (Long et al., 2013):

$$\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma_v + (u_0 - u_f) \quad (5)$$

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 surface,  $u_0$  is initial pore pressure (just before embankment construction), and  $u_f$  is the final pore pressure can be taken as follows:

- For settlement due to service load,  $u_f$  = pore water pressure at the end of project life time
- For estimating settlement during preloading,  $u_f = u_0$

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.

### 5.1.2 Consolidation with PVD

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

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

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:

$$U_h = 1 - \exp(-8T_h/F) \quad (7)$$

$$T_h = c_h t / d_e^2 \quad (8)$$

$$F = F_n + F_s + F_r \quad (9)$$

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

$$F_s = (k_h/k_s - 1) \log_e(d_s/d_w) \quad (11)$$

$$F_r = \pi z (2L - z) k_h / q_w \quad (12)$$

$$d_w = (a+b)/2 \quad (13)$$

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:

$$d_s = 2 d_m \quad (14)$$

$$d_m = 2(w \cdot l / \pi)^{0.5} \quad (15)$$

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<sup>3</sup>/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.

*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 preloading load.

$$S_{ct} = U_t S_{cf} \quad (16)$$

The value of  $S_{cf}$  can be calculated using Eqs. (3) and (4), in which, the value of  $\sigma'_{vf}$  can be estimated as follows (Long et al., 2013):

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

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

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

$$p_{fill} = \sum \gamma_{fill} \cdot h_{fill} - \gamma_w \cdot h_w \quad (18)$$

where  $\gamma_{fill}$  is total unit weight of fill materials,  $h_{fill}$  is thickness of fill layer,  $\gamma_w$  is unit weight of water, and  $h_w$  is the thickness of embankment fill below water table.

- For PVD with vacuum pumping using sand blanket and airtight membrane :

$$\sigma'_{vf} = \sigma'_{v0} + \Delta\sigma_v + p_{vac} \quad (19)$$

where  $\Delta\sigma_v$  is induced vertical stress due to embankment fill that should be determined with embankment pressure of  $p_{fill} = \sum \gamma_{fill} \cdot h_{fill}$  considering that pore pressure in embankment fill under the airtight membrane is negative (Long et al., 2013).

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

$$S_s = H \cdot C'_\alpha \log(t/t_p) \quad (20)$$

where  $H$  is thickness of soft clay,  $C'_\alpha$  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  $C'_a$  in Eq. (20) should be taken in normally consolidated (NC) state,  $C'_{a(NC)}$ , or in over-consolidated (OC) state,  $C'_{a(OC)}$ , depending on stress state of the soil under service load in operation period.

### 5.1.3 Back analyses of settlement data

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

$$S_t = S_f [1 - \exp(-\lambda t)] \quad (21)$$

where  $S_t$  is consolidation settlement at time  $t$  under a constant load,  $S_f$  is final consolidation settlement, and  $\lambda$  is a constant. Asaoka (1978) pointed out that Equation 21 is a solution of following differential equation:

$$dS/dt - \lambda S = f \quad (22)$$

where  $f$  is an unknown constant.

From Eq. (22), the time is evenly discretized in  $\Delta t$  interval, and following expression can be obtained:

$$S_k = \beta_0 + \beta_1 S_{k-1} \quad (23)$$

$$\beta_1 = 1/(1 - \lambda \Delta t) \quad (24)$$

$$\beta_0 = f \beta_1 \quad (25)$$

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. (23), the values of  $\beta_0$  and  $\beta_1$  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:

$$S_f = \beta_0 / (1 - \beta_1) \quad (26)$$

From Eqs. (3), (21), and (24), the following expression can be derived for  $c_h$  value that is often used for back analysis of monitored settlement data.

$$c_h = \frac{(1 - \beta_1) d_e^2 F}{8 \beta_1 \Delta t} \quad (27)$$

## 5.2 Back calculation of compression index, $C_c$

From the measured settlements as given in Fig. 12 and Fig. 13, using Asaoka method to construct the  $S_k$  versus  $S_{k-1}$  plots, the final settlements for sub soil layers in SPA and VCA can be obtained as tabulated in Table 1. Having the final settlement,  $S_f$ , the value of compression index  $C_c$  can be back calculated using Eq. (3) as follows:

- Dividing the soft ground into many sub-layers.
- From Eq (1), Eq (2) to determine the average value of pre-consolidation stress,  $\sigma'_p$ , for each sub-layer.
- From Fig. 1 and using data from bore hole CB8 and BH15 to determine the average value of natural content,  $\omega$ , for each sub layer under SP-18 and SP-V15 in SPA and VCA, respectively.
- Calculate the induced vertical stress in each sub-layer due to embankment fill pressure. For vacuum pressure, uniform distribution of effective vacuum pressure along PVD length was assumed (Long et al., 2015).

Using the relation of  $C_c = 0.016\omega - C$  as generalized by Long et al. (2013) and  $C_r = 0.2 C_c$ , the value of constant  $C$  can be back-

calculated by trial-and-error until matching the final settlement calculated from Eq. (3) and the value of  $S_f$  from Asaoka plot. Back calculated results summarized in Table 1 indicated that the value of  $C$  varies from 0.235 to 0.303. With the average value of  $C = 0.282$ , following relationship can be made for soft clays in this region:

$$C_c = 0.016\omega - 0.282 \quad (28)$$

Table 1: Back calculated values of constant  $C$

Soil layer	Thick-ness (m)	$\omega$ (%)	$S_f$ (m)	$C$
SP-18 to EX03-2	13	100 to 66	2.31	0.293
EX03-2 to EX03-4	18	66 to 62	1.48	0.235
SP-V05 to EX1	13	92 to 66	2.04	0.298
EX1 to EX4	16.5	65 to 62	2.06	0.303

Notes:  $S_f$  = final settlement obtained from Asaoka method. Elevations of settlement plates (SP) and sub-surface extensometers (EX) can be seen in Fig. 9.

## 5.3 Back calculation of smear effect and flow parameters

From measured settlements of surcharge preloading area with PVD spacing of 1.2 m and vacuum consolidation area with PVD spacing of 1.0 m in Fig. 12 and Fig. 13, respectively, Asaoka plots for treated soft clay layer were constructed as seen in Fig. 19 and Fig. 20, respectively.

From Eq. 27, using  $\beta_1$  values from above Asaoka plots and assuming the field value of PVD discharge capacity  $q_w = 100 \text{ m}^3/\text{year}$  (Long et al., 2013), back-calculated results for  $c_h$  values are presented in Fig. 21 and Fig. 22. It can be seen from these figures that the back-calculated value of  $c_h$  is depending on the smear zone diameter ( $d_s/d_m$ ) and smear ratio  $R_s$ . For a constant value of  $d_s/d_m = 2$  and  $R_s = k_h/k_s = 2$ , the  $c_h$  values of  $3.5 \text{ m}^2/\text{year}$  that is about 4 times of  $c_{v(NC)}$  can be obtained for SPA. Taking the same value of  $c_h = 3.5 \text{ m}^2/\text{year}$  for VCA, the corresponding value of smear permeability ratio of VCA is  $R_s = 1.5$  that is 25% smaller than that of SPA. This is reasonable because under vacuum consolidation with very high hydraulic gradient at the soil/PVD interface in early stage of vacuum pumping, the clogging and blinding in the PVD filter can be improved and soil particles around the PVD might be re-arranged for better horizontal flow.

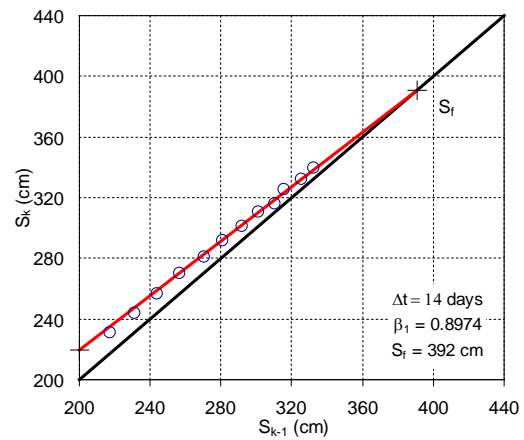


Fig. 19 Asaoka plot for soft clay layer between SP-18 and EX03-4 in surcharge preloading area (SPA)

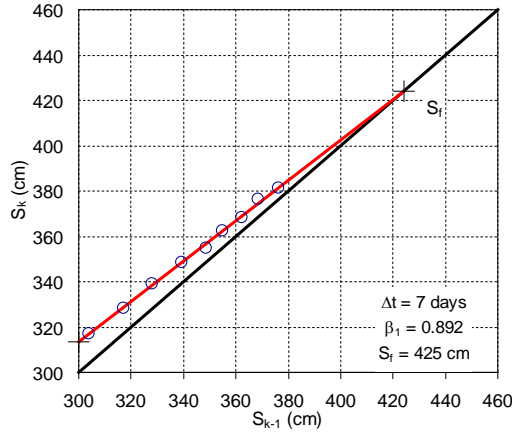


Fig. 20 Asaoka plot for soft clay layer between SP-V15 and EX3 in vacuum consolidation area (VCA)

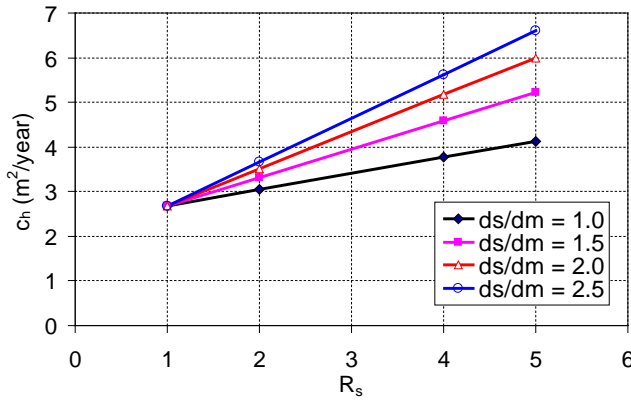


Fig. 21 Back-calculated  $c_h$  value for SPA

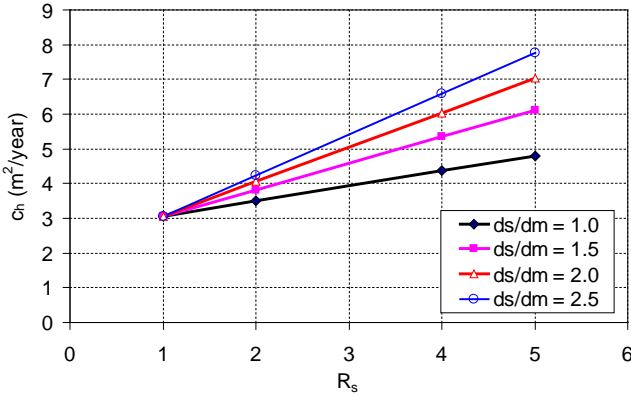


Fig. 22 Back-calculated  $c_h$  value for VCA

#### 5.4 Back calculation of time-settlement during preloading

1-D consolidation method was applied for back calculation of time-settlement during preloading for VCA and SPA at settlement plates SP-18 and SP-V15, respectively, as follows:

- Using boreholes CB8 and BH15 (Fig. 3 and Fig. 8) for determination of soil profile and soil properties under SP8 and SPV-15, respectively.
- Devising the soft ground into several sub-layers, using the back-calculated relation in Eq. (28) for calculating the value of  $C_c$  for each sub-layer.
- Stepwise the historical loading as multi-stages loading (the red lines in upper part of Figs. 23 and 24) and calculating the

induced vertical stress due to embankment fill for each sub-layer.

- For VCA, uniform distribution of effective vacuum pressure along the PVD length was assumed (Long et al., 2015).
- Using back-calculated results of  $d_s/d_m = 2$ ,  $c_h = 3.5 \text{ m}^2/\text{yr}$ , with  $R_s = 2$  for SPA and  $R_s = 1.5$  for VCA.

The back-calculated settlements versus time are plotted as the solid lines in Fig. 23 and Fig. 24 that are in excellent agreement with the measured data.

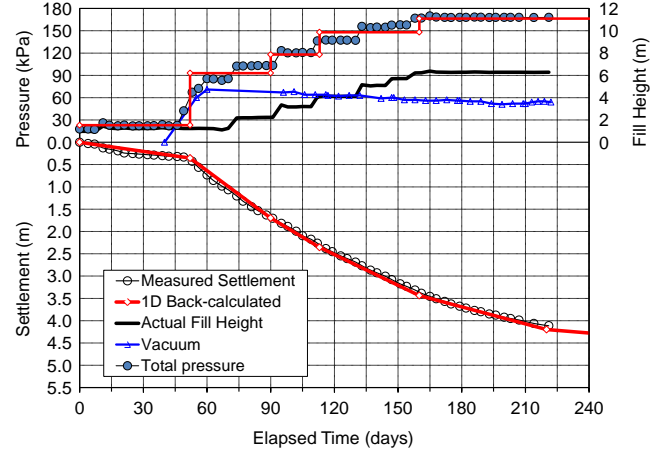


Fig. 23 Back calculation of settlement at SP-V15 of VCA

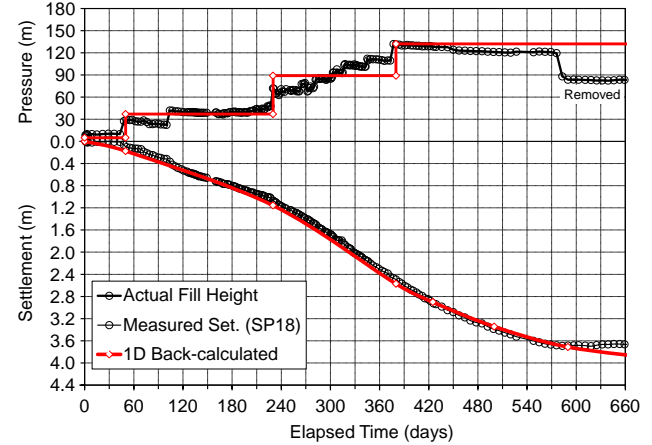


Fig. 24 Back calculation of settlement at SP-18 of SPA

#### 5.4 Back analyses of stability during construction

Back analyses of stability during construction were performed for the side slope of surcharge preloading area (Fig. 9a). From the measured pore pressures, the increase of effective stress,  $\Delta\sigma'_{vt}$ , can be calculated from the increase of total vertical stress,  $\Delta\sigma_{vt}$ , and excess pore pressure  $\Delta u_t$  at time  $t$  as below:

$$\Delta\sigma'_{vt} = \Delta\sigma_{vt} - \Delta u_t \quad (28)$$

Increase of undrained shear strength,  $\Delta s_u$ , corresponding to the increase of effective stress for this project can be estimated as  $\Delta s_u = 0.22 \Delta\sigma'_{vt}$ . From monitored excess pore pressures in Fig. 16, the increase of undrained shear strength can be estimated. Using Bishop simplified method and SLOPE/W software, the factor of safety against overall stability, FS, can be calculated as seen in Fig. 25. The calculated values of FS during banking and preloading are plotted in Fig. 26 together with the stability chart (Wakita & Matsuo, 1994). From this figure, it can be concluded that the stability chart suggested by Wakita & Matsuo (1994) is too conservative for PVD improved soft ground.



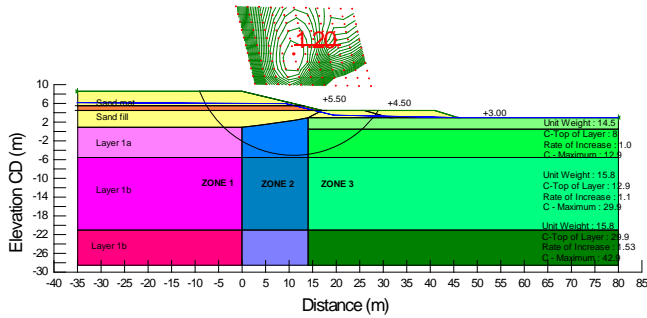


Fig. 25 Calculation of factor of safety using SLOPE/W soft ware

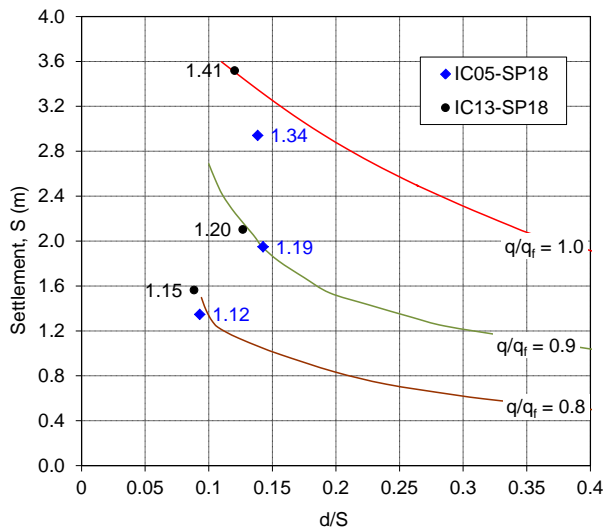


Fig. 26 Stability chart for side slope of SPA (numeric value is factor of safety, FS, calculated from measured pore pressure; solid lines is shear stress contour,  $q/q_t$ , from Wakita & Matsuo, 1994)

### 5.5 Analyses of residual settlement

After removing surcharge and completion of pavement construction, settlement monitoring points were installed for measurement of residual settlements during operation stage. The monitored results presented in Fig. 28 indicated that the residual settlement in the first 3 years of operation period was about 20 cm to 45 cm satisfying the design criteria of 50 cm. Residual settlement in vacuum area is significantly smaller than that in surcharge preloading area.

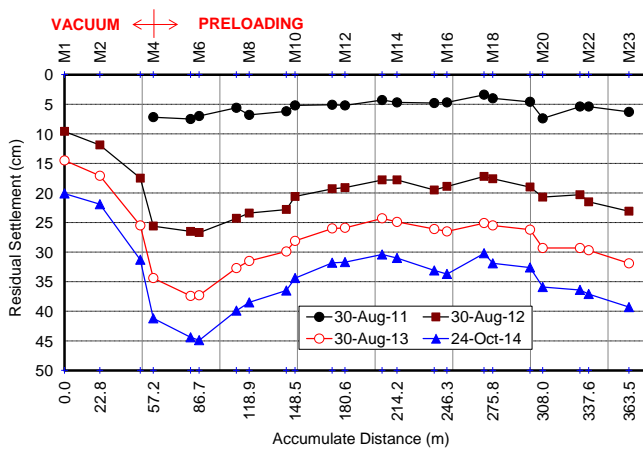


Fig. 27 Residual settlement measured from EOC (30 May 2011)

Plot of settlement versus time in log scale in Fig. 28 indicated that settlements in the first year may include the compression of embankment fill and additional primary consolidation settlement of

soft ground under container load, and then mainly due to secondary compression in following years (for  $t > 500$  days). From the relation of average compressive strain,  $S/H$ , versus  $\log t$  with  $t > 500$  days in Fig. 29, one can get the secondary consolidation ratio of this soft ground  $C'_{\alpha} = 1.2\%$  that is about 1.4 times higher than the average value of  $C'_{\alpha(NC)}$  from conventional oedometer test in Fig. 3. With  $C'_{\alpha} = 1.2\%$  and the improved soft ground thickness of  $H = 35$  m, the estimated residual settlement in 50 year is about 89 cm that is smaller than the design criteria of 120 cm.

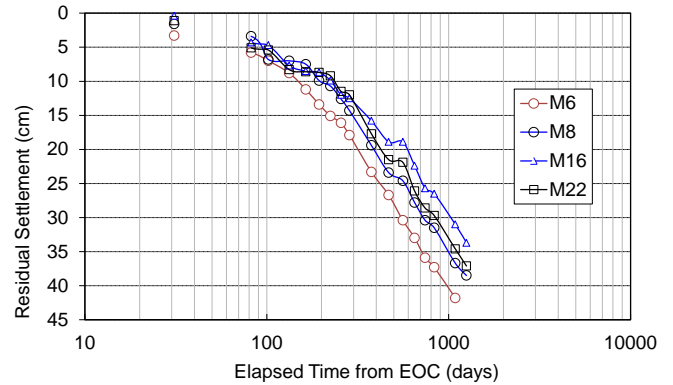


Fig. 28 Residual settlement versus time

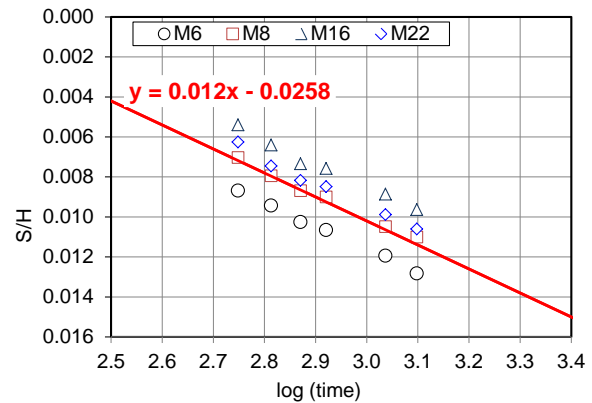


Fig. 29 Compressive strain versus time

## 6. CONCLUSIONS

One of the largest port projects in Vietnam constructed on 35 m thick soft clay deposit improved by PVD using conventional embankment preloading and reduced embankment with vacuum pumping has been presented including soil conditions, construction sequences, monitoring program, and analyses of monitored data. Based on the results of this study, following conclusions and recommendations can be made:

- PVD thickness of 3 mm with spacing of 0.9 m to 1.2 m can be used effectively for thick soft clay deposit in both methods of preloading with and without vacuum pumping.
- Comparison with conventional surcharge preloading, vacuum pumping combined with reduced embankment preloading can reduce significantly preloading time and post construction settlement.
- The back-calculated value of  $c_h$  is strongly dependent on the assumptions of smear effects. For an assumed value of  $d_s/d_m$ , the  $c_h$  value is directly proportional to smear permeability ratio  $R_s = k_s/k_h$ . Assuming  $d_s/d_m = 2$  as popular used, and  $c_h = 4c_{v(NC)} = 3.5 \text{ m}^2/\text{year}$ , the back-calculated values of  $R_s = 2$  for SPA and  $R_s = 1.5$  for VCA have been obtained that indicated that the

- smear resistance under vacuum pumping is significantly smaller than that under embankment surcharge preloading.
- Back analyses of settlement data illustrated that the secondary compression ratio,  $C'_\alpha$ , of soft marine clay in this area is about of 1.2 % that is about 1.4 times higher than the average value from conventional oedometer tests.
  - Using the back-calculated values of compressibility and flow parameters, the time-settlements re-calculated by 1-D consolidation method are in very good comparison with measured data for both conventional preloading and vacuum consolidation considering the vacuum pressure as induced vertical stress distributed uniformly in the PVD zone.
  - Analyses of factor of safety from measured pore pressures illustrated that the commonly used stability chart as given by Wakita & Matsuo (1994) is too conservative for the case of PVD improved soft ground.
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