

## EVALUATION OF THE PVD PERFORMANCE AT THE SECOND BANGKOK CHONBURI HIGHWAY (SBCH) PROJECT

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**ABSTRACT:** The soft Bangkok clay foundation at the Second Bangkok Chonburi Highway Project (SBCH) was improved using prefabricated vertical drains (PVD). Monitoring instruments such as surface settlement plates, deep settlement plates, inclinometers and piezometers were installed in the subsoil in order to observe the deformation behavior of the embankments under loading. The Department of Highways, Thailand, arranged for the monitoring and documentation of the deformation behavior. These monitored records, supplemented with the laboratory test results, were analyzed in order to verify the in-situ horizontal coefficient of consolidation of the soil as well as the rate and amount of settlement. The maximum surface settlement was calculated using one-dimensional consolidation theory, Skempton-Bjerrum method, as well as Asaoka's method. In addition, a one-dimensional FEM computer software, capable of calculating the consolidation of multi-layered soil, named PVD-SD was also used successfully to predict the rate and amount of settlement. Finally, the monitored deformation behavior was compared with the predictions during the design stage of SBCH to evaluate the performance of PVD. The amount of settlement predicted by Asaoka's method was in excellent agreement with the observed values, whereas the one-dimensional consolidation method, Skempton-Bjerrum method and the PVD-SD FEM method showed some overprediction. The PVD performance at SBCH Project confirmed and validated the ground improvement by preloading and drainage on soft Bangkok clay.

### INTRODUCTION

The Second Bangkok-Chonburi Highway (SBCH) is one of the major infrastructure project recently built in Thailand using prefabricated vertical drain (PVD) as foundation improvement. This motorway is a part of the extensive road construction program planned by the Department of Highways (DOH), Thailand, in order to relieve the heavy traffic in the Bangkok Metropolitan area. This highway is constructed on the way to the Eastern Seaboard Area, in order to divert the traffic, especially heavy trucks and towed vehicles. The general location of this project is shown in Fig. 1 which is adjacent to the site of the Second Bangkok International Airport.

To evaluate the performance of the PVD installed at the Second Bangkok-Chonburi Highway (SBCH), the prediction of the maximum surface settlement as well as the rate of surface settlement by using different methods was done. The predicted settlements during the design phase of SBCH was also compared with their respective field observations. The field and laboratory data were collected from five sections namely: Section 1C/2 (km 15+700 to km 19+600), Section 2A/1 (km 22+000 to km 26+400), Section 2A/2 (km 26+400 to km

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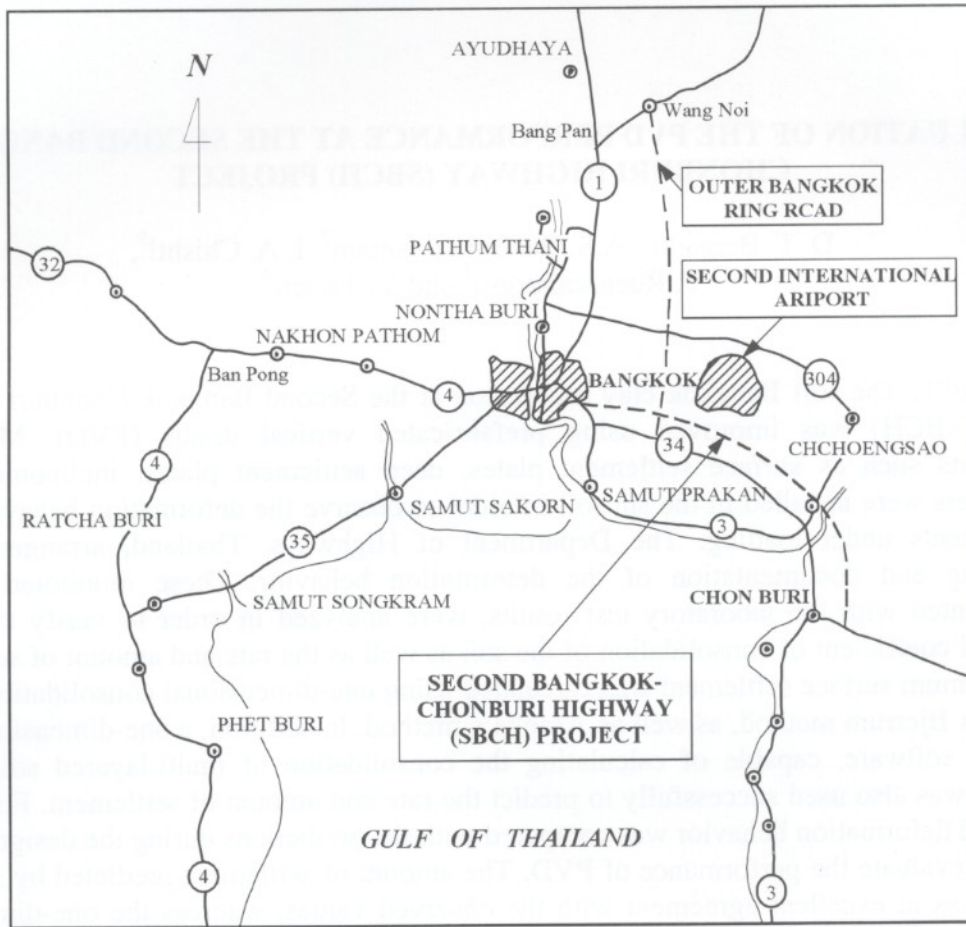


Fig. 1 Location of SBCH Project

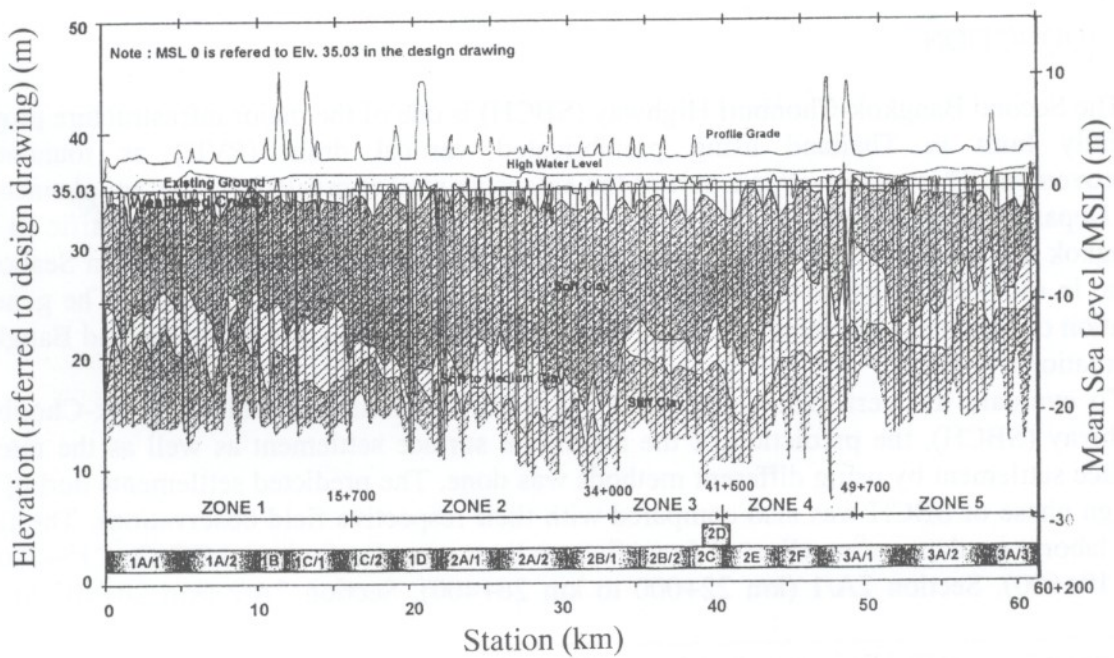


Fig. 2 General soil profile along the PVD portion of Second Bangkok-Chonburi Highway (SBCH) Project

30+700), Section 2B/1 (km 30+700 to km 35+200) and Section 2E (km 41+500 to km 45+450) as shown in Fig. 2. Only Sections 2A/2 and 2B/1 are included here. Most of the data in this paper were taken from the work of Chishti (1998) under the supervision of the first author. The maximum surface settlements were calculated using inverse analysis technique of Asaoka (Asaoka 1978), one-dimensional consolidation method (Terzaghi 1943) and Skempton-Bjerrum method (Skempton and Bjerrum 1957). In addition, a one-dimensional finite element modeling (FEM) technique using the computer software PVD-SD was also used. The rate of settlement was calculated using a combination of one-dimensional consolidation theory (Terzaghi 1943) and Hansbo's theory (Hansbo 1981). Finally, a detailed comparison was made among the calculated values (during present study), monitored values (under actual loading conditions) and predicted values (during design phase of SBCH) of soil parameters as well as rate and amount of settlement, in order to evaluate the performance and effectiveness of PVD at SBCH Project.

## SITE CONDITIONS

The general soil profile along the Bangkok-Chonburi Highway Project is shown in Fig. 2 which also indicates the locations of Sections 2A/2 and 2B/1. The detailed soil profile and soil properties of Sections 2A/2 and 2B/1 are similar. The soil profile and soil properties for Section 2A/2 is shown in Fig. 3. The topmost layer is a hardened weathered crust of about 1 m to 3 m thick underlain by a very soft clay layer, 8 m to 10 m thick. This is in turn underlain by soft clay up to a depth of 18 m to 20 m. Underlying the soft clay is about 2 m thick layer of medium stiff clay followed by stiff to very stiff clay extending until the end of borings. The water level is about 0.5 m below ground level.

The very soft clay layer is characterized by a low unit weight (about  $13.6 \text{ kN/m}^3$  to  $15 \text{ kN/m}^3$ ). The water content ranges from about 130% to 140% with higher values at the upper part of the layer. The liquid and plastic limits vary from 100 to 150 and 35 to 50, respectively, with the lower values found at Section 2B/1. The underlying soft clay has a water content of about 70%. The liquid limit is about 80%, while the value of plastic limit is about 35%. The unit weight is the same as the upper limit of very soft clay layer at  $15.5 \text{ kN/m}^3$ .

Prefabricated Vertical Drains (PVD) were installed at seventeen sections out of the total twenty-one sections of SBCH. The pattern of installation was triangular, having a center to center spacing of 1.2 m. The embankment was constructed in stages. Figure 4 shows the typical embankment cross section and general layout of PVD installation at SBCH. The maximum height of fill was 2.85 m. Monitoring instruments were installed at all construction sections along the whole length of SBCH. The instruments included piezometers, inclinometers, surface settlement plates and deep settlement plates. A total of nine pneumatic piezometers were installed below and outside the highway embankment. Six of these were used as active piezometers, two each at depths of 5 m, 10 m and 15 m. Three piezometers were located below the centerline of the embankment while three are located outside the embankment. The remaining three piezometers, located away from the embankment, were used as dummy piezometers, one each at depth of 5 m, 10 m and 15 m for reference. Three inclinometers were installed at each control section. Two were located at the right and left side of the embankment while the other one is located outside the embankment under the berm. Seven surface settlement plates were installed below the sand blanket 10 m apart. In addition, a surface settlement plate was also installed in the region beyond the influence zone of the embankment load, and was utilized as a temporary benchmark. The settlement caused by the embankment load only was monitored because this temporary benchmark was also subjected to ground subsidence similar to the test embankment.

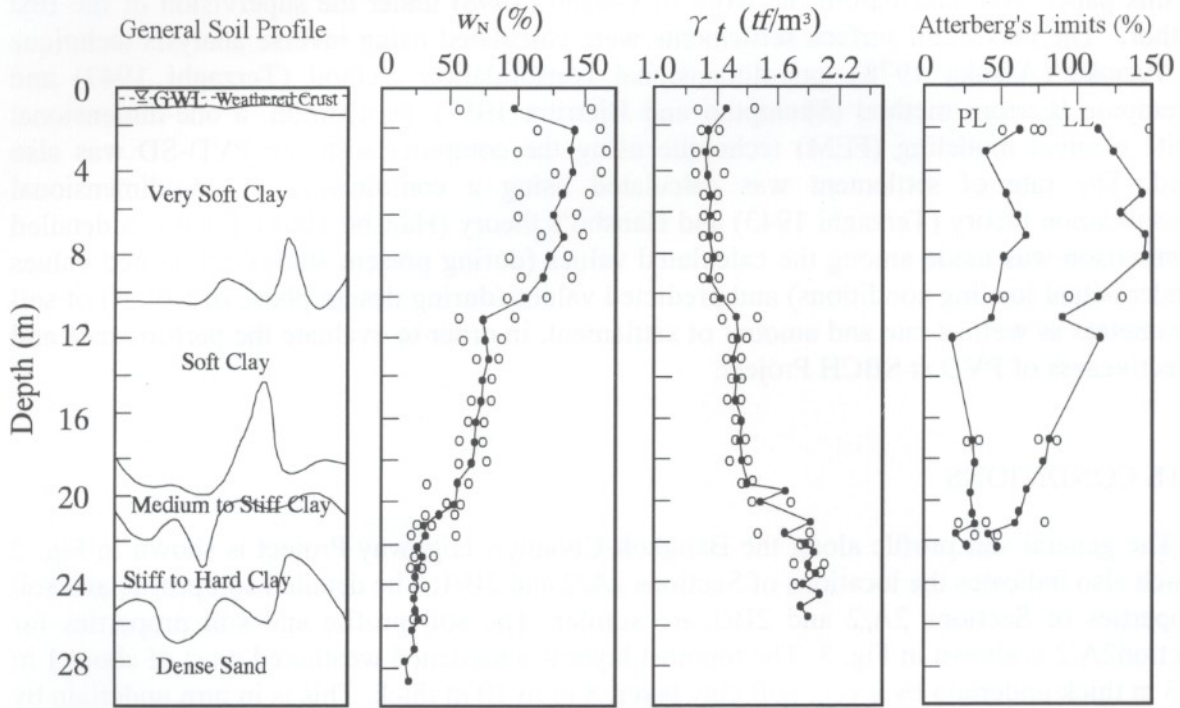


Fig. 3 Index soil properties along Section 2A/2

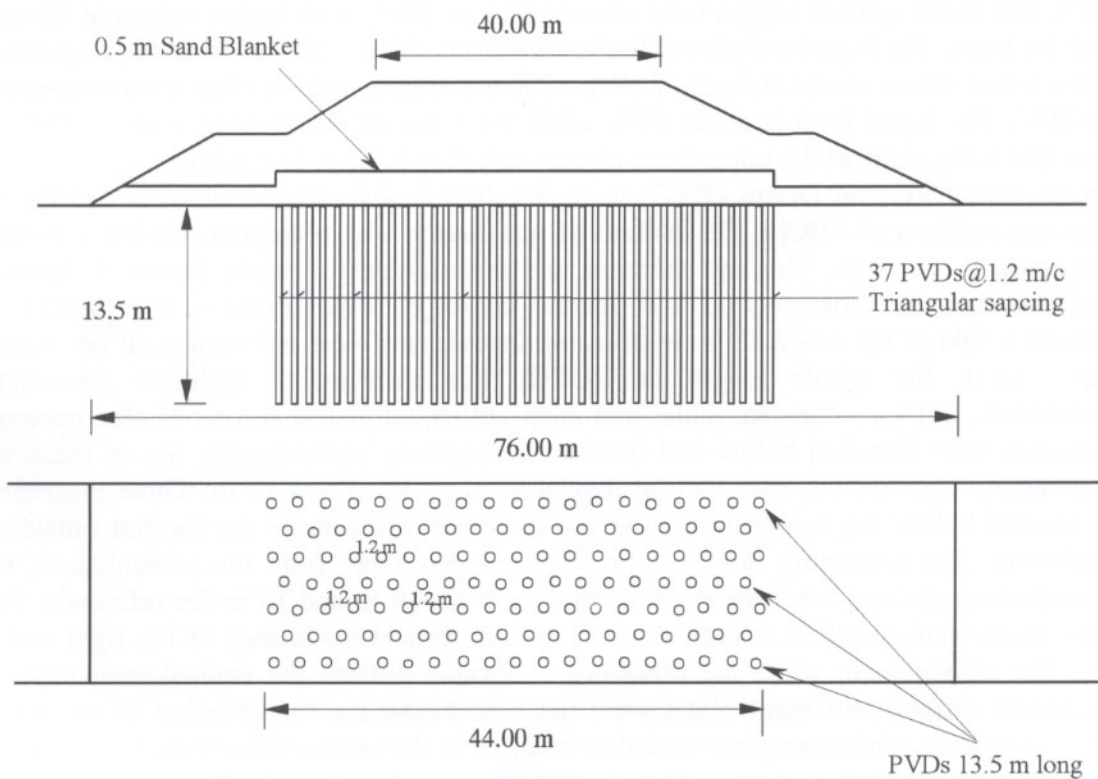


Fig. 4 General layout of PVD installation at SBCH Project

## SETTLEMENT ANALYSIS

For the prediction of amount as well as rate of settlement of the embankment, the load was divided into different stages. The stress increase in the subsoil, due to the embankment load, was calculated using the elastic theory. The settlement of the ground was predicted for each loading stage separately and the results were accumulated in order to predict a complete load-settlement response of the ground. The various approaches and methods used for the prediction of maximum settlement as well as the rate of settlement are described below.

## Amount of Settlement

The maximum surface settlement of the ground was calculated using the conventional methods, i.e. one-dimensional consolidation method (Terzaghi 1943), Skempton and Bjerrum method (Skempton and Bjerrum 1957) and Asaoka's method (Asaoka 1978). In addition, a one-dimensional finite element method computer software, named PVD-SD, was also used for the settlement prediction. The important assumptions made during the application of these methods were as follows: the stiff clay and harder soils were assumed to be practically incompressible, and since the embankment width was very large as compared to the depth of compressible soil, one-dimensional consolidation was more applicable. The immediate settlement was ignored and only primary consolidation settlement was considered in the analysis.

## a) One-Dimensional Consolidation Method

According to the one-dimensional consolidation method, the maximum final settlement,  $S_c$ , can be calculated using the following expression (Terzaghi 1943):

$$S_c = H \cdot \left[ RR \cdot \log \frac{\sigma'_{vm}}{\sigma'_{vo}} + CR \cdot \log \frac{\sigma'_{vf}}{\sigma'_{vm}} \right] \quad (1)$$

where  $\sigma'_{vo}$  is the initial in-situ vertical effective stress;  $\sigma'_{vm}$  is the maximum past pressure;  $\sigma'_{vf}$  is the final effective stress after completion of primary consolidation;  $H$  is the initial thickness of the compressible layer;  $RR$  is the recompression ratio,  $C_r / 1 + e_0$ ;  $CR$  is the compression ratio,  $C_c / 1 + e_0$ ;  $C_r$  is the recompression index; and  $C_c$  is the compression index.

In applying this method for calculation of maximum settlement, all the soil parameters were taken from laboratory tests as tabulated in Tables 1 and 2 for Sections 2A/2 and 2B/1, respectively. The maximum settlement was calculated for each loading stage separately and the results were superimposed at the end to give the cumulative effect of embankment fill taking into consideration the settlement of the embankment as well as the reduction in soft clay thickness due to consolidation. The contribution of confined plastic flow to the magnitude of settlement using the ratio of lateral to vertical consolidation,  $\alpha$ , was also considered in predicting the maximum final settlement. An average value of  $\alpha = 0.24$ , proposed by Loganathan (1992), was used for the analysis. During the immediate settlement,  $\alpha$  was 1.0. Assuming the immediate settlement is 0.50 times the consolidation settlement or 0.33 times the total settlement (ignoring creep settlement), as proposed by Loganathan (1992), the weighted average value of  $\alpha$  used during all stages of loading was 0.50.

$$\alpha_{\text{weighted average}} = \frac{(0.33 \times 1.0) + (0.67 \times 0.24)}{1.0} = 0.50 \quad (2)$$

Table 1 Summary of soil parameters used for settlement analysis of Section 2A/2

Depth (m)	$\gamma_t$ (kN/m <sup>3</sup> )	$w_0$ (%)	$\sigma_{vm}'$ kN/m <sup>2</sup>	CR	RR	$C_{v(\text{field})}$ (m <sup>2</sup> /yr)	$C_{h(\text{field})}$ (m <sup>2</sup> /yr)
0-1.5	14.7	70	50	0.20	0.040	2	4
1.5-10	13.6	130	35	0.51	0.100	2	4
10-12	15.5	75	50	0.45	0.090	2	4
12-13.5	15.5	70	70	0.35	0.070	2	4
13.5-19	15.5	65	90	0.33	0.060	2	4
19-21	17.0	55	130	0.20	0.040	2	4

Table 2 Summary of soil parameters used for settlement analysis of Section 2B/1

Depth (m)	$\gamma_t$ (kN/m <sup>3</sup> )	$w_0$ (%)	$\sigma_{vm}'$ kN/m <sup>2</sup>	CR	RR	$C_{v(\text{field})}$ (m <sup>2</sup> /yr)	$C_{h(\text{field})}$ (m <sup>2</sup> /yr)
0-3	15.0	70	75	0.30	0.060	2	4
3-12	15.0	140	45	0.60	0.120	2	4
12-13.5	15.5	70	65	0.42	0.084	2	4
13.5-17	15.5	60	80	0.32	0.064	2	4

#### b) Skempton and Bjerrum Method

Skempton and Bjerrum (1957) proposed that the method of estimating consolidation settlement ( $S_c$ ), is to apply a correction factor ( $\beta$ ) to the one dimensional laboratory settlement ( $S_{\text{oed}}$ ). This correction factor is a function of the geometry of loading and the pore pressure coefficient 'A' (Burland et al. 1977). The amount of consolidation settlement is calculated as follows:

$$S_c = \beta \cdot S_{\text{oed}} \quad (3)$$

The overconsolidation ratio of a soil is a function of overburden pressure. Hence, it changes as the embankment load is increased. It also varies from layer to layer of the foundation soil. For the present study, the variation of overconsolidation ratio ( $OCR$ ) with respect to soil depth was ignored to avoid the complexity of analysis. The change of  $OCR$  was only used in response to the change in the surcharge load. The values of  $\mu$  used during different stages of loading are shown in Table 3 for Sections 2A/2 and 2B/1.

Table 3 Values of pore pressure parameter ( $\mu$ ) at different loading stages

Section	Loading stage	OCR	$\mu$
2A/2 (28+250)	I ( $H_f = 0.95$ m)	1.18	0.99
	II ( $H_f = 2.10$ m)	1	1
	III ( $H_f = 2.85$ m)	1	1
2B/1 (32+550)	I ( $H_f = 0.95$ m)	1	1
	II ( $H_f = 1.95$ m)	1	1
	III ( $H_f = 2.55$ m)	1	1

c) Asaoka's Method

Using the graphical approach of Asaoka's method (Asaoka 1978), the maximum final settlement,  $\rho_f$ , was calculated by

$$\rho_f = \frac{\beta_0}{1 - \beta_1} \quad (4)$$

where coefficient  $\beta_0$  and  $\beta_1$  were obtained from the plot ( $S_i$  vs.  $S_{i-1}$ ) of measured settlement as illustrated below:

- i) The observed time-settlement curve plotted to an arithmetic scale is divided into equal time intervals  $\Delta t$  (usually between 30 and 100 days; longer  $\Delta t$  yields better prediction). The settlements,  $\rho_1, \rho_2, \dots$  corresponding to times  $t_1, t_2, \dots$  are read off and tabulated.
- ii) The settlement values  $\rho_1, \rho_2, \dots$  are plotted as points ( $\rho_{i-1}, \rho_i$ ) in a coordinate system with axis  $\rho_{i-1}$  and  $\rho_i$ . A 45° straight line with  $\rho_i = \rho_{i-1}$  is also drawn.
- iii) The plotted points are fitted by a straight line whose corresponding slope is read as  $\beta_1$  and its intercept with the ordinate axis is  $\beta_0$ . The point of intersection with the 45° line gives the final consolidation settlement,  $\beta_f$ .

The slope of the fitted straight line, mentioned in step (iii), depends on the mean coefficient of consolidation and the geometry of the consolidating soil mass. It has been observed in practice that points ( $\rho_{i-1}, \rho_i$ ) always define one or more straight lines, each of them corresponding to either primary consolidation or secondary compression under constant loading (Magnan 1981).

The Asaoka's method is based on settlement data monitored from the field. Due to the availability of only surface settlement records for SBCH, the Asaoka's method could be used only to predict the surface settlement. The maximum surface settlement back-calculated by Asaoka's method included the settlement of the whole compressible stratum.

Rate of Settlement

To predict the settlement at any instant of time, the degree of consolidation,  $U$ , at that time is obtained as:

$$U = \frac{\text{settlement in time, } t}{\text{final settlement}} \quad (5)$$

The degree of consolidation, in the above relation, was computed using the Terzaghi's one-dimensional vertical consolidation theory (Terzaghi 1943) and Hansbo's radial consolidation theory (Hansbo 1979).

It was assumed that the presence of prefabricated vertical drains in the ground ensures mostly radial drainage case throughout the depth of PVD. Hence, only Hansbo's theory was used for the analysis. In the unimproved layers, one-dimensional consolidation theory with vertical drainage was used. The rate of settlement was calculated separately, corresponding to the maximum final settlement calculated by all the three methods, namely: one-dimensional consolidation method, Skempton-Bjerrum method and Asaoka's method. The parameters used for the case of Terzaghi's one-dimensional consolidation theory as well as the Skempton-Bjerrum method were all obtained from laboratory tests.

The time-settlement responses predicted by the one-dimensional consolidation method and the Skempton-Bjerrum method were based on the assumption that the load was applied instantaneously. In the laboratory, the load can be applied on the soil instantaneously, which is not possible in the field where the load is applied through gradual increments. Hence, the time-settlement plots based on the one-dimensional consolidation method and the Skempton-Bjerrum method were corrected for the embankment construction period. No such correction is needed in case of the Asaoka's method since it is already based on the field data.

#### PVD-SD FEM Method

In addition to the above-mentioned conventional methods of settlement analysis, the computer software named PVD-SD, based on one-dimensional finite element analysis, was also used. This can handle the problems relating to multi-layered soil strata, and the settlement-time plot was possible to be obtained at different depths. The program can deal with both single layer and multi-layer cases (up to a maximum of six layers).

In case of a single layer case, the maximum settlement of soil is calculated using Terzaghi's one dimensional consolidation theory (Terzaghi 1943). In this program, a normally consolidated ground is considered and the final settlement of the  $i$ th layer ( $S_{fi}$ ) is calculated as:

$$S_{fi} = \frac{\lambda_i}{1 + e_{0i}} H_i \ln \left( 1 + \frac{\Delta p'}{p'_0} \right) \quad (6)$$

where  $\lambda_i$  is the slope of  $e$  vs  $\ln(p')$  plot for  $i$ th layer ( $\lambda = 0.434C_c$  or  $0.434 C_r$ ) depending upon the state of soil being normally consolidated or overconsolidated, respectively;  $e_{0i}$  is the initial void ratio for  $i$ th layer;  $H_i$  is the thickness of  $i$ th layer;  $p'_0$  is the initial effective vertical stress of  $i$ th layer; and  $\Delta p'$  is the applied surcharge.

For multi-layer case, since there is no closed form solution, it is treated numerically. Finite element method is used for this purpose. The effect of PVD or SD is modeled by modifying the continuity equation as follows (Chai et al. 1997):

$$\frac{K_v}{\gamma_w} \frac{\partial^2 u}{\partial z^2} - \frac{8K_h u}{\gamma_w D^2 \mu} + \frac{\partial \varepsilon}{\partial t} = 0 \quad (7)$$

where  $\gamma_w$  is the unit weight of water;  $z$  is the depth;  $t$  is the time;  $\varepsilon$  is the volumetric strain (the vertical strain in 1-D case);  $K_v$  is the vertical permeability of natural ground;  $K_h$  is the horizontal permeability of natural ground;  $D$  is the diameter of influence zone of the PVD; and  $\mu$  is the factor expressing the combined effect of drain spacing, smear and well resistance.



The second term in the above equation is the effect of PVD or SD. It is important to note that for overconsolidated clay, when  $\sigma'_{vf} = (\sigma'_{vo} + \Delta\sigma'_v) < \sigma'_{vm}$ , the settlement in the overconsolidation range was ignored as it is very little as compared to the one in normally consolidated range.

The average degree of consolidation, caused by the vertical drainage due to permeability of natural ground, is calculated using Terzaghi's theory (Terzaghi 1943). The time,  $t$ , to obtain a given average degree of consolidation,  $U_h$ , caused by the horizontal drainage due to PVD or SD is calculated by Hansbo's theory (Hansbo 1979) as follows:

$$t = \left( \frac{D_e^2}{8 C_h} \right) (F_n + F_s) \log_e \left( \frac{1}{1 - U_h} \right) \quad (8)$$

where  $D_e$  is the diameter of the equivalent soil cylinder and is equal to  $1.13S$  for square pattern and is equal to  $1.05S$  for triangular pattern;  $S$  is the PVD spacing;  $F_n$  is equal to  $\log(D_e/d_w) - 0.75$ ;  $F_s$  is equal to  $((K_h/K_s) - 1) \log_e(d_s/d_w)$ ;  $d_w$  is the equivalent diameter of the drain; and  $d_s$  is the diameter of the disturbed zone around the drain. The effect of drain or well resistance has been neglected.

Finally, the average degree of consolidation,  $U$ , due to the combination of vertical and radial flow is calculated by Carillo's equation (Carillo 1942), expressed as follows:

$$U = 1 - (1 - U_v)(1 - U_h) \quad (9)$$

where  $U_v$  and  $U_h$  represent the degrees of vertical and horizontal consolidation, respectively.

## COMPRESSIBILITY PARAMETERS

The values of consolidation coefficients,  $C_{v(\text{lab})}$  and  $C_{h(\text{lab})}$ , were converted into the respective field values using the following conversions:

$$C_{v(\text{field})} = 2C_{v(\text{lab})} \quad (10)$$

$$C_{h(\text{field})} = 2C_{v(\text{field})} = 4C_{v(\text{lab})} \quad (11)$$

The ratio  $C_h/C_v$  is around 2 to 5 according to Hansbo (1979). An important point to be noted here is that the coefficients of consolidation ( $C_h$  and  $C_v$ ) have different values in the field and the laboratory. A high ratio of  $C_{v(\text{field})}/C_{v(\text{lab})}$  and  $C_{h(\text{field})}/C_{h(\text{lab})}$  has been found by many investigators. This high ratio may be caused by the presence of numerous fine sand lenses and silt seams; by the presence of fissures and root holes; and by the multi-dimensional drainage paths created by the effects of groundwater pumping from the underlying aquifers that caused ground subsidence (Bergado et al. 1988). Bergado et al (1992) and Bergado et al. (1996) obtained  $C_{h(\text{field})}/C_{h(\text{lab})}$  ratio of 4. Chai and Miura (1999) suggests that  $(K_h/K_s)_{\text{field}} = C_f (K_h/K_s)_{\text{lab}}$  where  $C_f$  values ranges from 2 to 25. Bergado et al (1993) recommends a value of  $K_h/K_s = 10$  and  $d_s = 2d_m$  where  $d_m$  is the diameter of a circle with an area equal to the cross-sectional area of the mandrel. The  $K_h/K_s$  value used for this study was assumed to be 10 (Likitwanagarn 1996) and the well-resistance effect ( $F_r$ ) was ignored. As tabulated in Tables 1 and 2, a  $C_h$  value of  $4 \text{ m}^2/\text{year}$  was utilized. This  $C_h$  value was obtained by the piezocone tests at the site of the nearby Second Bangkok International Airport Project (Bergado et al. 1996).

## EVALUATION OF PVD PERFORMANCE

This study concerns with the evaluation of PVD performance at Sections 2A/2 and 2B/1 of SBCH. The sections selected were those having the thickest layer of very soft to soft clay and the settlements were maximum throughout the highway. In order to evaluate the performance of PVD at the project, the field settlements as well as fill height were compared with those proposed and predicted by the designers, respectively. The results of the study, along with the comments, are presented in the following sections.

### Instrumentation Results

The excess pore pressure records at Section 2B/1 are summarized in Fig. 5 as a function of time and fill height. The construction sequence shows that the embankment was constructed in five stages, namely: the sand platform (0.5 m thick), sand blanket (0.95 m thick), first loading (1.35 m thick), second loading (1.95 m to 2.1 m thick) and the final loading (2.55 m to 2.85 m thick) stage. The embankments were raised to their final heights within 400 days to 500 days after the start of embankment construction. Figure 5 shows that until 350 days, the pore pressures remained almost constant and then starts decreasing. As the load was increased further, the excess pore pressure built up again but at the end of monitoring, a descending trend is clearly visible. This shows the progress of consolidation with time. The figure also shows that the dissipation of pore pressure during the 2nd stage loading is quite high as compared to the final stage.

The lateral deformation record shows that the subsoil at Sections 2A/2 and 2B/1 experienced lateral deformation at depths of 4 m to 6 m as shown in Fig. 6. This depth corresponds to the very soft clay layer. Due to its high stiffness as well as high strength, the lateral deformation of the top crust was smaller as compared to the soft clay. The layers below the soft clay registered minimal lateral displacements. The ratio of lateral to vertical displacement varied mostly within the range of 0.1 to 0.3.

Figures 7 and 8 show the settlement and the degree of consolidation as a function of time for Sections 2A/2 and 2B/1, respectively. The surface settlement data showed that there was some lag time period after which the effect of PVD started. The effective start of PVD consolidation was characterized by a sharp increase in the rate of settlement. From the settlement record, it was quite clear that the rate of settlement increased quite abruptly as soon as the PVD started working. The degree of consolidation for other sections was observed to reach 90% within 700 days to 800 days. For Section 2B/1, it reached 90% consolidation after 900 days. For Section 2A/2, even after 1000 days, the degree of consolidation was still less than 90%.

### Predicted Settlements

Tables 4 and 5 show the predicted maximum surface settlement of ground at different stages of loading, using Asaoka's method for Sections 2A/2 and 2B/1. The settlements predicted by Asaoka's method represented the settlement of the whole soil stratum (including both improved and unimproved layers), as it is based upon the monitored surface settlement record. The tabulated data also indicated the actual monitored settlement at the respective stages of loading. At the final stage of loading, the subsoil at all sections has gone through about 90% or more degree of consolidation. For settlements under other initial stages of loading, there is no constant degree of consolidation achieved for all sections. At some sections, soil seems to be consolidated more under a certain loading stage whereas at others the degree of consolidation is quite less.

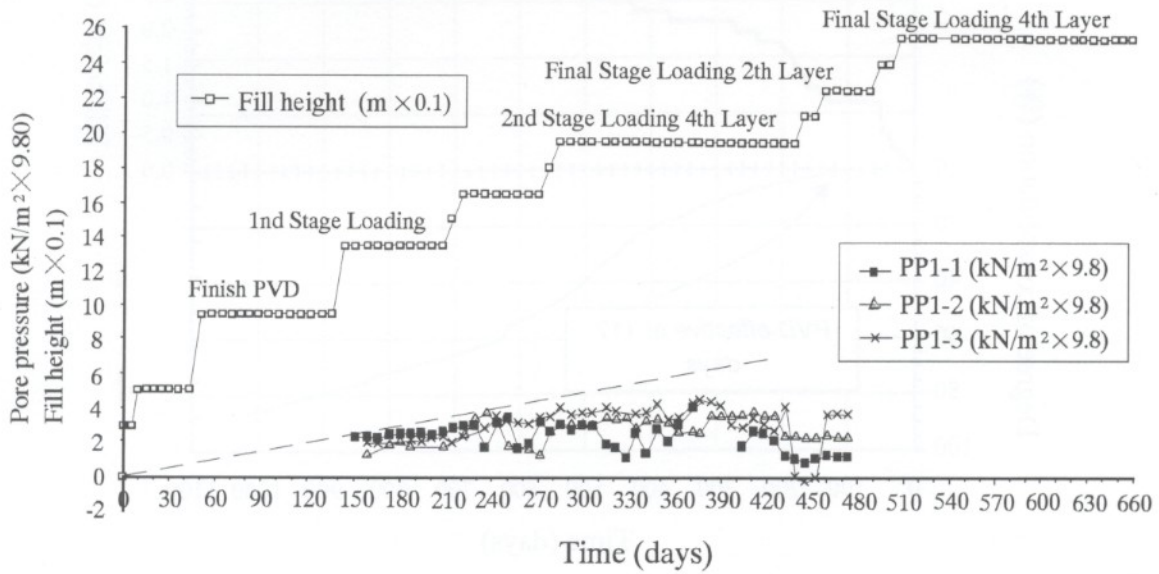


Fig. 5 Excess pore pressure (wet Dummy) under embankment center at Section 2B/1

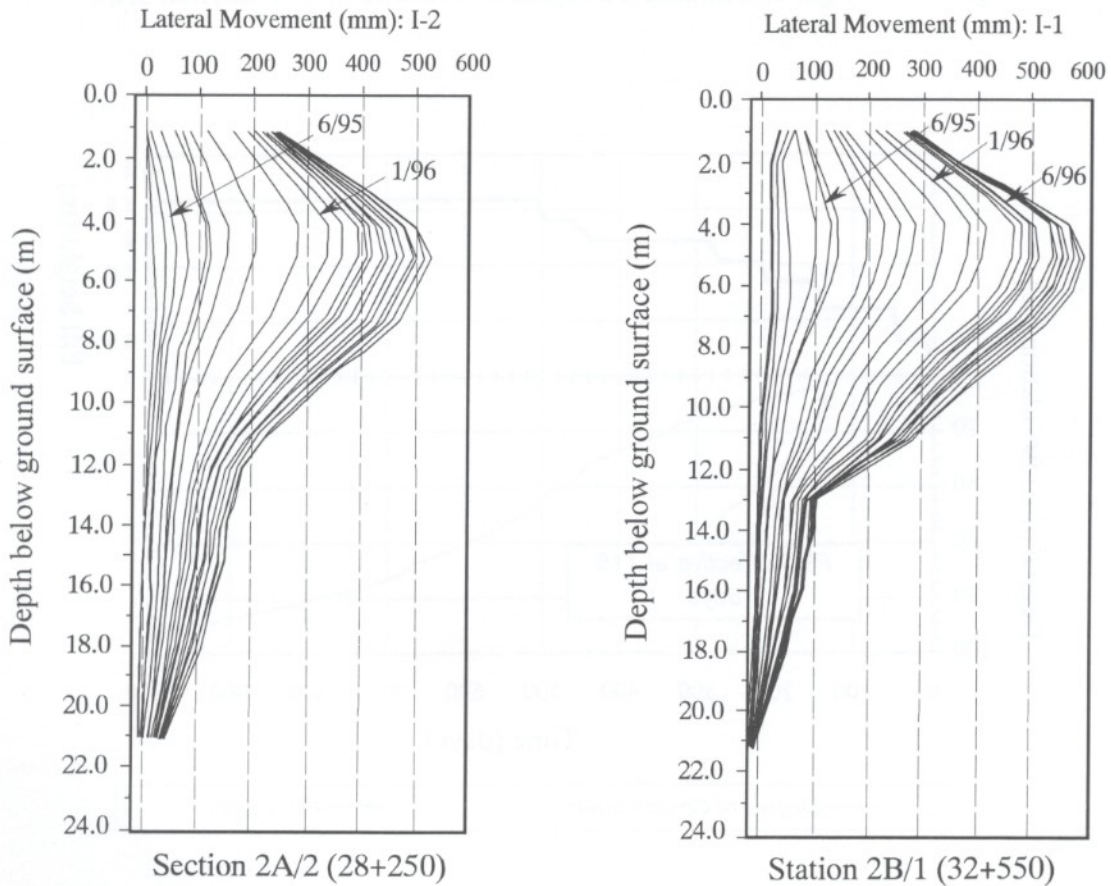


Fig. 6 Lateral deformation at Section 2A/2 and 2B/1

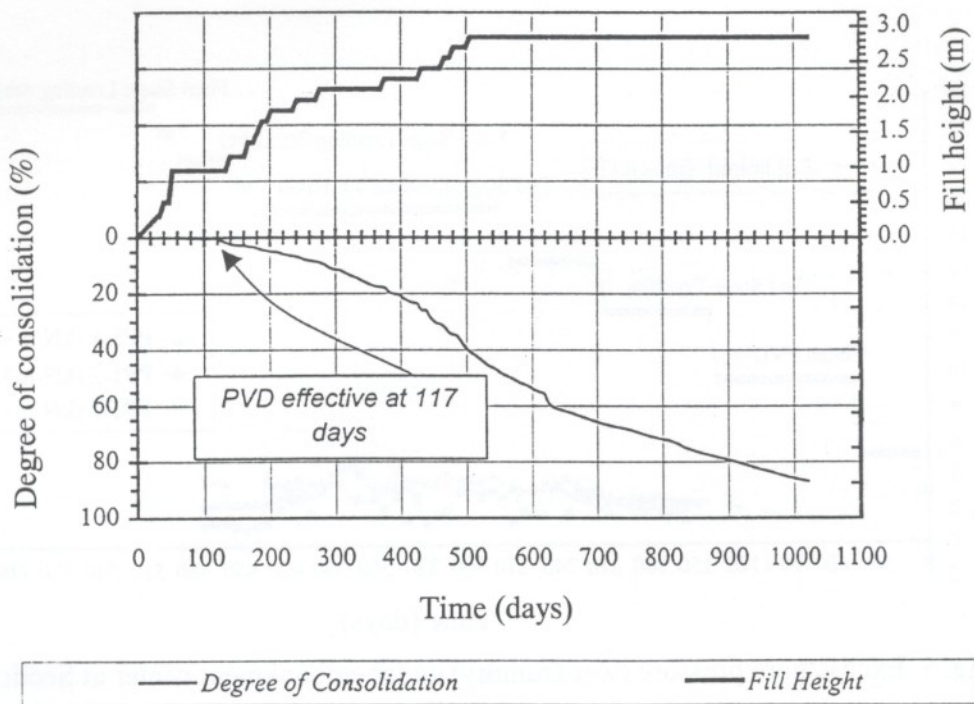


Fig. 7 Fill height and monitored degree of consolidation at Section 2A/2

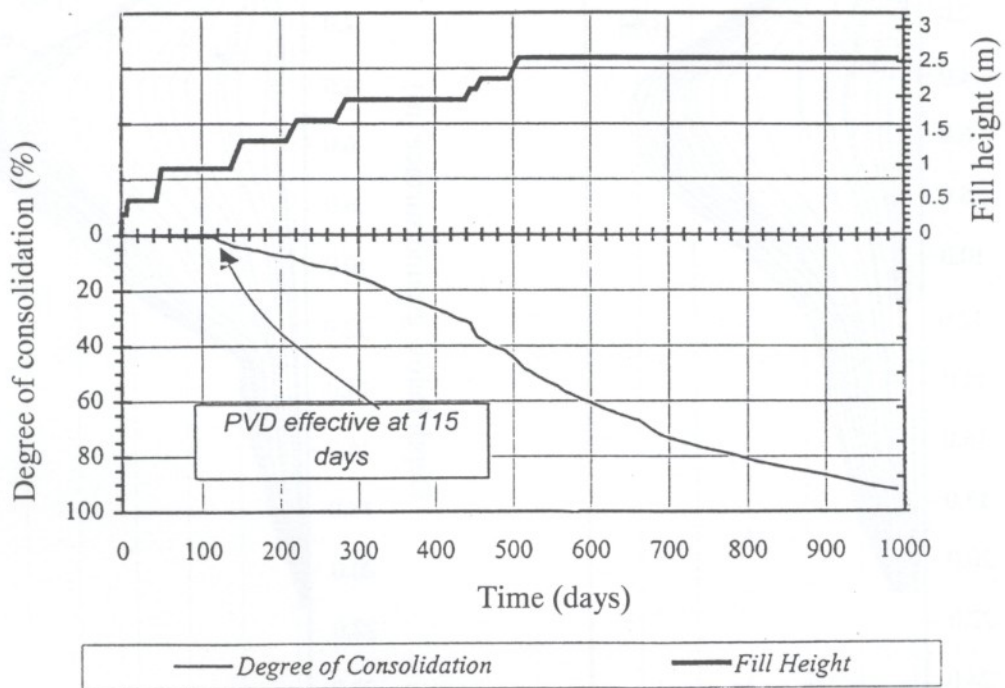


Fig. 8 Fill height and monitored degree of consolidation at Section 2B/1

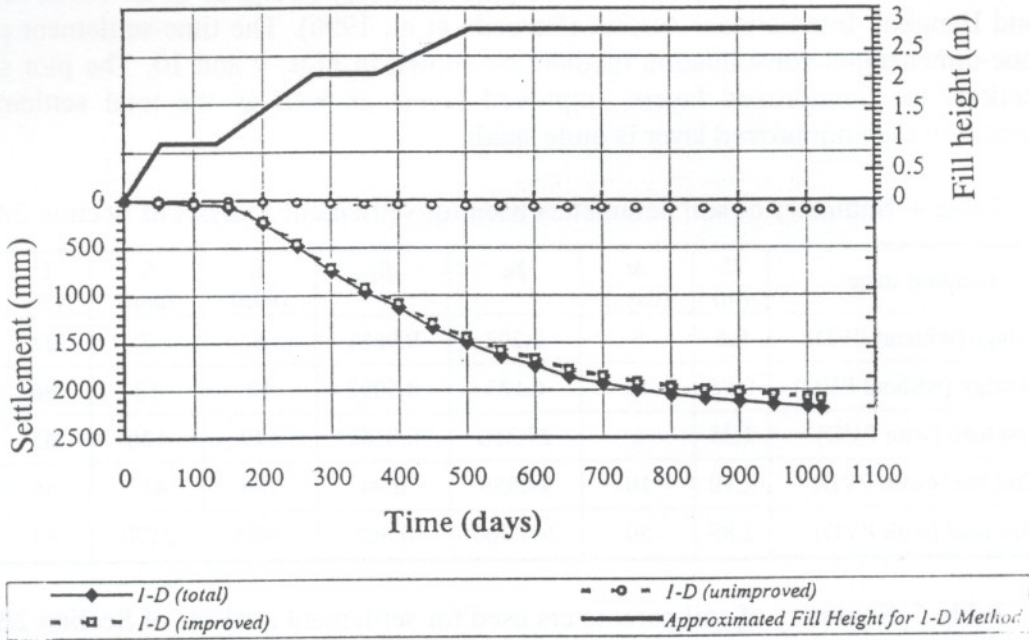


Fig. 9 Time-settlement plot for Section 2A/2 using one-dimensional theory

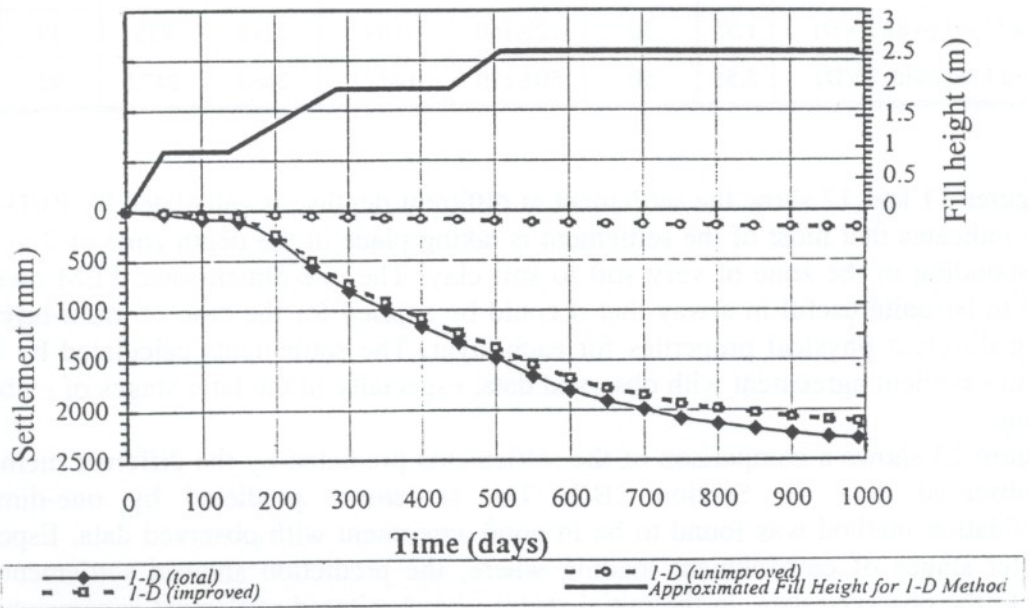


Fig. 10 Time-settlement plot for Section 2B/2 using one-dimensional theory

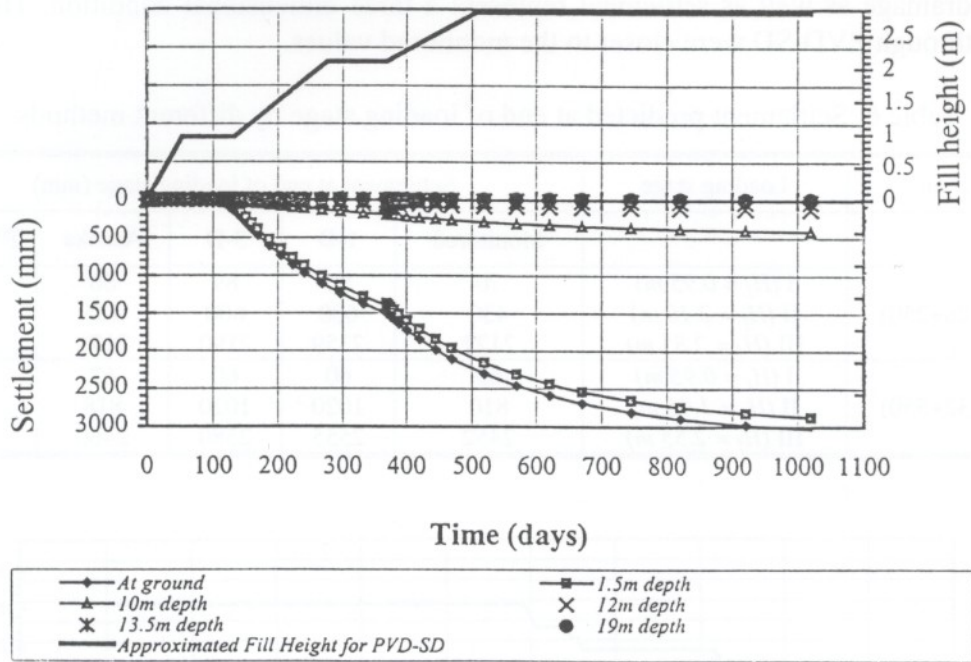


Fig. 11 Settlement of different layers computed used PVD-SD at Section 2A/2

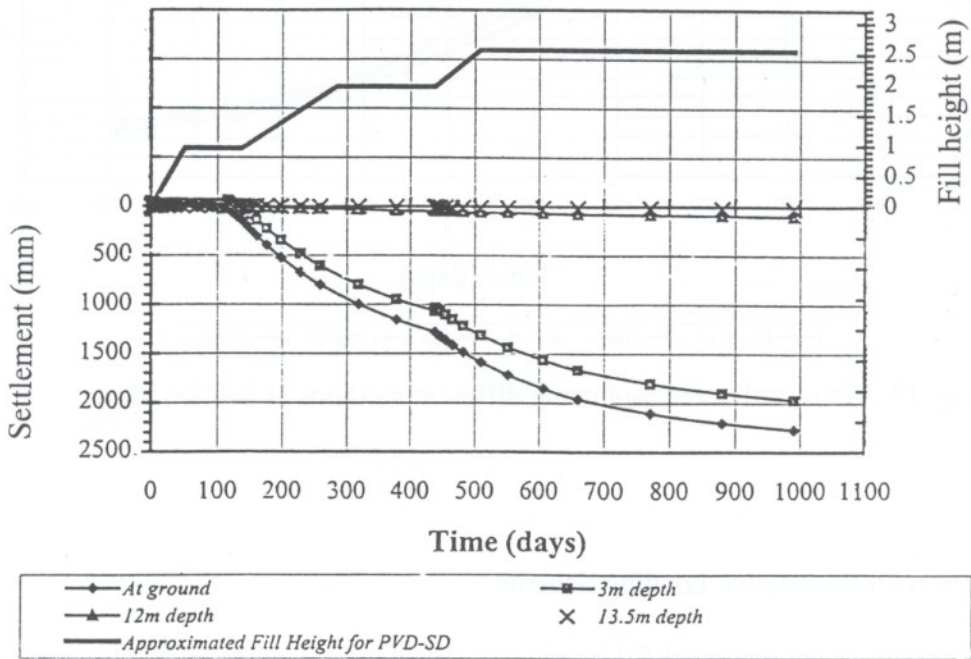


Fig. 12 Settlement of different layers computed used PVD-SD at Section 2B/1

The PVD-SD method yielded very good predictions whereas the one-dimensional method slightly overpredicted the settlements. The slight overprediction may be due to the fact that in field the drainage as well as settlement represent a three dimensional condition. The results obtained through PVD-SD were closer to the monitored values.

Table 6 Settlement predicted at end of loading stage by different methods

Section	Loading stage	Settlement at end of loading stage (mm)				
		Monitored	1-D	3-D	Asaoka	PVD-SD
2A/2 (28+250)	I ( $H_f = 0.95\text{ m}$ )	70	90	89	80	73
	II ( $H_f = 2.10\text{ m}$ )	430	600	600	435	520
	III ( $H_f = 2.85\text{ m}$ )	2172	2359	2190	2175	2182
2B/1 (32+550)	I ( $H_f = 0.95\text{ m}$ )	65	60	60	62	62
	II ( $H_f = 1.95\text{ m}$ )	810	1020	1020	818	976
	III ( $H_f = 2.55\text{ m}$ )	2452	2555	2580	2460	2506

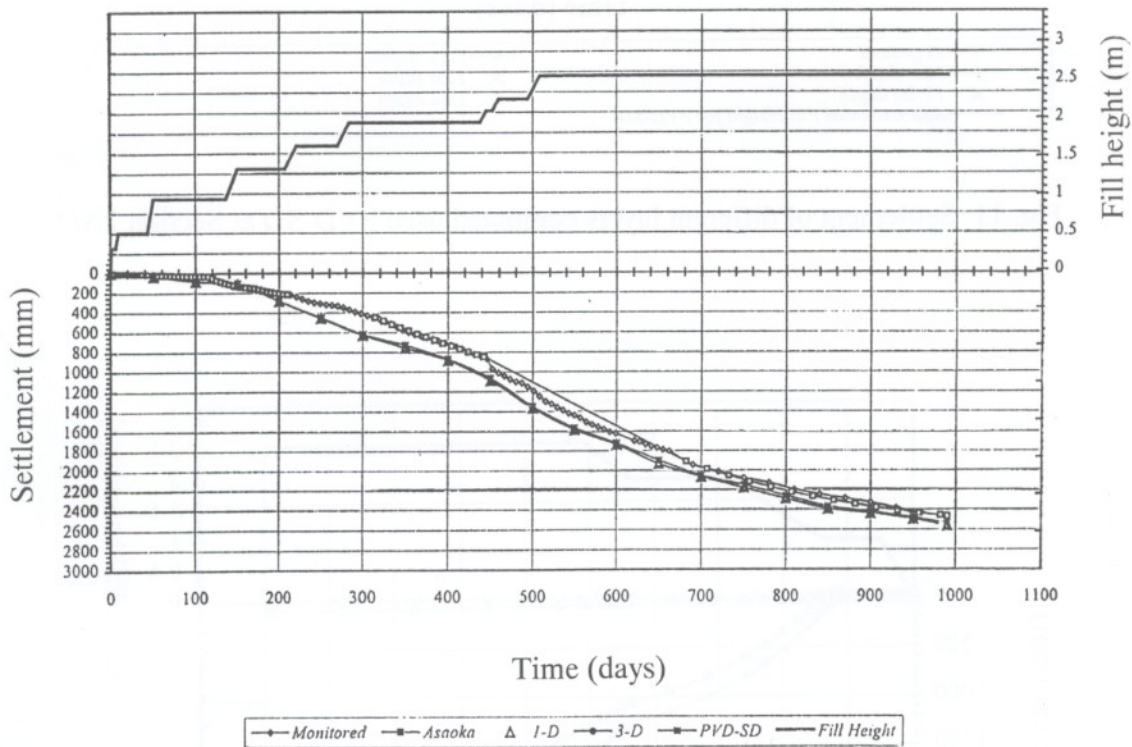


Fig. 13 Time-settlement plot using different methods at Section 2B/1

Comparison with Alternative Loading Scheme

A comparison of the designed fill height and fill thickness with the actual one is shown in Figs. 14 and 15. The height of embankment was designed such that it fulfilled the criterion of the minimum road level of 1.10 m, to be safe against any flood (Ruenkairregsa et al. 1997). During the construction phase, the ideal embankment height was not implemented. The embankment heights in the field were lower. Consequently, the level of embankment after

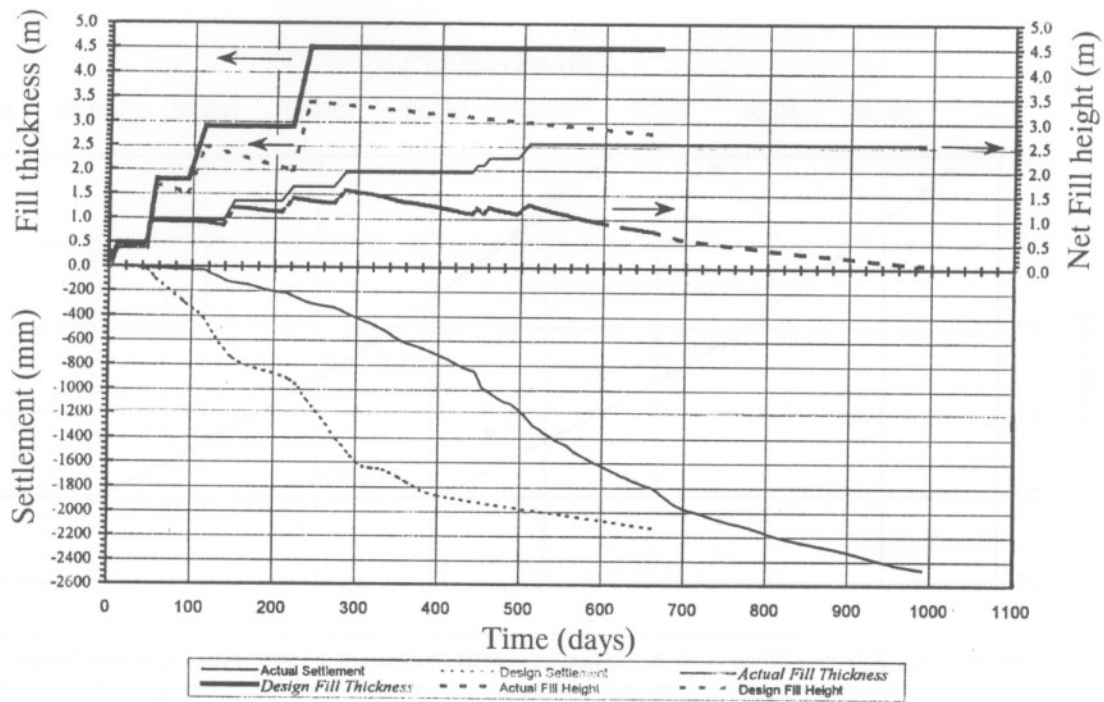


Fig. 14 Comparison of alternative and actual loading schemes for Section 2A/2

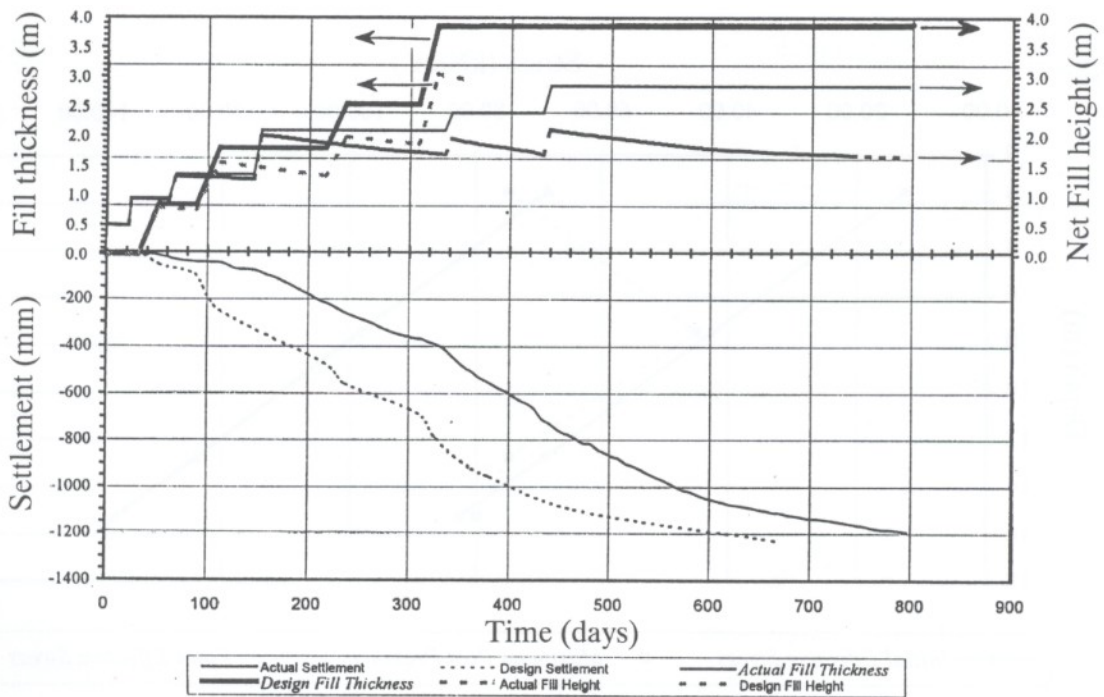


Fig. 15 Comparison of alternative and actual loading schemes for Section 2B/1



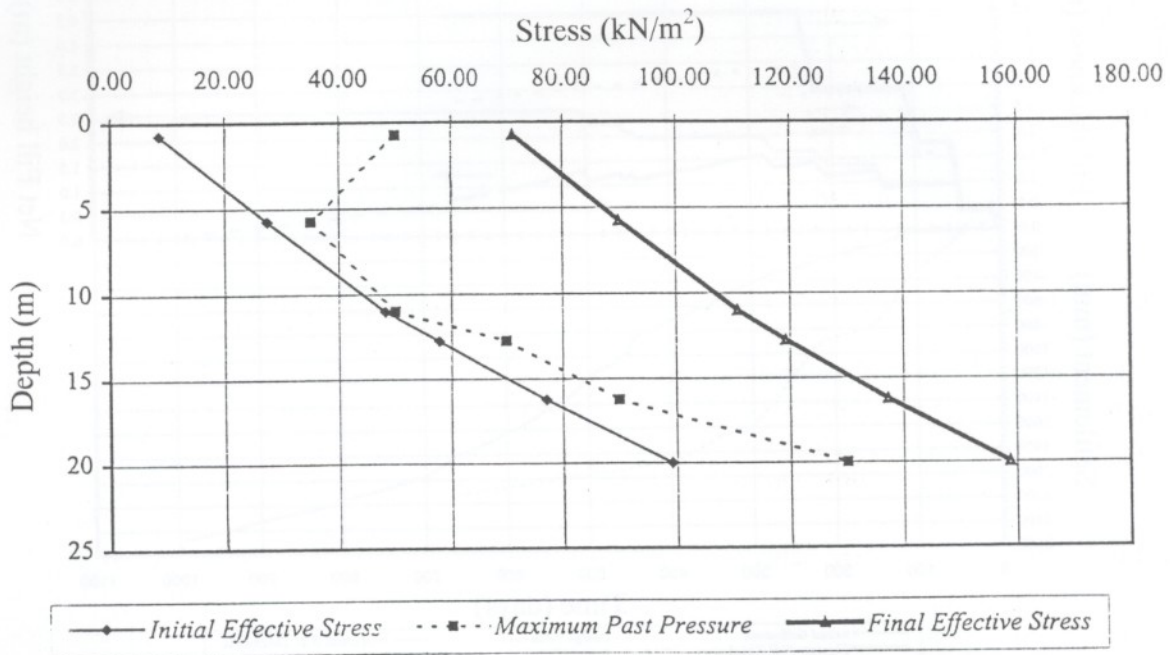


Fig. 16 Variation of effective stress ( $\sigma_{v0}$ ), maximum past pressure ( $\sigma_{vm}$ ) and final effective stress ( $\sigma_{vf}$ ) at Section 2A/2

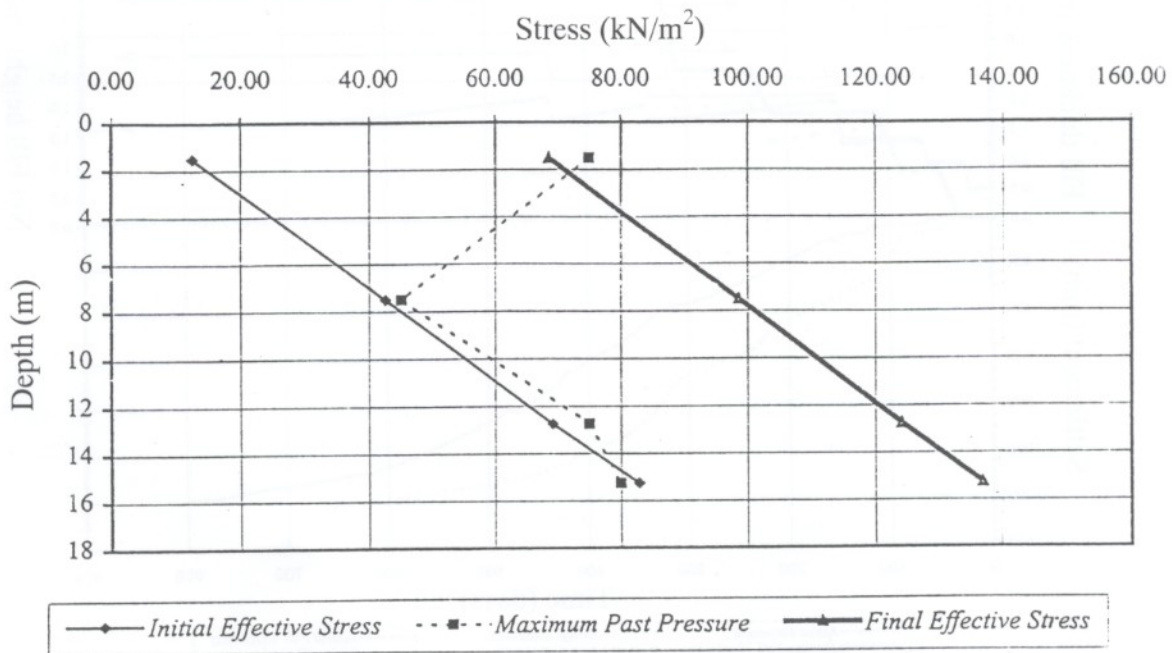


Fig. 17 Variation of effective stress ( $\sigma_{v0}$ ), maximum past pressure ( $\sigma_{vm}$ ) and final effective stress ( $\sigma_{vf}$ ) at Section 2B/1

completion of 90% consolidation was found to be less than that required for protection against flooding.

The recommendation was to achieve 50%, 70% and 90% degree of consolidation in 7 months, 10 months and 19 months, respectively. On the other hand, in actual cases, for Section 2A/2, 50% and 70% degrees of consolidation were found to be achieved in 19 months and 26 months, respectively, whereas 90% consolidation was not achieved until the end of monitoring. A similar kind of response was observed for Section 2B/1. This low rate of actual settlement may be attributed to the insufficient embankment height used in field.

## EVALUATION OF PVD PERFORMANCE AT SBCH PROJECT

Past studies showed that for embankment construction without using PVD in Bangkok area the maximum fill height attained before failure of embankment varied in the range of 1.8 m to 2.2 m (Ruenkairrergsa et al. 1997). Over these heights, cracks were observed followed by excessive settlements and landslides. In other words, it had been nearly impossible to construct the embankment higher than 2.0 m to 2.2 m. On the other hand, in case of SBCH, fill heights up to 3 m were achieved without failure. This evidence clearly showed that PVD proved to be quite effective at the site.

The PVD is believed to be fully effective under loading conditions where high excess pore pressures are induced. The loading is, therefore, recommended to be increased as much as possible so that it exceeds the preconsolidation pressure. Figures 16 and 17 show the variation of maximum past pressure as well as the final effective stress as a function of depth. It was observed that the preloading produced final effective stresses that exceeded beyond the maximum past pressure, throughout the entire length of PVD. An exception was in the top weathered crust.

Calculation of the degree of consolidation at the end of monitoring showed results of 87% to 92% and 44% for the case of with PVD and without PVD, respectively. These results indicate that the degree of consolidation achieved by using PVD is almost double than the case without PVD proving that the PVD is an effective ground improvement at SBCH Project.

## CONCLUSION

This study has been carried out in order to analyze the ground improvement using prefabricated vertical drains (PVD) at the Second Bangkok Chonburi Highway (SBCH) Project. After the settlement analyses by different methods, the performance of PVD was evaluated at selected sections of SBCH Project. The conclusions drawn from the study are as follows:

1. The rate and amount of settlements predicted by Asaoka's method proved to be in good agreement with the observed values. On the other hand, Terzaghi's one-dimensional consolidation theory as well as the Skempton-Bjerrum method resulted in overestimated predictions, especially during the middle stages of embankment loading. The settlements predicted by the one-dimensional FEM computer program PVD-SD proved to be in reasonable agreement with the predicted values.
2. The  $C_h$  value of  $4 \text{ m}^2/\text{yr}$  in the field as obtained from piezocone tests was confirmed to be in agreement between the observed and predicted settlements in the SBCH Project. The values of  $C_{v(\text{field})} = 2C_{v(\text{lab})}$  and  $C_{h(\text{field})} = 2C_{v(\text{field})}$  were also confirmed.
3. It is important to note that the preloading surcharge should exceed the preconsolidation pressures of the underlying soft clay layer during ground improvement with PVD.

4. Ideally, the embankment surcharge is applied in stages to the highest possible height in order to prevent the delay in gaining the required degree of consolidation.
5. The settlement of the unimproved soil proved to be very small as compared to that of improved soil proving the effectiveness of PVD as a ground improvement technique.

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