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Evaluation of ground improvement and recharge using prefabricated vertical drain (PVD) for the second bangkok international airport (SBIA) project

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ABSTRACT: The improvement of the inherent properties of soft Bangkok clay by preloading with prefabricated vertical drain (PVD) is extensively and presently applied. However, the issues related to analysis and design have not been widely available to geotechnical engineers. This paper attempts to illustrate the consolidation parameters for design and to introduce a method to capture the settlement characteristics of the soft ground improved by PVD. The method is the one-dimensional (1-D) finite element analysis, which can handle the case of the multi-layered soil deposits with different soil properties. The analysis of the settlement for the soil improved by PVD at the Second Bangkok International Airport (SBIA), Thailand with the different consolidation parameters at each soil layer is done by the PVD-SD software Version 2.3. It is found that the predicted curves were in agreement with the measured ones. Furthermore, based on the measurements at the SIBA site and the full-scale recharge test at the campus of Asian Institute of Technology, it was demonstrated that the PVD was also effective in recharging the ground and erased the piezometric level drawndown down to the depth of PVD installation. The piezometric drawdown is due to the excessive withdrawal of groundwater leading to the ground subsidence.

1 INTRODUCTION

The accelerated consolidation of thick soft clay can be done with the use of prefabricated vertical drain (PVD). PVDs are artificial drainage paths consisting of a geosynthetic central core which serves are flow channel wrapped around by a geotextile filter jacket.

For the PVD design, geotechnical engineers must clearly understand the factors affecting the behavior of the PVD so as to properly select the design parameters. Chai and Miura (1999) revealed that the parameters affecting the behavior of the PVD include (1) coefficient of consolidation in horizontal direction (2) discharge capacity of PVD (3) smear zone parameters (diameter, d_s, and horizontal hydraulic conductivity ratio, k_h/k_s).

Hanbo (1981) was the first person who modified the equations developed by Barron (1984) for PVD application, which has been widely used. The solutions assumed a uniform subsoil condition. However, most natural clay deposits are not uniform and normally has a crust layer at the surface such as the Bangkok soil deposit. A multi-layer one-dimensional (1-D) finite element program for PVD improved subsoil (PVD-SD) was proposed by Chai and Bergado (1998) for handling this problem. The features of the program and the theoretical background are being described in the next section. The application of the program is demonstrated by analyzing the settlement of the full-scale test embankment on PVD improved subsoil at the Second Bangkok International Airport (SBIA), Thailand. Finally, it was observed that the PVD installation recharged the subsiding ground and erased the negative piezometric drawdown. In most PVD projects, the selection of PVDs is based mainly on: (a) safe installatin of PVD and (b) optimum performance during the project duration.

2 1-D FEM PROGRAM (PVD-SD)

2.1 *Features of the program*

The PVD-SD has been developed for calculating the consolidation of soft clay deposit improved by PVDs. The program can consider (Chai and Bergado, 1998):

(1) Muti-layer improved subsoil.
(2) The effects of both vertical hydraulic conductivity of natural subsoil and horizontal drainage of PVDs.
(3) Both full and partial penetration condition of PVDs.
(4) Effect of surface vacuum pressure.
(5) Either one-way or two-way drainage.

2.2 *Theoretical background*

In finite element formulation, the effect of PVDs is considered by modifying 1-D continuity equation of consolidation as follows (Chai and Bergado, 1998).

$$\frac{k_v}{\gamma_w}\frac{\partial^2 u}{\partial z^2} - \frac{8k_h u}{\gamma_w D^2 F} + \frac{\partial \varepsilon_v}{\partial t} = 0 \tag{1}$$

$$F = \ln\frac{n}{s} + \frac{k_h}{k_s}\ln s - \frac{3}{4} + \pi\frac{2l^2 k_h}{3q_w} \tag{2}$$

where γ_w is the unit weight of water; z is the depth; t is the time; ε_v is the volumetric strain, u is the excess pore pressure; k_v and k_h are hydraulic conductivities in vertical and horizontal directions, respectively; k_s is the hydraulic conductivity in smear zone; l is the drainage length; D is the diameter of a unit cell (equal to 1.13 times spacing of PVD for square pattern and 1.05 times spacing of PVD for triangular pattern); q_w is the discharge capacity of PVD; $n = D/d_w$ (d_w is the equivalent diameter of PVD); $s = d_s/d_w$; and d_s is the diameter of smear zone = 2 to 3 times the equivalent diameter of the cross-sectional area of mandrel.

Undrained shear strength increased by the reduction of water content is calculated by two ways. One way is to use an empirical equation (Ladd, 1991), which relates the S_u with effective vertical stress σ'_v and overconsolidation ratio (OCR).

$$S_u = S\sigma'_v(OCR)^m \tag{4}$$

where S and m are constants, In the program, m is fixed as 1.0.

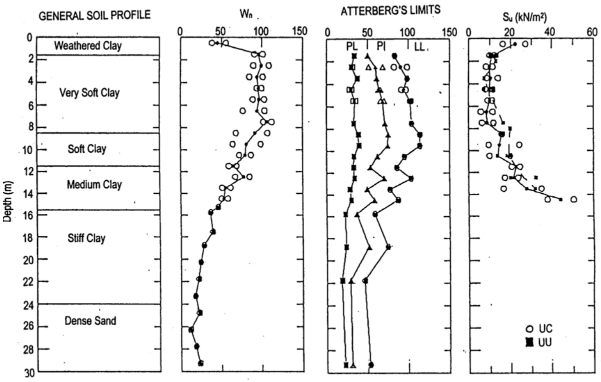

Figure 1. Soil profile and soil properties at the SBIA site.

The other way is using Modified Cam clay theory (Roscoe and Burland, 1968) as follows:

$$S_u = \frac{p'}{2^{1+\left(1-\frac{\kappa}{\lambda}\right)}} M \left(\frac{M^2 + \eta^2}{M^2} \right)^{\left(1-\frac{\kappa}{\lambda}\right)} \tag{3}$$

where p' is the mean effective stress, M is the slope of the critical state line in (q, p') plot (q is the deviator stress), $\eta = q/p'$, and κ and λ are the slopes of unloading-reloading and virgin loading curves in void ratio versus ln p', respectively. In the program, κ is fixed as $\lambda/10$.

3 SELECTION OF PVD

For safe PVD installation, the following tests are designated to confirm the strength of the geotextile filter jacket: (a) grab tensile test, (b) trapezoidal tear test, (c) puncture test, and (d) burst test, based on the standard procedures of the American Society of Testing Materials (ASTM). The U.S. Federal Highway Authority (FHWA) has specified the values of grab tensile strength, trapezoidal tear strength, puncture resistance and burst strength as 355 N, 220 N, 900 kPa, and 110 N, respectively. Voskamp et al (1998) obtained the tensile forces of the whole PVD (core + filter) ranging from 230 to 390 N from field measurements.

The requirements for optimum performance of PVD include the apparent opening size (AOS) of the geotextile filter and the discharge capacity of the PVD in both straight and deformed conditions. The pore size distribution of the filter is selected to restrain soil migration while allowing pore water to pass through. If the maximum value of D_{85} is taken as 0.03 mm for soft Bangkok clay and using the criterion that: $O_{95}/D_{85} \leq 3$, then the AOS or O_{95} can be taken as 0.09 mm. The relationship: $O_{95}/D_{15} \geq 3$, is recommended for preventing clogging of the geotextile filter.

Using the lateral pressure of 200 kPa, the empirical values of the factors affecting the discharge capacity from 10 different PVD types at straight condition and at varying severity of deformed conditions. Thus, for soft Bangkok clay, considering the effects of time, deformation, and filtration/clogging, the discharge capacity measured in the laboratory at straight condition with a lateral pressure of 200 kPa and hydraulic gradient of unity, was found to be 500 m^3/ year. The results demonstrated the decrease of discharge capacity with the increase in lateral pressures and increasing severity of deformed conditions Six PVD types were considered suitable for the SBIA Project. Eventually, Mebra Drain (MD7007) were mostly used with minor portions allocated to CN Drain, a local licensee of Geodrain.

4 ANAYSIS OF SBIA TEST EMBANKMENT

4.1 *Test embankment at SBIA*

The project site is located at Nong Ngu Hao, Samutprakan Province, Thailand. The project site is situated on the swampy land in flat marine deltaic deposit. The soil profile and soil properties are shown in Fig. 1. The total area of full-scale embankment test is about 118m x 560m. The depth of installed PVD was 10 m deep below the original ground surface with an improved area about 77m x 518m. The PVD was installed in 1 m square pattern. Table 1 illustrates the loading stage. A 1.0 m thick sand blanket was constructed before installation of PVD and an overlying 0.5 m thick sand drainage was then built. Sand blanket was used as a working platform for PVD to rig on the top of the soft clay. Drainage pipes were installed after the completion of sand blanket. Thereafter, crushed rock was used as the surcharge fill to provide overburden load for initiating consolidation process.

Table 1. Features of surcharge fill.

Conditions	Stage 1	Stage 2	Stage 3	Stage 4
Fill material	Sand	Sand	Crushed Rock	Crushed Rock
Fill height	1.0	0.5	1.3	1.0
Unit weight (kN/m^3)	18	18	21	21
Total surface load (kPa)	18	9	27.3	21
Total time (days)	160	20	129	201

Table 2. Summary of soil properties at SBIA for settlement analysis.

Depth (m)	γ_t (kN/m^3)	w_n (%)	σ_p (kPa)	C_c	M	OCR	C_v (m^2/yr)	C_h (m^2/yr)
0-2	16.50	70	42	0.262	1.2	3.62	1.00	2.00
2-8	14.55	110	38	0.463	0.9	1.17	2.01	4.02
8-10	14.70	100	67	0.375	1.0	1.30	2.22	4.50
10-15	16.35	59	90	0.300	1.2	1.24	2.61	3.00

Eight types of instrument including surface settlement plates, deep settlement gauges, observation wells, electric piezometer, AIT type piezometers, inclinometers, surface settlement movements and permanent benchmark were installed to monitor the embankment performance.

4.2 *Parameters for analysis*

For the analysis, the subsoil was divided into four layers as given in Table 2 together with the relevant soil properties. The values of $C_{v(field)} = 4C_{v(lab)}$ were taken in this analysis based on the suggestion of Bergado et al. (1992) for Bangkok subsoil. The assumption that $C_{h(field)} = 2.0C_{v(field)}$ was further made based on past experiences.

The parameters for PVD are listed in Table 3. The equivalent diameter, d_w was calculated from the equation proposed by Rixner et al. (1986).

$$d_w = \frac{a+b}{2} \tag{5}$$

where a and b are the width and thickness of a band-shaped drain, respectively. The diameter of the smear zone was estimated as 2.0 times the equivalent mandrel diameter for soft Bangkok clay (Bergado et al., 1991). Chai and Miura (1999) suggested relationship between the laboratory and field parameters for smear effect as follows.

$$\left(\frac{k_h}{k_s}\right)_{field} = \left(\frac{k_h}{k_s}\right)_{lab} C_f \tag{6}$$

where C_f is the hydraulic conductivity ratio between field and laboratory values. C_f is 4.0 for Bangkok clay, which was obtained by back analysis (Chai et al., 1996). Assuming that a single value of representative hydraulic conductivity in smear zone is 0.4 times that of undisturbed zone, the k_h/k_s is 10. This value is the same as that suggested by Bergado et al. (1992). They showed that $k_h/k_s = 10$ for $d_s/d_w = 2.5$. The value of the discharge capacity is 50 m^3/yr obtained from back analysis of three full-scale embankments (Bergado et al.1996).

Table 3. Parameters related to PVD for settlement analysis

Item	Symbol	Unit	Values
Equivalent drain diameter	d_w	m	0.052
Unit cell diameter	D	m	1.13
Diameter of smear zone	d_s	m	0.186
Hydraulic conductivity ratio	k_h/k_s	-	10
Discharge capacity	q_w	m^3/yr	50
Length of PVD	L	m	10
Spacing (Square pattern)	S	m	1.0

4.3 *Prediction of settlement*

The calculated surface settlements on the embankment at centerline is compared with the measured values in Fig.2 Also the predicted value by Asaoka method (1978) is shown in the figure. It is found that the settlement predicted by PVD-SD and Asaoka's method agreed well with the observed data. The Asaoka's method is an analysis based on the field results, so it can successfully predict the settlement. However, in practice, the settlement characteristics are required before the commencement of the construction. The determination of settlement of multi-layer subsoil with the laboratory soil parameters is necessary. It is found that the analysis using PVD-SD can serve this need.

5 RECHARGE OF PIEZOMETRIC DRAWDOWN

The other advantage of the PVD besides to accelerate the settlement is to recharge the ground and to remove the piezometric drawdown, which leads to the ground subsidence. It is well known that in Bangkok, the ground subsidence is likely caused by groundwater extraction due to deep well pumping, surcharge loading, and self-weight consolidation. Bergado et al. (1990) revealed that an average ground subsidence rate was about 0.02 to 0.04 m/year.

The measured pore pressures at SBIA site was recorded on a previous full-scale test embankment after the installation of PVD (Bergado et al. 1997). It was an interesting observation that the effect of the piezometric drawdown was markedly reduced as found in Fig. 3. Line ABC is the dummy pore pressures and line DEF shows the theoretical pore pressure immediately after loading with 75 kPa surface. The dummy pore pressures were obtained at the unloaded part of the site and were located away from the influences of the test embankment. The dummy readings represent the initial pore pressure conditions before preloading which indicates the pore pressure drawdown due to excess withdrawal of groundwater from the underlying aquifers. Curve MNPQ indicates the average total pore pressure reading in February 1996 (about 21 months after construction), showing that excess pore pressure fully dissipated. It is observed that there is recharge from 8 to 12 m depth. Thus, the recharge extends to the entire PVD zone down to 12 m depth.

In order to understand clearly this phenomenon, another full-scale recharge test was conducted at the campus of Asian Institute of Technology. The test embankment was constructed in November 1999. The pore pressure versus depth in the dummy area (without PVD installation) is shown in Fig. 4. The drawdown of the piezometric level was observed starting at 5 m below the ground surface. In this study, the sand embankment, which served as drainage blanket and construction platform was constructed to a height of 1.2m with a dimension of 20 m x 25 m. The PVD brand Mebra (MD88-80) was installed to 10 m depth from the original ground in 1 m square pattern.

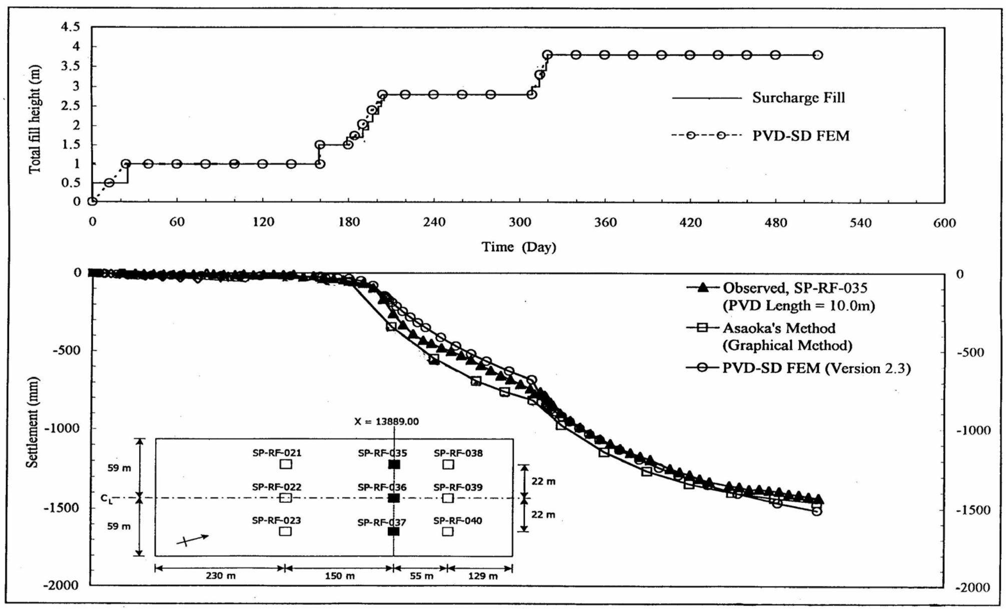

Figure 2. Comparison of settlement between observed and predicted settlement (Bunthai, 2001).

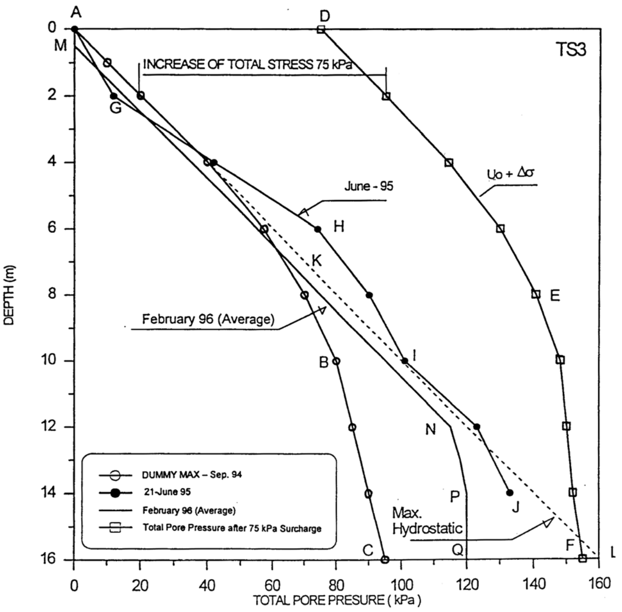

Figure 3. Recharge of piezometric level due to PVD installation at the SBIA site.

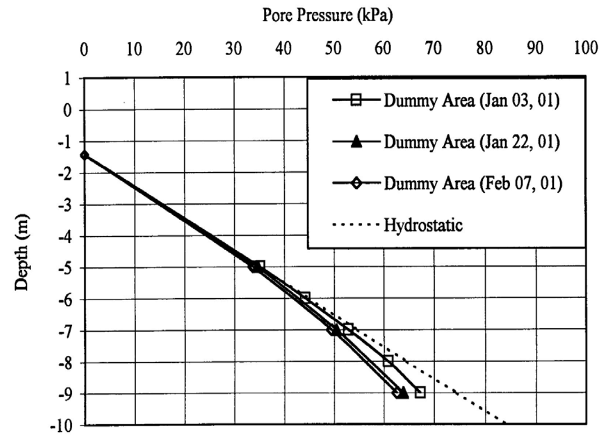

Figure 4. Pore pressure versus depth in dummy area at AIT campus.

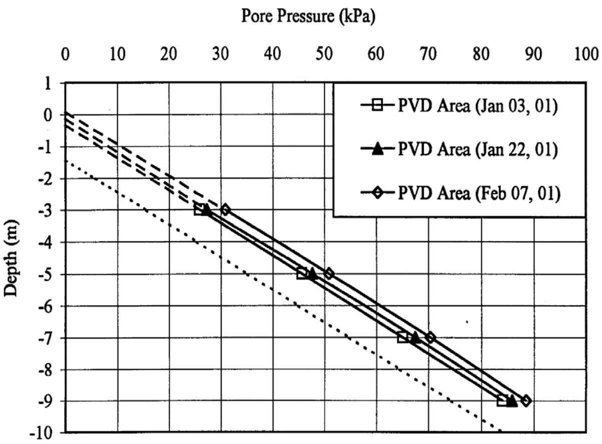

Figure 5. Pore pressure versus depth in PVD area at AIT campus.

The pore pressures at depths of 2, 4, 6 and 8 m from the ground surface were monitored from open standpipe piezometers. After finishing the consolidation, the water was sprayed on PVD area for 6 hours every day in order to study the recharging effect. The pore pressure was measured with time for 4 months. Figure 5 shows the piezometric level in PVD area. It is found that no piezometric drawdown was recorded. Moreover piezometric level lies above the hydrostatic level. As a result, it is confirmed that the PVD is effective in recharging the ground water to remove the piezometric level drawdown up to the depth of PVD installation.

6 CONCLUSIONS

This paper deals with the application of prefabricated vertical drain (PVD) for acceleration of consolidation time and recharge of piezometric drawdown. The method for capturing the settlement of multi-layer subsoil with PVD is also presented. The following conclusions are arising out of this investigation.

1) Based on data from SBIA project, the PVD-SD Version 2.3 is verified for the settlement analysis of the multi-layer subsoil with PVD. It is useful for geotechnical engineers to make a preliminary design with the available laboratory test results.

2) The PVD not only accelerates the consolidation time but also recharges the piezometric drawdown caused by the excessive withdrawal of the groundwater. This recharge phenomenon were observed in two sites of the soft Bangkok clay.

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