Performance of PVD improved soft ground using vacuum consolidation methods with and without airtight membrane

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ABSTRACT

In order to investigate the performance behavior of soft ground improvement using different vacuum consolidation methods (VCM) and different PVD thicknesses, four trial sections namely C1, C2, D1, and D2 were constructed. VCM without airtight membrane using cap drains and direct tubing system (VCM-DT) were used for the first two sections with PVD thickness of 3 mm and 7 mm for C1 and C2, respectively. VCM with airtight membrane and band drains (VCM-MB) were applied for the last two sections for D1 and D2 with PVD thickness of 3 mm and 7 mm, respectively. The soil conditions, construction procedures, instrumentation program, and monitored results of the above trial sections are presented in this paper. The results confirmed that the effective vacuum pressure in PVD mainly depends on vacuum consolidation methods and the assumption of uniform distribution of vacuum pressures along the PVD depth which can be suggested for practical design. For VCM-MB using PVD thickness of 3 mm arranged in triangular pattern of 0.9 m spacing, the degree of consolidation of more than 90% can be achieved in less than 8 months of vacuum pumping. However, for VCM-DT, further investigation is needed for preventing air leakage in vacuum system particularly for the case of thick soft clay deposits with large deformations during the preloading.

1. Introduction

Soft clay deposits are wide spread in Mekong River Delta (MRD) and Saigon-Dong Nai River lower plain (SDR) including Ho Chi Minh city(HCMC) in Southern Vietnam. The low shear strength and the high compressibility of these soft clays have challenged the wit of the geotechnical design engineers in solving problems related to the stability condition during embankment construction as well as to the post construction settlements. Residual settlements including differential settlements during operation period constitute major problem of most highway projects constructed on soft ground in MRD and SDR. Preloading soft clay deposits for increasing stability and controlling the post construction settlements using PVD with embankment surcharge and vacuum pumping has been extensively applied (Bergado et al., 1998; Chu et al., 2000; Yan and Chu, 2005; Kelly and Wong, 2009; Rujikiatkamjornand and Indraratna, 2007, 2009, 2013; Indraratna et al., 2005, 2011, 2012; Artidteang et al., 2011; Geng et al., 2012; Long et al., 2013; Chai et al., 2013a, 2013b; Voottipruex et al., 2014). In Viet Nam, the first application of surcharge combined vacuum consolidation for soft ground improvement under expressway embankment was proposed for the North-South Expressway (NSEW) connecting HCMC and Dong Nai province. In order to evaluate the performance behavior including vacuum pumping techniques and the influence of PVD thickness on vacuum pressure distribution as well as other assumptions to be used in design calculations for this project, four trial sections, namely: Sections C1, C2, D1, and D2 were conducted using different vacuum consolidation methods (VCM) and different PVD thicknesses. VCM without airtight membrane using cap drains and direct tubing system (VCM-DT) were used for Sections C1 and C2 with PVD thickness of 3 mm and 7 mm, respectively. VCM with airtight membrane and band drains (VCM-MB) were applied for Sections D1 and D2 with PVD thickness of 3 mm and 7 mm, respectively. Site conditions, construction procedures, instrumentation program and monitored results as well as analyses and discussions on the

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List of notations

		p_{wt}	pore water pressure inside the vertical drain at time t
		t	time
a	thickness of prefabricated vertical drain	u_0	initial excess pore water pressure at a depth z
L_d	length of prefabricated vertical drain	u_z	excess pore water pressure at a depth z at time t
C_c	compression index	U_z	degree of consolidation at a depth z
c_v	coefficient of consolidation in vertical direction	z	depth from ground surface to a considered point
e_0	void ratio	γ	total unit weight of soil
H	embankment height	$\Delta\sigma_v$	increase of total vertical stress at a depth z due to the self weight of embankment fill
p_c	pre-consolidation pressure of improved soil		
p_{fill}	pressure at the base of embankment due to the self weight of embankment fill	$\Delta\sigma'_{vt}$	increase of effective vertical stress at a depth z at time t
		$\Delta\sigma'_{vf}$	increase of effective vertical stress at a depth z when excess pore water pressure is fully dissipated
p_{vac}	effective vacuum pressure inside the vertical drain		
p_{w0}	pore water pressure inside the vertical drain before vacuum pumping	σ'_{v0}	effective overburden stress of natural soil at a depth z

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performance behavior of the above trial sections are presented in following sections.

2. Soil conditions

The project site was located at Nhon Trach District in Saigon-Dong Nai River delta (SDR), about 25 km to the east of HCMC. The site area is quite flat with natural ground surface at elevation (EL) of +0.5 m to +0.70 m. The ground water table in rainy season is near the ground surface at EL of about +0.5 m and it is about 0.5 m−1.0 m lower in dry season. The soil profile along the center line of trial sections is presented in Fig. 1, consisting of following sub-soil layers:

- Current fill: Fine sand fill with thickness of about 0.5 m−1.0 m.
- Layer 1a: Very soft clay of around 6−10 m thick with some organic matters, average values of water content are of 90.3%, plastic limit of 45%, liquid limit of 95%, unit weight of 14.5 kN/m³, and SPT-N values from 0 to 1.

- Layer 1b: Underlying layer 1a to the depth of about 11 m to 23 m, soft clay with some organic matters, average values of water content are of 72.2%, plastic limit of 44%, liquid limit of 81%, unit weight of 1.51 kN/m³, and SPT-N values from 1 to 3.
- Layer 2: Underlying layer 1b to the depth of about 15 m to 28 m, firm to stiff clay with some organic matters, average values of water content are of 34.2%, plastic limit of 30%, liquid limit of 72%, unit weight of 1.89 kN/m³, and SPT-N values from 10 to 22. Sand lenses (L2-1) were found in this soil layer.
- Layer 3: Underlying layer 2 to the depth beyond the bottom of borehole is medium dense, fine to medium sand with SPT-N values from 10 to 34.

The basic properties of foundation soils consisted of wet unit weight, γ_w, natural water content, ω, and initial void ratio, e_0, from 4 boreholes along the center line of the trial embankments are plotted together in Fig. 2. The soil parameters from laboratory oedometer tests for the geotechnical parameters along the expressway including the trial sections are represented in Fig. 3, in
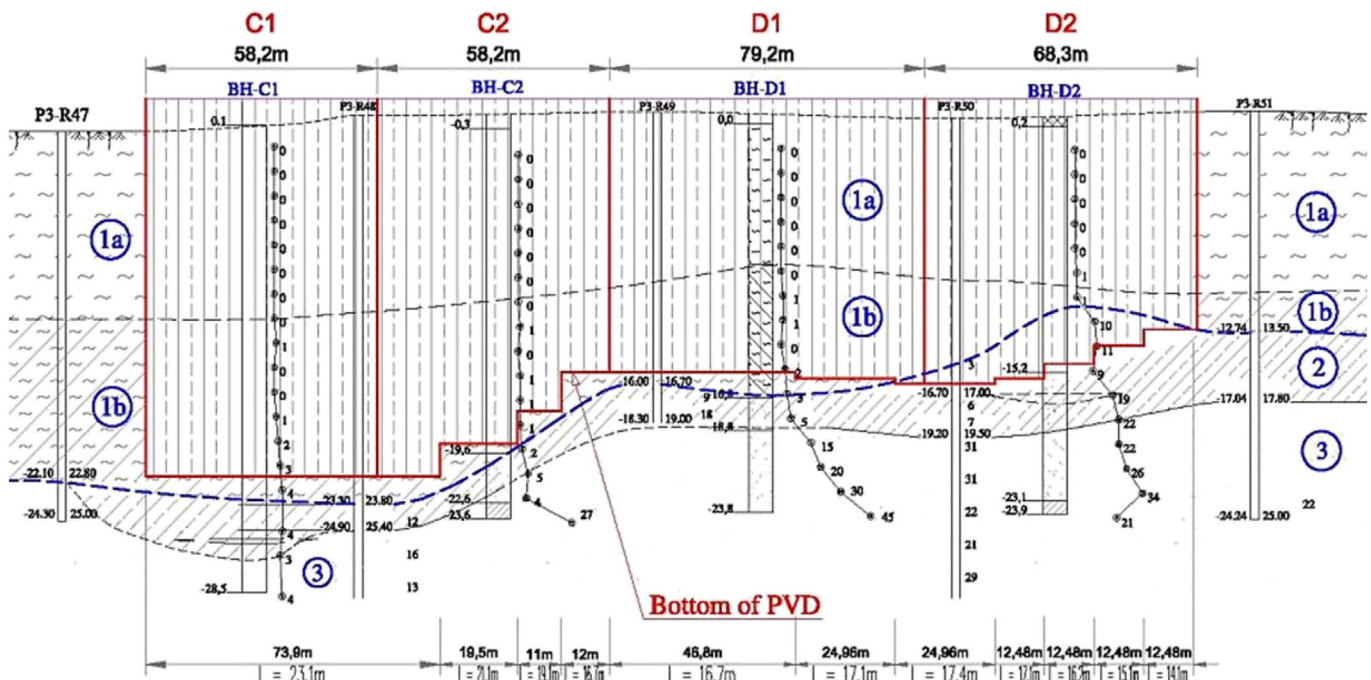

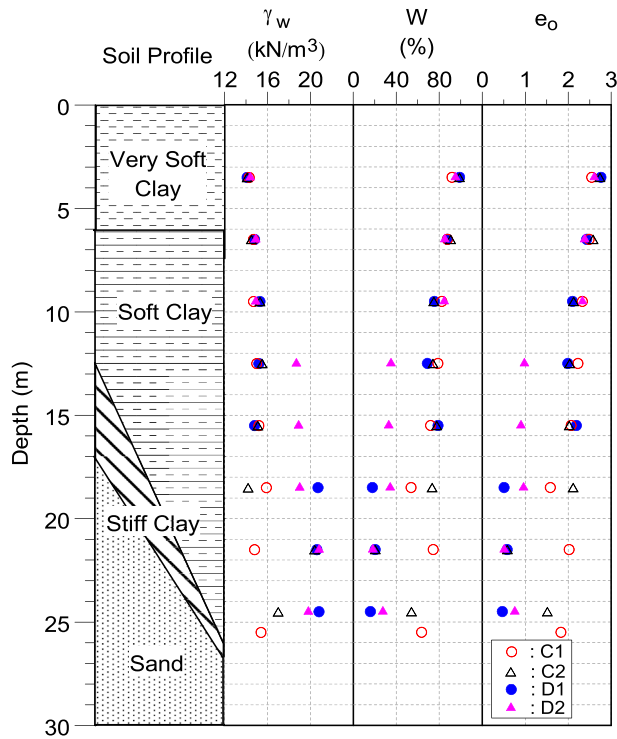**Fig. 1.** Soil profile along the Trial Sections C1, C2, D1 and D2.

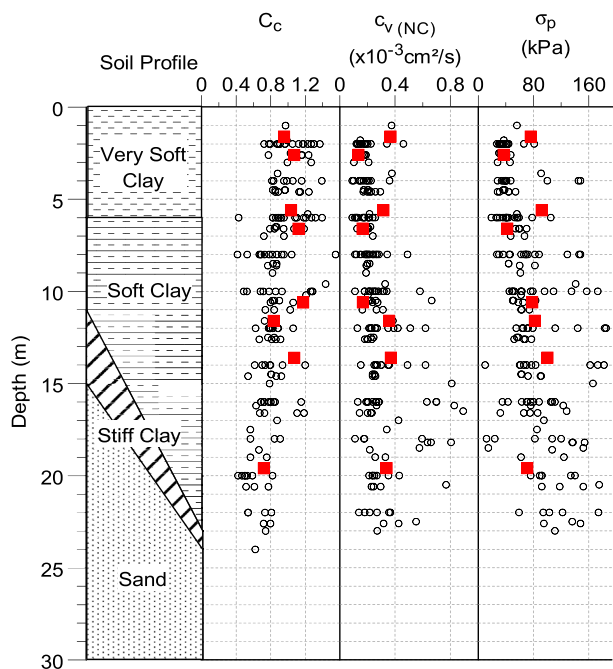**Fig. 2.** Index properties versus depth.

Fig. 3. Compressibility and flow parameters versus depth.

which, the solid square markers are the values from boreholes in these trial sections. The scatter of compressibility and flow parameters including compressibility index, C_c, coefficient of consolidation, c_v, and pre-consolidation stress, σ_p, as seen in Fig. 3 may be mainly due to inevitable disturbance of soft clay samples (Ladd and DeGroot, 2003).

3. Construction and instrumentation

3.1. Construction procedures

The layout of trial embankments is presented in Fig. 4. The embankment height, H, length of PVD (to the top of working platform), L_d, thickness of PVD, a, and vacuum consolidation method (VCM) for each embankment are summarized in Table 1. The PVD spacing of 0.9 m arranged in triangle pattern was used for all embankments. PVD lengths along the center line of embankments can be seen in Fig. 1. Generally, the PVDs were penetrated to the depth of about 0.5–1.0 m above the bottom of the soft clay layer. However, in some areas such as Section D2, PVDs were installed beyond the soft clay layer into the underlying layer with sand lenses.

Vacuum consolidation method (VCM) with direct tubing system (VCM-DT) using cap drains without airtight membrane was applied for C1 and C2 sections. Schematic configuration of VCM-DT can be seen in Fig. 5. In this method, PVDs are connected to the vacuum pump through a flexible tubing system using a prefabricated fitting cap for each individual PVD. Construction procedures of C1 and C2 sections are summarized as follows:

- Removing top soil and backfilling with silty sand for working platform.
- Cutting PVD and assembling with fitting cap and HDPE pipe of 16 mm diameter (Fig. 6), PVD thickness of 3 mm and 7 mm were used for C1 and C2, respectively.
- Installation of PVDs ensuring that the fitting caps shall be embedded in soft clay of not less than 0.5 m, the opening above the caps must be filled with in-situ soft clay slurry.
- Installation of monitoring devices.
- Testing and checking of vacuum system at least in one week before placing embankment surcharge.
- Placing embankment surcharge using fine to medium silty sand, compacted with lift thickness of 300 mm.

For D1 and D2 section, the VCM using airtight membrane and horizontal band drains (VCM-MB) were applied. Schematic configuration of VCM-MB is shown in Fig. 7. The construction procedures are as follows:

- Removing top soil and backfilling with silty sand for working platform.
- Installation of PVDs using PVD thickness of 3 mm and 7 mm for D1 and D2, respectively.
- Laying down horizontal band drains (HBD) of 7 mm thickness and connecting with PVDs by plastic tag pins.
- Installation of 55 mm diameter perforated pipe in excavated trench and backfilling with fine crushed stones. The perforated pipes were wrapped around with non-woven geotextile and placing directly on HBDs.
- Laying down non-woven geotextile for protection of the high density polyethylene (HDPE) membrane against punching from sharp materials (Fig. 8).
- Installation of 1.5 mm thickness airtight HDPE membrane on top of the protection geotextile. The edges of the airtight membrane were embedded below the soft clay surface of not less than 0.5 m in the excavated trench and backfilling with in-situ clay and bentonite slurry.
- Start vacuum pumping for testing and checking of airtightness for at least of one week before placing embankment surcharge.
- Placing embankment surcharge using fine to medium silty sand compacted with lift thickness of 300 mm.
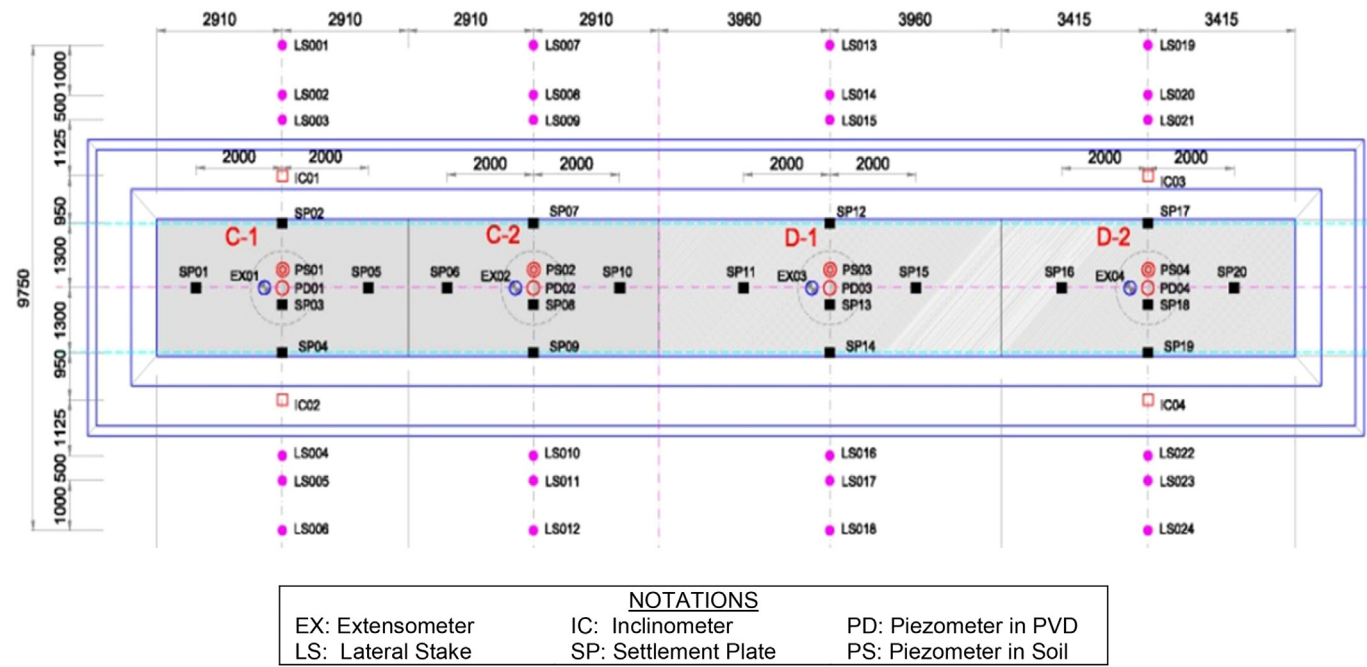
Fig. 4. Layout of trial embankments and monitoring devices.

Table 1
Embankment height (H), PVD length (L_d), PVD thickness (a), and VCM type.

Section	H (m)	L_d (m)	a (mm)	VCM
C1	4.78	23.1	3	VCM-DT
C2	4.65	17—22	7	VCM-DT
D1	4.85	16.7—17.4	3	VCM-MB
D2	4.73	17.4—14.1	7	VCM-MB

Connecting PVD to fitting cap

Flexible pipe system connecting to vacuum pump

Fig. 6. Connecting PVD to fitting cap and tubing system.

Fig. 5. Schematic configuration of VCM-DT.

3.2. Instrumentation program

The layout of monitoring instruments can be seen in Fig. 4 and the typical cross section is presented in Fig. 9. The following devices were installed:

- Settlement plates (SP) for measurement of total settlement at the level of the ground surface.
- Magnetic extensometers (EX) for measurement of settlements at various depths.
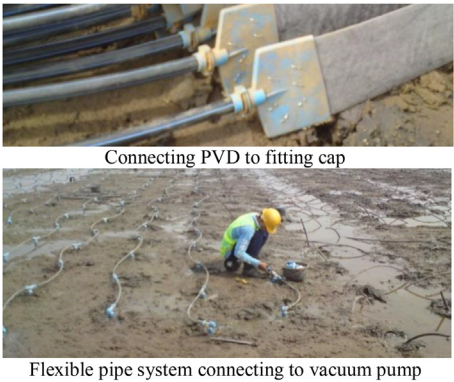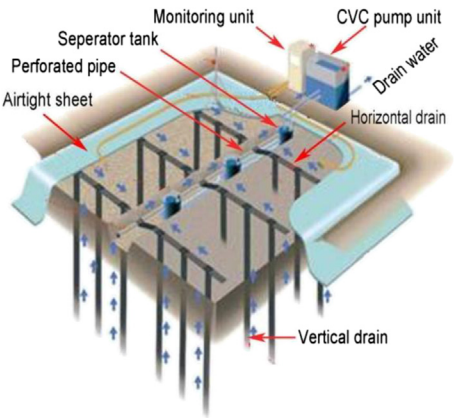

Fig. 7. Schematic configuration of VCM-MB.

Fig. 8. Installation of horizontal band drains and perforated pipes.

- Vibrating wire type piezometers denoted as PS were installed in between PVDs for measurement of total pore pressures in sub-soils at the depths of middle and bottom of PVD.
- Vibrating wire type piezometers denoted as PD were installed inside the PVD (attached in-between PVD's core and jacket) at the middle depth of PVD for measurement of total water pressure inside the PVD.
- Inclinometers (IC) for monitoring lateral displacements, two inclinometers were installed in both sides of each section C1 and D2, but no ICs for the other sections.
- Lateral movement stakes (LS) were installed in both sides of all sections for observation of ground surface movements.

The installed depths of PD, PS, and EX for each trial section can be read in Fig. 9. The monitoring works were carried out daily during surcharge filling and weekly during waiting period.

4. Monitored results

4.1. Vacuum pressure and pore pressure

Pressure at the base of embankment due to the self weight of embankment fill, p_{fill}, vacuum pressure at vacuum pump, p_{pump}, effective vacuum pressure at the middle depth of PVD, p_{vac}, pore pressure in soft soil, PS, total water pressure inside vertical drain, PD, and corresponding excess pore pressures are plotted with time in Figs. 10a,b–13a,b for C1, C2, D1, D2, respectively. In these figures, the value of p_{vac} is defined as $p_{vac} = p_{w0} - p_{wt}$ where p_{wt} is pore water pressure measured at time t, and p_{w0} is the pore water pressure before vacuum pumping. The values of total pore pressures presented herein were corrected with settlements of the piezometer tips.

For Section C1 using VCM-DT with 3 mm PVD thickness, a 4.78 m thick embankment surcharge was placed in 110 days with the average rate of surcharge filling of about 330 mm per week. Vacuum pumping was applied in a continuous period of 295 days (from day 35 to day 330) with vacuum pressure, p_{pump}, of 90 kPa to 95 kPa. In the first stage of about 75 days (from day 35 to day 110), the effective vacuum pressure from PD01 (inside the PVD) was quite stable with the average value of p_{vac} of about 67 kPa. Since the day 110, the effective vacuum pressure at PD01 was totally lost and the values of pore pressures measured in soft soil (between PVDs) at PS01 became identical with that at PD01 as seen in Fig. 10a. The excess pore pressures in sub-soils interpreted from PS01 and PS05 corresponding to the depths of −10.44 m and −22.00 m, respectively, together with that inside PVD at the depth of −10.44 m from PD01 are presented in Fig. 10b. The maximum excess pore pressure built up in soft ground at location of PS01 is 25 kPa corresponding to about 28% of the embankment pressure. Negative excess pore pressures in sub-soil were recorded from all piezometers installed at bottom of PVD improved zone including PS05 in Section C1 and PS06, PS07, PS08 in Sections C2, D1, D2, respectively. This can be
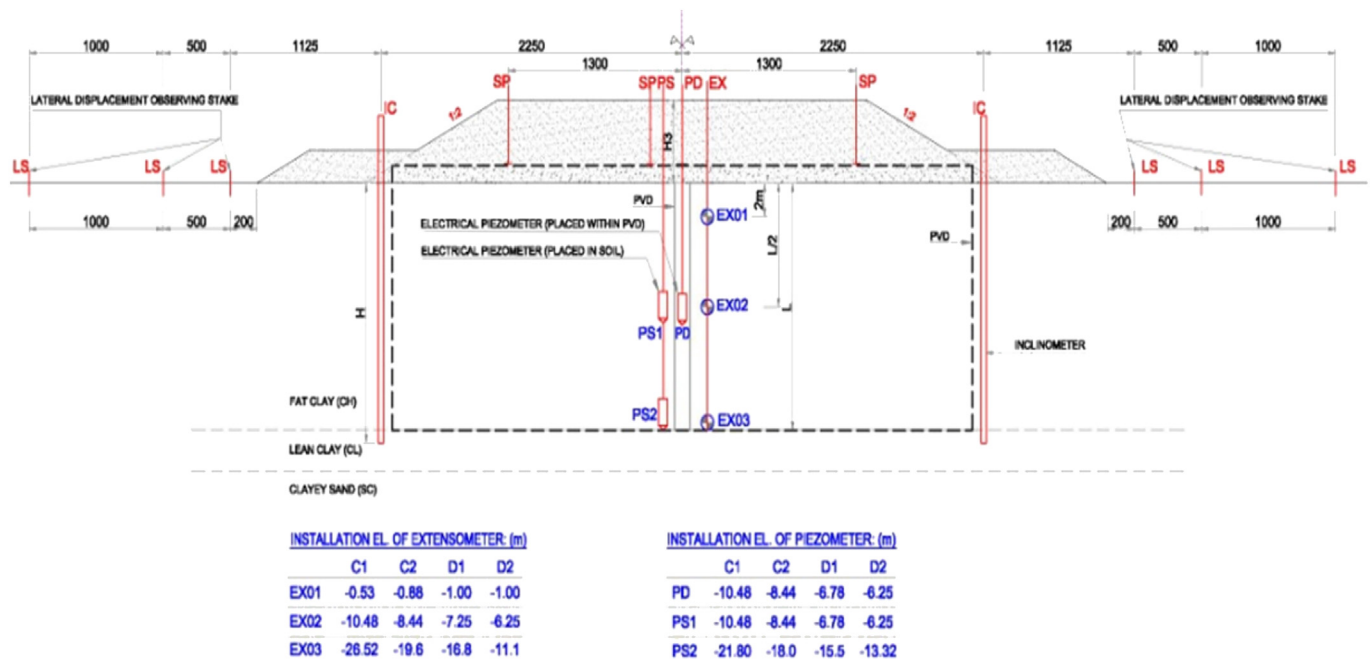

INSTALLATION EL. OF EXTENSOMETER (m)				
	C1	C2	D1	D2
EX01	-0.53	-0.88	-1.00	-1.00
EX02	-10.48	-8.44	-7.25	-6.25
EX03	-26.52	-19.6	-16.8	-11.1

INSTALLATION EL. OF PIEZOMETER (m)				
	C1	C2	D1	D2
PD	-10.48	-8.44	-6.78	-6.25
PS1	-10.48	-8.44	-6.78	-6.25
PS2	-21.80	-18.0	-15.5	-13.32

Fig. 9. Typical cross section of embankment with monitoring devices.
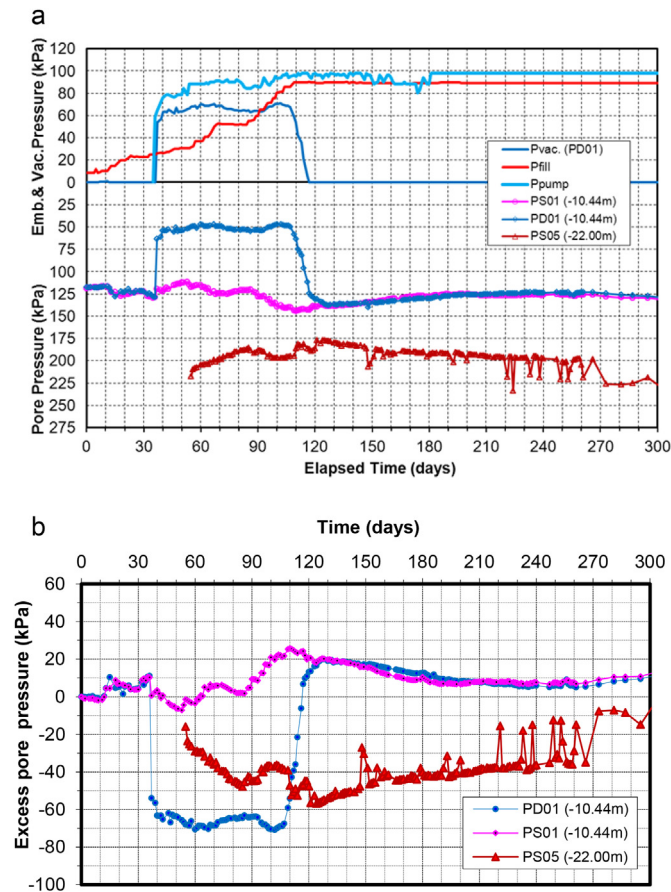
Fig. 10. a. Vacuum pressures, embankment pressure, and pore pressures for Section C1.
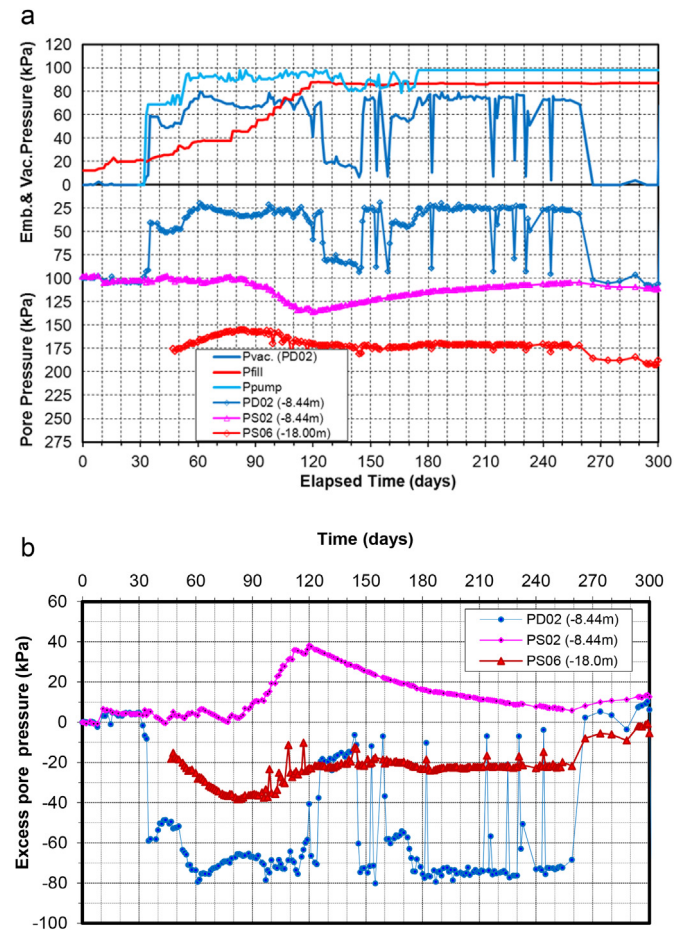b. Excess pore pressures in soil (PS01, PS05) and inside PVD (PD01) for Section C1.

Fig. 11. a. Vacuum pressures, embankment pressure, and pore pressures in Section C2.
b. Excess pore pressures in soil (PS02, PS06) and inside PVD (PD02) for Section C2.

explained as these piezometers were over-penetrated into sand lenses connected to PVDs and/or the factual location of piezometer was too close to the PVD due to PVD spacing as small as of 0.9 m and the unavoidable inclination of bore hole for installation of piezometer to the deep level of about 20 m.

For SectionC2 using 7 mm PVD thickness, a 4.65 m thick embankment surcharge was placed in 120 days with the average rate of surcharge filling of about 270 mm per week as seen in Fig. 11a. Vacuum pumping was applied in a continuous period of 298 days (from day 32 to day 330) with vacuum pressure, p_{pump}, of about 95 kPa. In the first stage of about 93 days (from day 32 to day 125), the effective vacuum pressure from PD02 (inside the PVD) was quite stable with the average value of p_{vac} of about 68 kPa. However, from the time of 125 days, the effective vacuum pressure was strongly fluctuated and then totally lost at day 265. The excess pore pressures in soft soil interpreted from PS02 and PS06 corresponding to the depths of −8.44 m and −18.00 m, respectively, together with that inside the PVD at the depth of −8.44 m from PD02 are presented in Fig. 11b. The maximum excess pore pressure built up in sub-soil at PS02 is 40 kPa corresponding to about 46% the embankment pressure which is about 1.7 times higher than that at PS01 in C1 section. The construction procedures including vacuum pumping and surcharge filling of C1 and C2 were almost the same, thus, the higher excess pore pressure in C2 may be explained as the factual distance from PS01 to the PVD was smaller than that of PS02.

For Section D1 using VCM-MB with 3 mm PVD thickness, a 4.85 m thick embankment surcharge was conducted in 170 days

including the first 100 days for construction of working platform and other preparation works. The average rate of surcharge filling was about 435 mm per week. Vacuum pumping was applied in a period of 240 days (from day 50 to day 290) with vacuum pressure, p_{pump}, of 90 kPa to 95 kPa. The effective vacuum pressure inside PVD at PD03 was quite stable at about 75 kPa throughout the time of vacuum pumping as seen in Fig. 12a. The excess pore pressures in sub-soils interpreted from PS03 and PS07 corresponding to the depths of −7.25 m and −15.5 m, respectively, together with that inside the PVD at the depth of −7.25 m from PD03 are plotted in Fig. 10b. The maximum excess pore pressure built up in soft ground at PS03 is 50 kPa corresponding to about 55% the embankment pressure.

For Section D2 using 3 mm PVD thickness, 4.73 m embankment surcharge was placed in 185 days. Excluding the first 90 days for construction of working platform and other works, the average rate of surcharge filling was about 320 mm per week. Vacuum pumping was applied in a period of 238 days (from day 42 to day 280) with average vacuum pressure, p_{pump}, of about 95 kPa. The effective vacuum pressure inside PVD at PD04 was quite stable with p_{vac} of about 75 kPa in the first stage but since the elapsed time of t = 160 days, it was gradually dropped to about 20 kPa at the end of pumping as shown in Fig. 13a. The responses at piezometer PS08 (at EL of −13.5 m) were quite similar to that at vacuum pump indicated that this piezometer was installed in the sand lens fully connected with PVDs. The excess pore pressures in sub-soils at PS04 and PS08 corresponding to the depths of −6.25 m
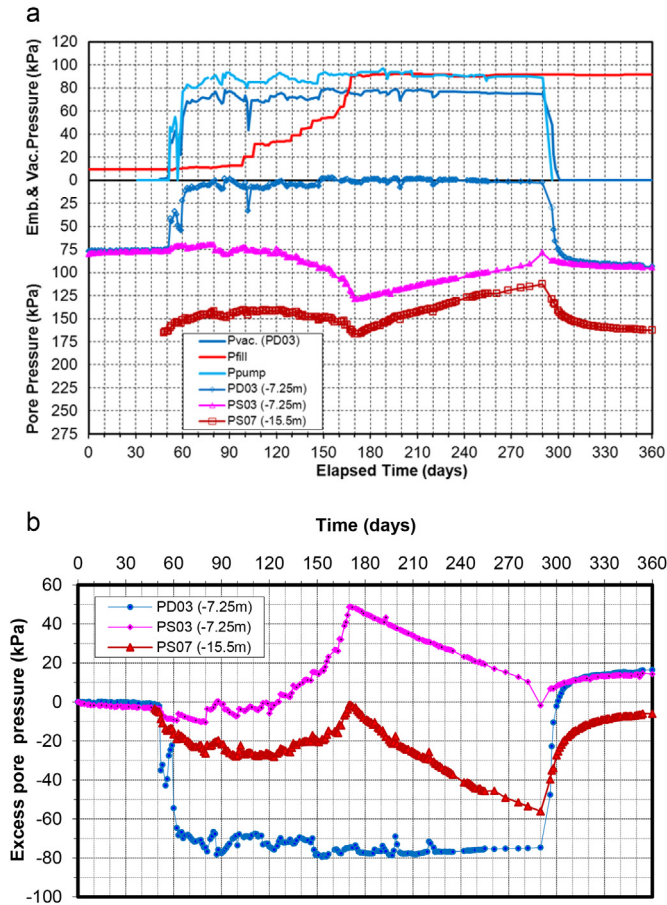
Fig. 12. a. Vacuum pressures, embankment pressure, and pore pressures in Section D1.
b. Excess pore pressures in soil (PS03, PS07) and inside PVD (PD03) for Section D1.
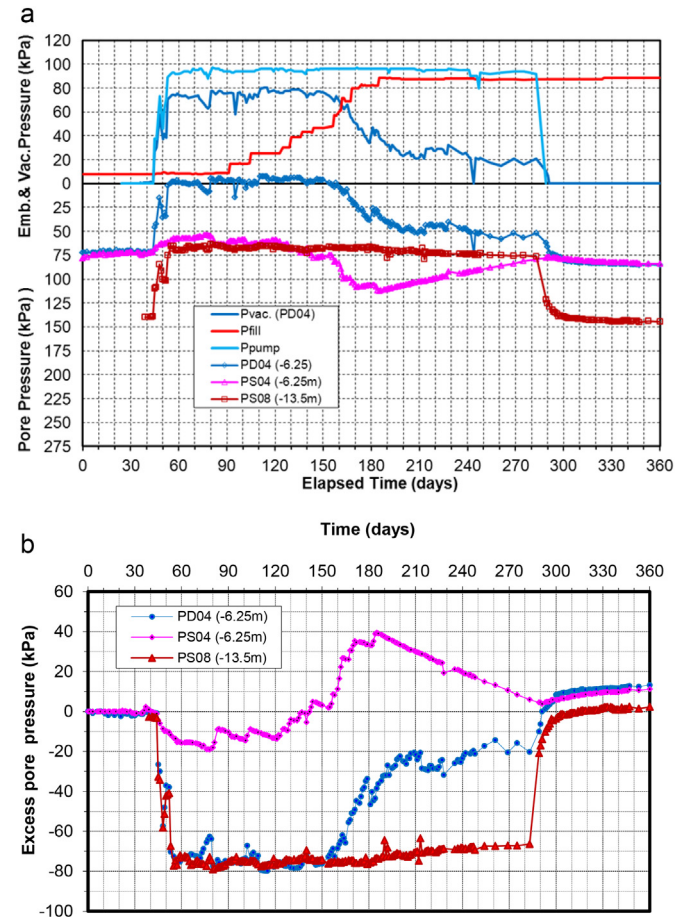

Fig. 13. a. Vacuum pressures, embankment pressure, and pore pressures in Section D2.
b. Excess pore pressures in soil (PS04, PS08) and inside PVD (PD04) for Section D2.

and −13.5 m, respectively, together with that inside PVD at the depth of −6.25 m from PD04 are plotted in Fig. 13b. The maximum excess pore pressure built up in soft ground at PS04 is 40 kPa corresponding to about 45% the embankment pressure. The negative excess pore pressure at PS08 was stable at about −75 kPa to −70 kPa during vacuum pumping period also confirmed that the vacuum loss at PD04 may be due to a local problem at this particular location while the whole vacuum system could be in good performance.

Further insight to the magnitude and distribution of vacuum pressure along PVD length, the effective vacuum pressures inside PVDs from PD03 and PD04, and effective vacuum pressure from PS08 (over-penetrated into sand lens fully connected to PVDs as explained above) are plotted together in Fig. 14. As seen in this figure, the average value of p_{vac} for VCM-MB is about 75 kPa while that value for VCM-DT (at PD01 and PD02 in Figs. 10a and 11a) is about 68 kPa, regardless of PVD depth or PVD thickness of 3 mm or 7 mm. Similar results were found previously for VCM-DT at Yamaguchi Prefecture project (Chai et al., 2008), in which, the effective vacuum pressures measured at different depths of 1.5 m and 27.0 m are in the same magnitude of about 65 kPa. Thus, the assumption of uniform distribution for effective vacuum pressure along the PVD depth as suggested by Long et al. (2013) for practical design has been confirmed. This also implies that the head loss, i.e. the well resistance, along the PVD length can be neglected in practice as discussed previously (Balasubramaniam et al., 1995; Bergado et al., 1996, 2002; Long et al., 2006).

4.2. Settlements

Surface settlements, SP, sub-surface settlement, EX, and rate of surface settlement, ROS, at the center of embankments are plotted with elapsed time. For Sections C1 and C2 using VCM-DT, the rate of settlement decreased significantly right after the time of vacuum pressure loss as seen in Figs. 15 and 16. The settlement observed at the time of vacuum loss is about 1.8 m for both sections. The
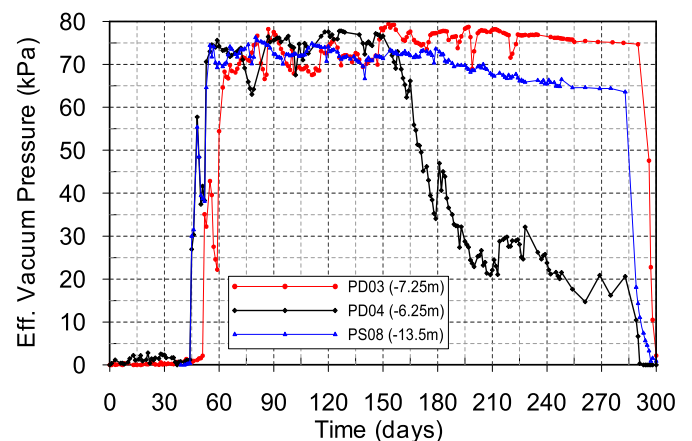

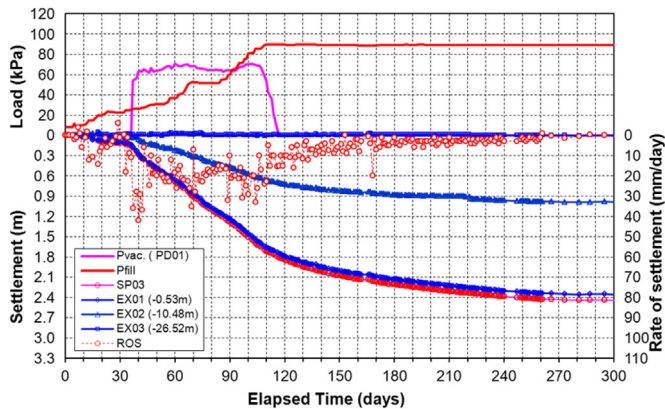**Fig. 14.** Effective vacuum pressures at various depths of Sections D1 and D2.

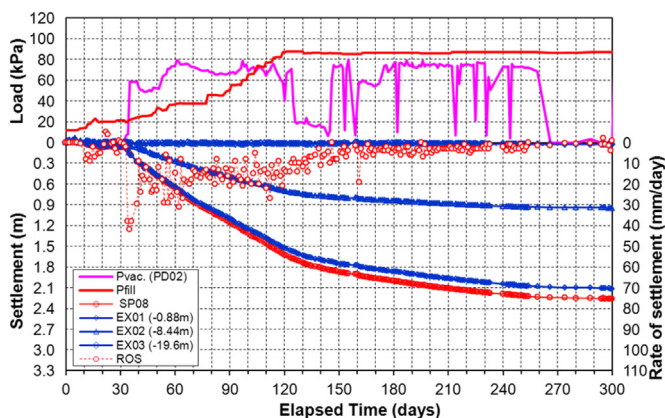**Fig. 15.** Loading pressures and settlements versus time of Section C1.

Fig. 16. Loading pressures and settlements versus time of Section C2.

settlements at the end of preloading, EOP (at t = 300 days), corresponding to Sections C1 and C2 are 2.44 m and 2.29 m, respectively. With the thickness of the improved soft ground of 22.5 m and 20.2 m, the average compressive strains at EOP can be obtained as 10.8% and 11.3% for Sections C1 and C2, respectively.

For Section D1 using VCM-MB, the measured settlement at EOP is 2.83 m with maximum rate of settlement as high as of 30 mm per day (Fig. 17). According to the soft ground thickness of 17.3 m, the average compressive strain of soft ground due to ground improvement is about 16.4%. Using Asaoka (1978) graphical
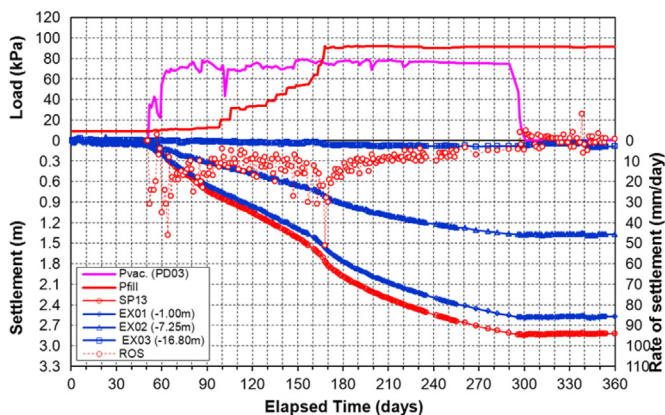
method to construct the relationship of $S_k \sim S_{k-1}$ for surface settlements at SP13, the value of final settlement, S_f, of 302 cm can be obtained for D1 as seen in Fig. 18. Thus, the average degree of consolidation at EOP estimated from measured settlement is 93.7%. It is noted the final primary 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. Otherwise, secondary settlement would be included in the value of S_f (Hausmann, 1990).

For Section D2, the monitored settlement at EOP is 2.06 m with maximum rate of settlement of about 25 mm per day (Fig. 19). According to the soft ground thickness of 11.6 m, the average compressive strain of soft ground due to ground improvement is about 17.7% which is much higher than that in Sections C1 and C2. Using Asaoka method for plotting the surface settlements at SP18 as seen in Fig. 20, the value of final settlement of 223 cm and the average degree of consolidation at EOP of 92.4% are obtained.

For comparison, the settlements at the end of preloading, S_{EOP}, depth of soft ground, D, average compressive strain of improved soft ground, S_{EOP}/D, and the average value of effective vacuum pressure, p_{vac}, of all Sections are tabulated in Table 2.

From aforementioned figures and Table 2, following comments can be made:

- Compressive strains of the improved soft ground of Sections D1 and D2 using VCM-MB are about 1.5 times greater than that of Sections C1 and C2 using VCM-DT. This is mainly due to the loss of effective vacuum pressure in later stage of VCM-DT. Disconnections at fitting caps (between PVD and fitting cap or between
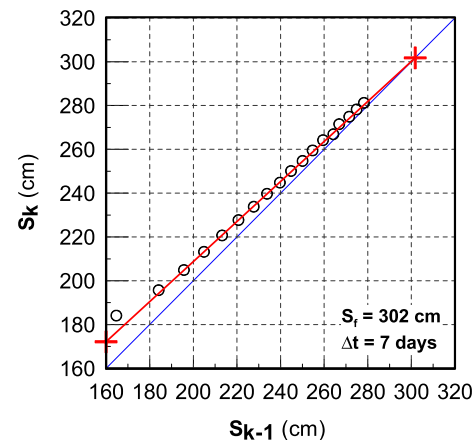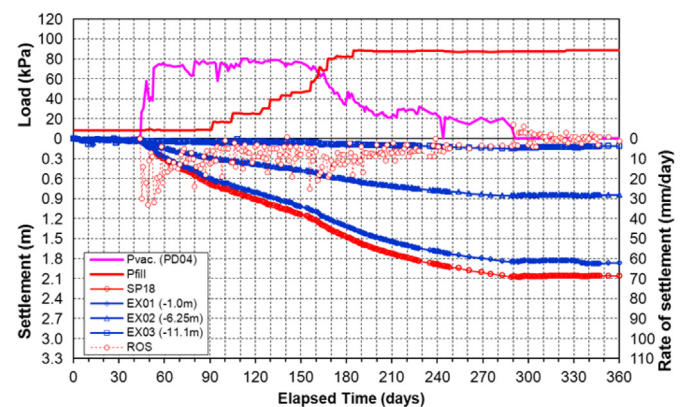

Fig. 18. S_k versus S_{k-1} plot for Section D1.

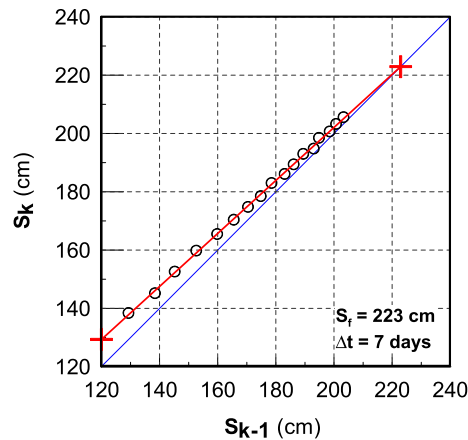**Fig. 17.** Loading pressures and settlements versus time of Section D1.

Fig. 19. Loading pressures and settlements versus time of Section D2.

Fig. 20. S_k versus S_{k-1} plot for Section D2.

Table 2
Comparison performances of Sections C1, C2, D1 and D2.

Section	p_{vac} (kPa)	S_{EOP} (m)	D (m)	S_{EOP}/D (%)	PVD thickness (mm)
C1	67	2.44	22.5	10.8	3
C2	68	2.29	20.2	11.3	7
D1	75	2.83	17.3	16.4	3
D2	75	2.06	11.6	17.7	7

fitting cap and vertical HDPE pipe) or at T-connectors (between vertical and horizontal HDPE pipes) could be the main problem of VCM-DT for thick soft clay deposits with large deformations during preloading.

- Compressive strain of the improved soft ground of Section D2 is well comparable with that of Section D1 (without vacuum loss) illustrated that the whole vacuum system in D2 was in good performance and the vacuum loss in piezometer PD04 may be a local problem at the vicinity of this piezometer.
- Effective vacuum pressures in PVDs mainly depend on the vacuum consolidation method irrespective to the depth of improved soft ground or PVD thickness of 3 mm or 7 mm. Thus, PVD thickness of 3 mm and uniform distribution of p_{vac} along PVD length can be used for practical design of VCM.

4.3. Lateral displacements

Lateral displacements of PVD improved soft ground using vacuum pumping and embankment preloading mainly depends on the surcharge loading rate, SRL, and load ratio, LR, defined as LR = p_{fill}/p_{vac} where p_{vac} is vacuum pressure and p_{fill} is embankment pressure (Ong and Chai, 2011). From Fig. 10a, the values of LR during surcharge filling and vacuum pumping for Section C1 can be obtained as follows: LR = 2.8 at t = 36 days (start vacuum pumping), LR = 0.5 at t = 50 days, LR = 1 at t = 90 days, and LR = 1.47 at t = 110 days (end of filling, EOF). The lateral displacements versus depth of soft ground for t = 50, 90, 110, and 300 days are plotted in Fig. 21.

For Section D2, from Fig. 13a, the values of load ratio during surcharge filling and vacuum pumping can be obtained as follows: LR = 0.13 from t = 45 days (start pumping) to t = 90 days, LR = 0.5 at t = 130 days, LR = 1 at t = 170 days, and LR = 1.3 at t = 190 days (end of filling). The lateral displacements versus depth of soft
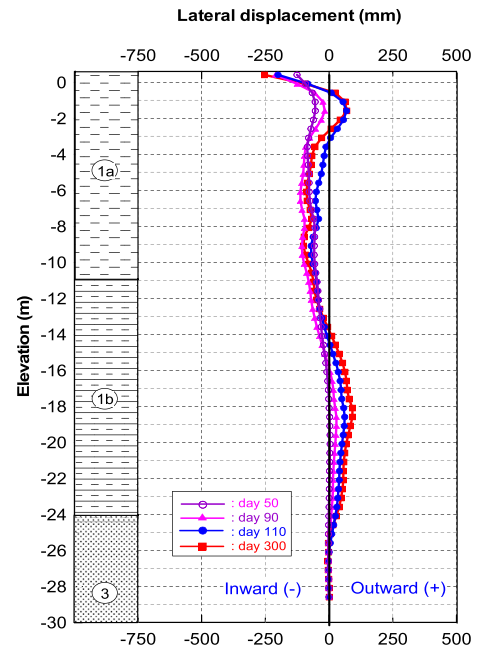

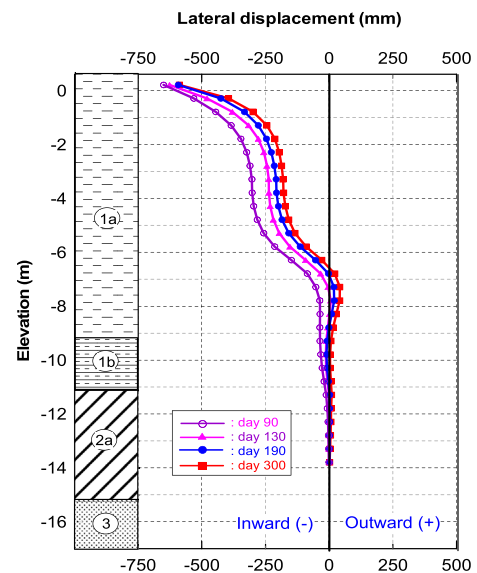**Fig. 21.** Lateral displacements of sub-soils in Section C1.

Fig. 22. Lateral displacements of sub-soils in Section D2.

ground for t = 90 days, 130 days, 190 days, and 300 days are plotted in Fig. 22.

From Figs. 21 and 22, it can be seen that at LR < 1 (corresponding to t < 90 days for Section C1 and t < 170 days for Section D2), inward lateral displacement occurred in both sections and it decreased with increase of loading ratio (Ong and Chai, 2011). The inward lateral displacement in the soft ground of Section D2 is about 2.5 times greater than that of Section C1. This is mainly due to the load ratio (LR) of Section D2 in early stage was about 21 times smaller than that of C1, particularly the vacuum pumping in D2 was maintained at maximum capacity during the first 45 days without adding embankment surcharge. Moreover, for both sections, the maximum inward lateral displacement occurred at shallow depth near the ground surface and decreasing with the increase in depth.
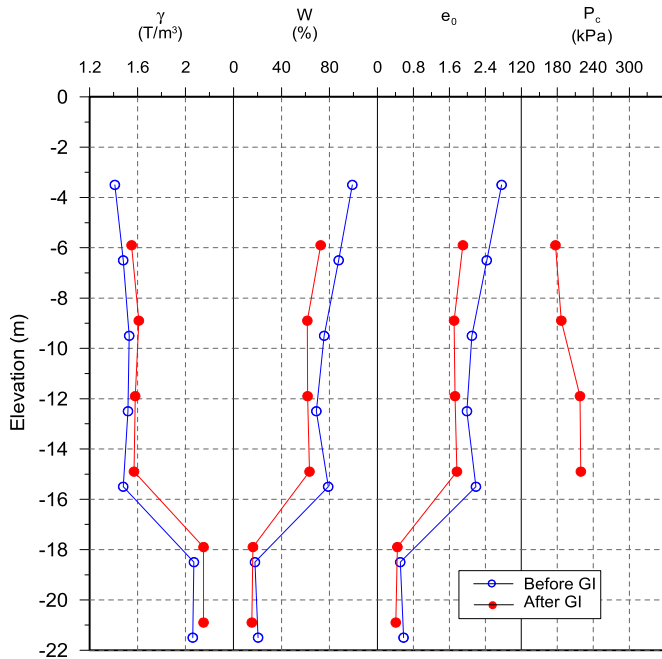
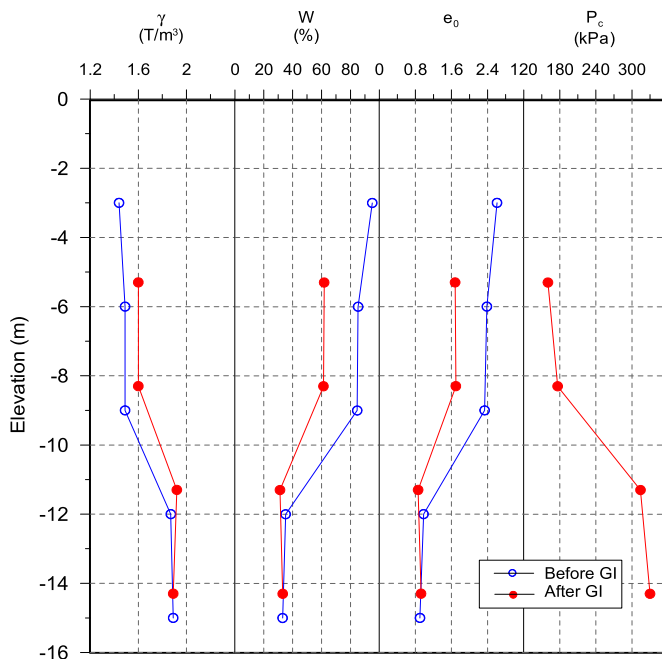Fig. 23. Index properties of sub-soils before and after GI at Section D1.

Fig. 24. Index properties of sub-soils before and after GI at Section D1.

5. Soil properties before and after improvement

After the end of vacuum pumping, two boreholes were done at locations nearby the previous ones conducted before ground improvement (GI). The index properties including total unit weight, γ, water content, ω, void ratio e_0, and pre-consolidation pressure, p_c, before and after ground improvement are plotted in Figs. 23 and 24 for Sections D1 and D2, respectively. After improvement, the water contents and void ratios of the soft ground were reduced to about 70% and the pre-consolidation pressures were increased significantly for both sections.

The degree of consolidation at depth z, U_z, at any time t can be calculated from excess pore water pressures as follows:

$$U_z = (u_0 - u_z)/u_0 \tag{1}$$

where u_0 is initial excess pore water pressure and u_z is excess pore water pressure at a depth z at time t. Because the consolidation progresses by the dissipation of excess pore water pressure, and the amount of excess pore water pressure dissipation is equal to that of effective stress increase, Eq. (1) can be re-written in terms of effective stress as follows:

$$U_z = \Delta\sigma'_{vt}\Big/\Delta\sigma'_{vf} \tag{2}$$

where: $\Delta\sigma'_{vt}$ = increase of effective stress at depth z, time t

$\Delta\sigma'_{vf}$ = increase of effective stress at depth z when excess pore pressure fully dissipated.

From Eq. (2), with the values of unit weight, γ, of sub-soils before GI, the degree of consolidation at EOP for the sub-layer at depth z can be estimated from the pre-consolidation pressure after GI at that depth, p_c, as follows:

$$U_z = (p_c - \sigma'_{v0})/(\Delta\sigma_v + p_{vac}) \tag{3}$$

where σ'_{v0} is initial overburden and $\Delta\sigma_v$ is the increase of total vertical stress due to embankment fill at depth z. For Sections D1 and D2, p_c = 180 kPa at EL of −8.0 m (corresponding to the depth of z = 8.6 m from the original ground surface), γ = 14.5 kN/m^3, p_{vac} = 75 kPa and $\Delta\sigma_v$ = 79 kPa (at z = 8.6 m under center of embankment), one can get the value of U_z = 91.8% for the sub-layer at middle depth of the improved ground.

6. Conclusions

Four trial sections of PVD improved soft ground using embankment surcharge combined with vacuum pumping have been presented with various vacuum consolidation methods, construction sequences, monitoring program, and discussions on monitored results. Based on the results of this study, following conclusions and recommendations can be made:

- The effective vacuum pressures in PVD, p_{vac}, mainly depend on the vacuum consolidation methods. The measured values of p_{vac} of about 68 kPa for VCM-DT and 75 kPa for VCM-MB, irrespective to PVD length, indicated that the assumption of uniform distribution of vacuum pressure along PVD depth can be used for practical design.
- The influence of PVD thickness (3 mm or 7 mm) is negligible and even the performance of Section D1 using 3 mm PVD thickness is better than that of Section D2 using 7 mm PVD thickness. Thus, the PVD thickness of 3 mm can be suitable for vacuum consolidation method.
- For VCM-MB using PVD thickness of 3 mm arranged in triangular pattern of 0.9 m spacing, the degree of consolidation of more than 90% can be achieved in less than 8 months of vacuum pumping. However, for VCM-DT, further investigation is needed for preventing air leakage in the vacuum system, particularly for the case of thick soft clay deposits with large deformations during preloading.
- Lateral displacements of soft ground improved by surcharge combined with vacuum consolidation depend on the load ratio (LR) between surcharge load and vacuum pressure. For Section D2 with load ratio applied in early stage as low as LR = 0.13, the

maximum inward lateral displacement as high as of 500 mm was observed. From the measured lateral displacements in Sections C1 and D2, it can be expected that the lateral displacement during preloading can be controlled by using the appropriate values of load ratio and the rate of surcharge loading.

Acknowledgments

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