

THE STUDY OF DISCHARGE CAPACITY OF VERTICAL DRAIN IN BANGKOK CLAY

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Abstract

A simulation scheme for the smear effect and well resistance of Prefabricated Vertical Drains (PVDs) improved ground was proposed in order to investigate the discharge capacity of vertical drain (q_w). Incorporating with the proposed simulation scheme a finite element analysis was performed on a unit cell PVD improved ground at Second Bangkok International Airport. The program SIGMA/W was mainly performed for the present study and the consolidation behavior of soft clay layer was modeled by critical state soil model. According to the analysis results of parametric studies showed that the settlement of filed monitoring is close to the finite element when analyzed with the effects of smear and well resistance with q_w in the rage of 284 to 474 m³/year. Form the back analysis of excess pore water pressure, it found that the q_w in the layers of weather crust and soft clay are in the range of 284 to 474 m³/year and 37 to 46 m³/year, respectively.

Keywords: Prefabricated Vertical Drain, the discharge capacity, Bangkok clay

Introduction

Recently, Prefabricated Vertical Drains (PVDs) is one of generally apply for the soft ground improvement. PVDs significantly decrease the consolidation time. The factors that influence in the efficiency of PVDs are smear effect and well resistance (Hansbo, 1981; Hird *et al.*, 1995). The well resistance of vertical

drain can retard the water flow in drainage path and alternately reduce the consolidation rate of improved soil layer. The previous researches (Bergado *et al.*, 1996 and Chai and Miura, 1999) stated that some factors such as increasing of drain length, deterioration of the drain filter (reduction of drain cross-section), silt intrusion

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into the filter (reduction of pore space), and folding of the drain due to lateral movement are significant influence in well resistance. In general, the well resistance can be expressed in term of the discharge capacity of vertical drain (q_w).

Mesri (1991) defined a well resistance factor, R as the function derived from q_w , horizontal permeability of the undisturbed soils (k_h) and drainage length of PVD (l_d). Lin *et al.* (2000) introduced an interface element to simulate the vertical drain with finite permeability to take the well resistance of PVD into account. Well resistance was automatically considered by the discharge of interface element, which possesses equivalent discharge capacity to actual vertical drain. Hansbo (1979), Bergado *et al.* (1996), and Chai and Miura (1999), investigated the discharge capacity. From the previous researches, the discharge capacity values can be summarized on the wide range as $10 \sim 1,580 \text{ m}^3/\text{year}$ which caused problems in selecting values in the design and analysis. Thus, the value of discharge capacity values in each project must be specifically investigated due to different of the site conditions and drainage material properties.

In order to clarify the discharge capacity values of Bangkok clay, this study investigated by comparing the field monitoring data with the two-dimensional (2-D) plane strain finite element analysis method of unit cell. The program SIGMA/W was mainly performed for the present study and the consolidation behavior of soft clay layer was modeled by critical state soil model. According to the calculation results of parametric studies, the values of the q_w are investigated in the constant values with depth and variable with soil types and depths for PVDs design in typical Bangkok subsoil.

Site Location, Construction Procedures and Instrumentation

The Second Bangkok International Airport (SBIA) is located about 30 km to the east of Bangkok (Figure 1). It occupies an area of approximately $4 \times 8 \text{ km}$. The project site are associated with the problems of low strength and high compressibility of the soft clay strata,

and ground improvement method by using PVDs and surcharge are adopted in the design to improve the soft, compressible subsoil before the development of pavement and other facilities on top of the site soils. From the soil investigation program and laboratory tests, the subsoil condition at can be classified into four strata with the average soil parameters as shown in Table 1.

The PVDs was installed by square configuration with spacing of $1 \times 1 \text{ m}$ and penetrated to 10 m depth from the ground surface. The PVDs geometry and the design parameters associated with numerical analysis are summarized in Table 2. For the analysis, the monitoring data of three area as Apron 1, Apron 2 and Reference areas (Figure 2) were applied.

The field ground improvement work, consists of site preparation (including site clearing/leveling/backfilling), subsoil investigation, instrument installation and monitoring, first layer filter fabric placement (on original ground), sand blanket construction, drainage facilities installation, PVD installation, sand drainage layer construction, second layer filter fabric placement (on sand drainage layer), surcharge fill construction (in two stages) and removal. Drainage facilities include collector pipes,

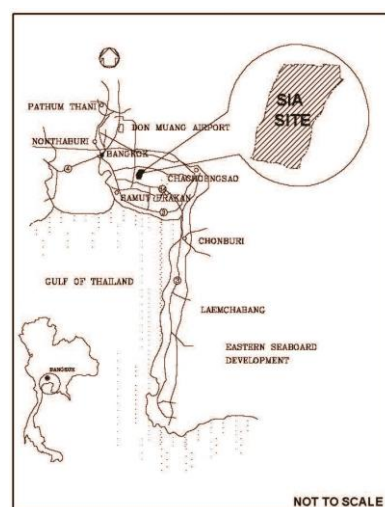


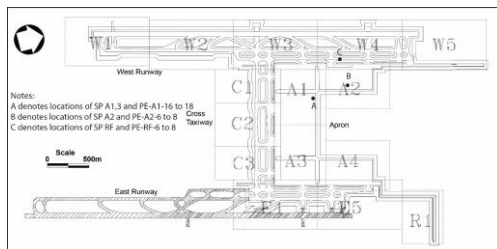
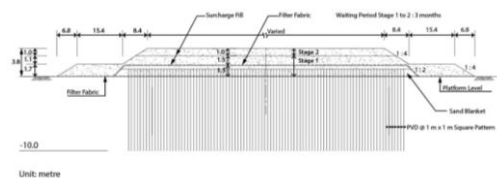
Figure 1. Location of the Second Bangkok International Airport (SBIA)

Table 1. Soil properties of The Second Bangkok International Airport (SBIA)

Soil type	Depth m	γ kN/m ³	Water content, %	LL %	PL %	Su kPa	e_0	Cc	Cs	OCR
Weather crust	0-2	15.7	70	100	30	26.5	1.68	0.98	0.16	2.5
Very soft to soft clay	2-10	13.7	110	115	40	19.6	2.72	1.64	0.29	1.5
Medium stiff clay	10-15	16.0	59	70	30	34.3	2.02	0.99	0.17	1.4
Stiff clay	15-20	17.3	40	70	30	78.5	2.01	0.88	0.20	1.08

Table 2. Geometry and the design parameters of PVDs for analysis

Configuration of PVD Installation	Square Pattern
Spacing of PVD Installation: S×S	1×1 m
Drain Length of PVD: l_d	10 m
Cross Section of PVD: a×b	4×98 mm
Mandrel Dimension: l×w	125×45 mm
Equivalent diameter of the mandrel: $d_m = 2r_m$	84.6 mm
Diameter of Smeared zone ($d_s = 2d_m = 2r_s$)	169.2 mm
Diameter of Influence Zone: ($d_i = 1.13S$)	1130 mm
Equivalent Diameter of PVD: $d_w = (a+b)/2 = 2r_w$	51 mm
Spacing ratio of PVD: $n = d_i/d_w$	22.2
Width of unit cell plain strain $2B = 2R = d_e$	1130 mm
Width of drain in plain strain: b_w	0.90 mm
Width of smear zone in plain strain: b_s	9.95 mm

**Figure 2. Location of apron A1, Apron 2, and reference area****Figure 3. PVD improvement procedure and construction sequence of embankment surcharge at apron A1, Apron 2, and reference area section**

sub-drainage pipes and manholes were installed. Therefore, the generated pore water under preloading would not only be drained

out from the sand blanket/drainage layer, but also be collected into the manholes to be pumped out. Sand blanket was designed to drain out the excess pore water as well as being the working platform for PVD installation. The surcharge fills by using crushed rock was placed in two stages to reach the final preloading height. The counterweight berms with 1.7 m height and 15.4 m width were constructed to stabilize the preloading embankment. The embankment construction in the first stage was to reach a total of 2.8 m fill thickness followed by 3 months waiting period for increase of subsoil strength prior to the next stage of construction. Final stage loading was then placed up to 3.8 m or 4.2 m as shown in Figure 3 and Table 3.

Surface Settlement Plates (SP) were installed above sand blanket layer (except for the Reference Section where SP were installed on the original ground) to measure vertical settlement under surcharge fill.

Electric Piezometers (PE) were installed at various depths to monitor the dissipation of excess pore water pressure. Dummy instruments including piezometers and deep settlement gauges, were installed at a distance

Table 3. Construction Detail of PVD Improvement and Embankment Surcharge

Stage	Construction Type	Duration (days)	Activities
1	1 st filter fabric + layer of sand blanket	7	first layer filter fabric placement (on original ground) + Place sand blanket with thickness of 0.5 m and used as the working platform for the installation of drainage instrumentation
2	2 nd layer of sand blanket + the installation of PVD	14	Place sand blanket with thickness of 0.5 m and used as the working platform for PVD installation + installation of PVD
3	Drainage layer	10	Provide the drainage path with thickness of 0.5 m
4	1 st stage loading	90	Place crush rock surcharge with thickness of 2.8 m with counterweight berm 1.7 m
5	2 nd stage loading	90	Place crush rock surcharge with thickness of 1.0 to 1.5 m

Table 4. Input parameter of various soil models for 2 D numerical analysis of pvd improved ground

Depth (m)	Soil type	Soil Model	γ (kN/m ³)	E_0 (kPa)	ν	C (kPa)	ϕ (deg.)
0-2	Weathered Crust	Mohr-Coulomb	15.7	1.32×10^4	0.30	30	28
10-15	Medium Stiff clay	Mohr-Coulomb	16.0	1.72×10^4	0.30	20	30
Depth (m)	Soil type	Soil Model	γ (kN/m ³)	κ	λ (v)	e_{cs}	M
2-10	Very soft to soft clay	Modified Cam-Clay	13.73	0.13	0.71 (0.31)	5.27	0.90
Thickness (m)		Fill Type	γ (kN/m ³)				
1.0 (0.5+0.5)		Sand Blanket	18.4				
0.5		Sand Drainage	18.3				
2.3 (1.3+1.0)		Crushed rock	21.0				

of 30 m from the toe of the preloading embankment to provide base readings. A typical instrumentation profile is shown in Figure 4. Monitoring work was carried out mainly on weekly basis and before and after each change in loading until the approval of surcharge removal.

Soil Modeling

The soil modeling parameters for finite element analysis are shown in Table 4. For the weathered crust and the stiff clay, Mohr-Coulomb plasticity model was used for numerical simulation. Modified Cam-Clay model was applied for the soft clay layer.

SIGMA/W Setup

SIGMA/W is a finite element software product that can be used to perform stress and deformation analyses of earth structures and

the pore-water pressure generation and dissipation in a soil. In Sigma/w, when 385 nodes and 320 elements (quadratic) as shown in Figure 5 is employed for the two-dimensional (2-D) plane strain of PVD unit cell. Whereas when using a 4-node soil elements (quadratic).

Both vertical and horizontal movements of the bottom boundary of unit cell is restrained, while the side boundaries are free to move vertically but restrained horizontally. A uniform pressure loading is applied at the upper boundary and which is the only drainage boundary during the consolidation analysis. The geometry model and the corresponding finite element mesh for PVD unit cell with 1.0 m drain spacing are displayed in Figure 5. The drain is in the middle of the cell. Two stages of analyses are applied. The first staged of analysis is "In situ" that is formulated

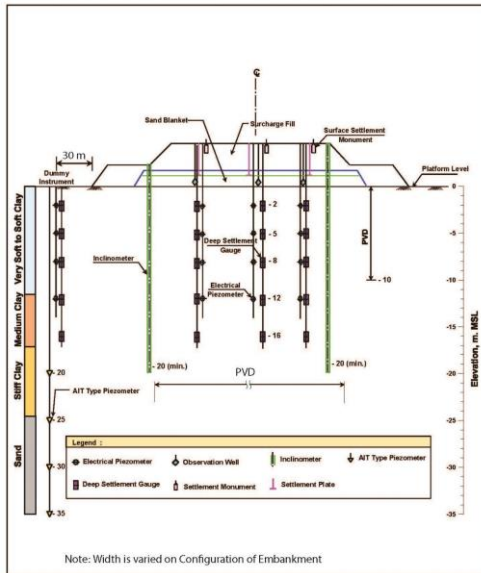


Figure 4. Typical cross section of instrumentation and embankment construction at apron A1, Apron 2 and reference area section

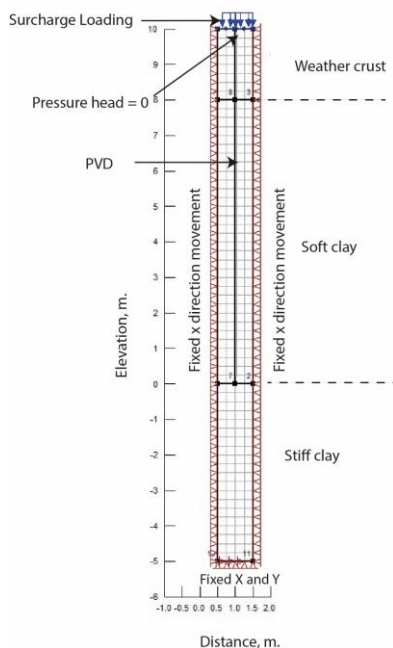


Figure 5. SIGMA/W modeling for unit cell of PVD 2-D plain strain

specifically for establishing the initial stresses. The second is “Coupled Stress-Pore Pressure Analysis”. The fully coupled analysis requires that both the stress-deformation and seepage dissipation equations be solved simultaneously. All hydraulic properties and boundary conditions can be developed and applied from within SIGMA/W.

In the case of perfect drain, meaning that there is no head loss in the drain or no well resistance. This condition can be modeled with a specified boundary condition. Making the boundary condition a total head equal to 10 m says that the pressure distribution will be perfectly hydrostatic at all times. The pore-pressure across the top is set to zero to represent the water table at the ground surface.

In order to investigate the well resistance, the drain itself can actually be modeled with what are known as interface éléments. The drain is included with special finite elements with their own properties. The drain in this case is sometimes referred to as a well. For this analysis the drain (interface) is given the same mechanical properties as the surrounding soil. This is to say that the mechanical stiffness of the drain has no effect on the behavior. The well resistance can be modeled with an equivalent hydraulic conductivity. A very high K value means little or no resistance and a low K would mean some well resistance. The Loading with the different stages of construction are applied in function of loading with time as shown in Figure 6.

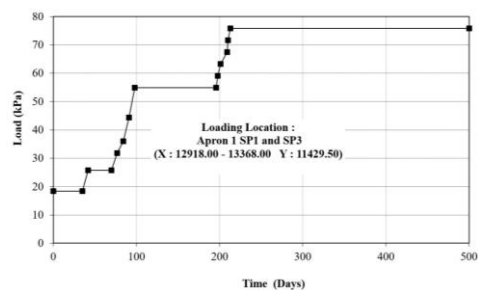


Figure 6. Loading with time at location apron 1 from the monitoring of settlement point SP1 and SP3 (X: 12918.00 - 13368.00, Y: 11429.50)

Table 5. The coefficient of permeability of the soil for numerical analysis

Soil Type	k_h (m/day)	k_v (m/day)	k_{hp} (Eq1) (m/day)	k_{sp} (Eq2) (m/day)	k_{blend} (Eq3) (m/day)
Weathered crust	2.1×10^{-4}	1.3×10^{-4}	6.1×10^{-5}	8.7×10^{-6}	3.7×10^{-5}
Soft clay	6.9×10^{-4}	4.2×10^{-4}	2.0×10^{-4}	2.8×10^{-4}	1.2×10^{-4}

Drainage Parameters of PVDs

Jamiokowski *et al.* (1983) investigated that the permeability of natural deposit clay in horizontal direction, k_h , is normally higher than in vertical direction, k_v , and the (k_h/k_v) ratio can various from 1 to 15. Seah *et al.* (2004) presents the result of k_h/k_v are in the range of 1 to 2 which are determined from the constant rate of strain consolidation tests with radial drainage, standard oedometer tests and the field through testing, along with back analysis from settlement measurement of embankment constructed with prefabricated vertical drains at the Suvarnabhumi Airport site in Bangkok. Then it can generally be assumed that $k_h/k_v = 1.6$ in this study.

In order to investigate the effect of well resistance in the plain strain 2-D, the drainage parameters are applied as shown in Table 5. The simplified ratio of plane strain to axisymmetric permeability is readily obtained, as proposed earlier by Hird *et al.* (1992)

$$\frac{k_{hp}}{k_h} = \frac{0.67}{[\ln(n) - 0.75]} = 0.29 \quad (1)$$

where k_h = horizontal permeability of soil
 k_{hp} = plain strain horizontal permeability of soil

n = (d_e/d_w) ratio of the drain spacing (d_w) to the equivalent drain thickness (d_e) when the drain spacing is equal to 1.13 m and the equivalent drain thickness is equal to 0.05, then n is equal to 22.2. When representing n with 22 in the equation (1), k_{hp}/k_h is equal to 0.29

For the smear effect, Bergado *et al.* (1991) reported in soft Bangkok clay that the ratio of the horizontal permeability of the undisturbed zone to that of the smear zone, (k_h/k_s) in laboratory, varied between 1.5 and 2.

If the k_h value can be replace as to 1.6 k_v from this study. The ratio of k_v/k_s can be assume as 1. Moreover, according to the oedometer tests performed on samples taken shortly after drain installation on reconstituted soft Bangkok clay at different distances from the drain, it can be found that the horizontal permeability of smeared zone, k_s , can be equivalent to vertical permeability of the undisturbed soils, k_v , determined by oedometer tests. The experimental results have been presented by Hansbo (1987). Then in this study, it is possible to apply ratio of vertical permeability to permeability of smeared zone (k_v/k_s) to 1 when considering the smear effect in the analyzed model.

The horizontal permeability of soil in smear zone (k_s) can be proposed in plain strain (k_{sp}) as shown in Equation 2 (Indraratna and Redana, 2000)

$$\frac{k_{sp}}{k_{hp}} = \left(\frac{k_{hp}}{k_h} \right) \frac{\beta}{\left[\ln\left(\frac{n}{s}\right) + \left(\frac{k_h}{k_s}\right) \ln(s) - 0.75 - \alpha \right]} \quad (2)$$

where

$$\alpha = \frac{2}{3} - \frac{4b_s^3}{2B^3} + \frac{2b_s^2}{B^2} - \frac{2b_s}{B}$$

$$\beta = \frac{b_s^2}{B} - \frac{b_s^3}{3B^3} - \frac{2b_w^3}{3B^3} - \frac{2b_s b_w}{B^2} + \frac{b_w^2}{B^2} + \frac{b_w^2 b_s}{B^3}$$

$$b_w = \frac{\pi r_w^2}{2S}$$

$$b_s = \frac{\pi r_s^2}{2S}$$

k_{sp} = plain strain horizontal permeability of soil in smear zone

The vertical drain system may be converted into equivalent parallel drain walls by adjusting the spacing of the drain wall and the coefficient of permeability of the soil. The appropriate conversion was conducted by assuming the plain strain unit cell to have a width of 2B. In a square pattern at a spacing of

S, the width of the drain and the smear zone may be expressed by b_w and b_s the details are shown in Table 4 and Figure 7.

In order to consider the effect of smear, the model is applied the effect of a smear zone by treating the soil as a layered system with a blended horizontal (K_{blend}) permeability. For a layered system like this an equivalent permeability can be computed as Equation 3 (Freeze and Cherry; 1979). Diameter of smeared

zone (d_s) is applied 2 times of equivalent diameter of the mandrel (d_m).

$$K_{blend} = \frac{d}{\left(\frac{d_1}{k_1} + \frac{d_2}{k_2}\right)} \quad (3)$$

The blended equivalent K then is 3.65×10^{-5} and 1.19×10^{-4} m/day for weathered crust and Soft clay layers, respectively. This information can be represented the conductivity of the native clay together with the smear zone rather than create separate

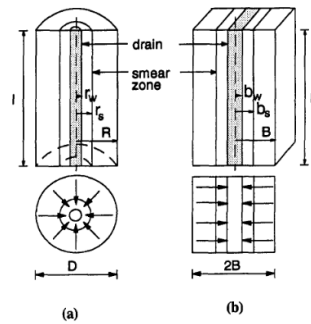


Figure 7. Conversion of axisymmetric unit cell in to plane strain: (a) axisymmetric radial flow; (b) plane strain (indraratna and redana, 2000)

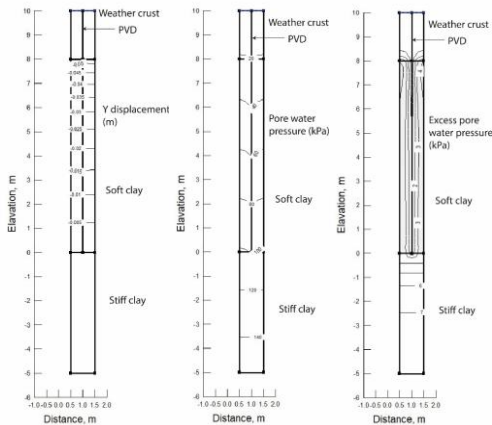


Figure 8. The contour of finite element analysis at Location Apron 1 use the loading pattern from the monitoring of settlement point SP1 and SP3 on 56 days (a) Settlement (b) pore water pressure (c) excess pore water pressure

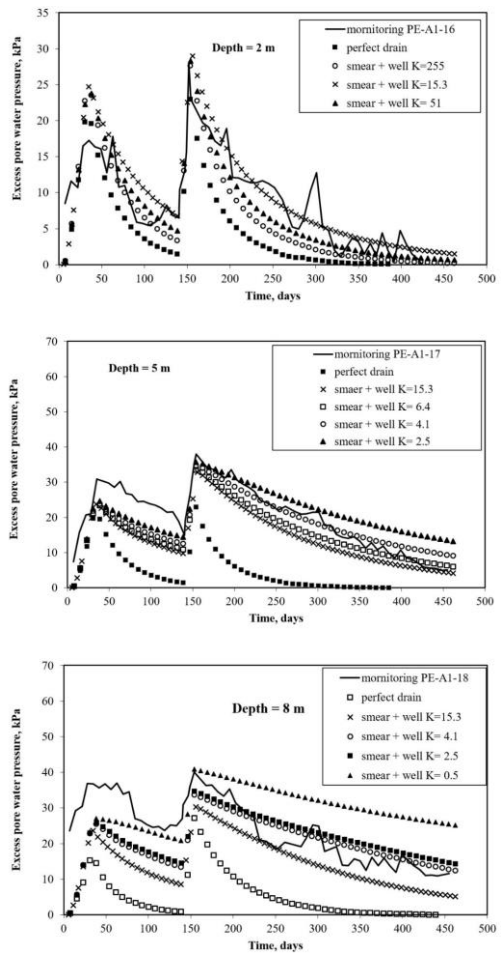


Figure 9. Variation of excess pore water pressures at Apron 1 for Piezometer at (a) 2 m, (b) 5 m and (c) 8 m below ground level. (X=13368, Y=11429.50)

geometric regions for the two different zones. This makes the numerical modeling easier.

Results and Discussions

Form the finite element analysis at location Apron 1 by SIGMA/ W, the contour of settlement, pore water pressure, and excess pore water pressure on 106 days can be plotted as showed in Figures 8(a), 8(b), and 8(c), respectively. Figure 9 showed the variation of excess pore water pressures at Apron 1 for piezometers at 2 m, 5 m, and 8 m below ground surface. Figure 10 showed the settlement of ground surface at Apron 1 by settlement plates SP1 and SP3 compared to the analysis results. The results of perfect drain analysis showed that the settlement of analysis is greater than which of the measured value. The settlement of the monitoring is nearly the same with the analysis which is taking into account in smear effect and well resistance by setting the permeability of vertical drain (k) equal to 15.3 m/day. Figure 9 showed the monitoring of excess pore water pressure at Apron 1 from piezometers PE-A1-16, PE-A1-17, and PE-A1-18 at depths of 2 m, 5 m, and 8 m, respectively compare to the analysis results with various conditions. The excess pore water pressure of perfect drain analysis is lower than which of measured value especially at depth 5 m and 8 m. At 2 m depth, the excess pore water pressure monitored from PE-A1-16 is

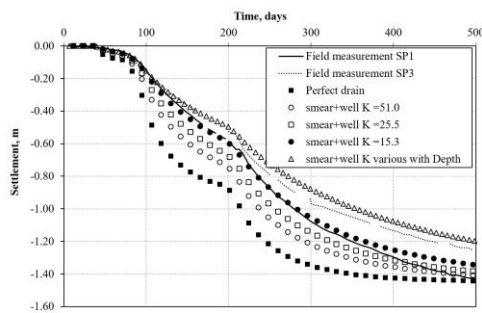


Figure 10. Consolidation settlement at ground surface at Apron 1 form settlement plate SP1 and SP3 (X: 12918.00 - 13368.00, Y: 11429.50)

nearly the same which of the analysis result of smear effect and k equal to 15.3 m/day. The excess pore water pressure of monitoring from PE-A1-17 at 5 m and PE-A1-18 at 8 m depths are nearly the same with which of the case analysis of smear effect and k equal to 4.1 m/day. From the results of the analysis, they showed that the permeability of vertical drain is variable according to soil properties and depth. The settlement of analysis result which setting k equal to 15.3 m/day and 4.1 m/day in weather crust layer (0-2 m) and soft clay layer (2-10 m), respectively, as shown in the symbol “K various with depths”, is somewhat less than which of the field monitoring result.

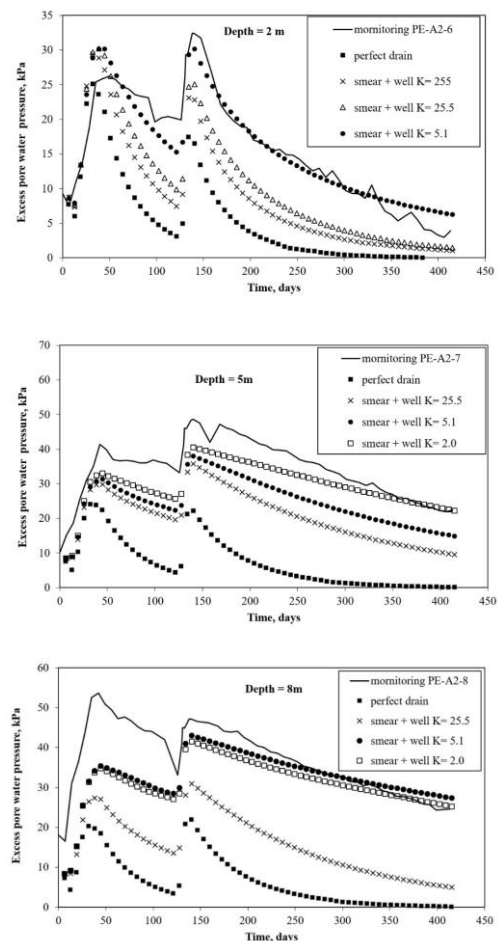


Figure 11. Variation of excess pore water pressures at Apron 2 for Piezometer at 2 m, 5 m and 8 m below ground level. (X: 13762.00, Y: 11529.00)

Figure 11 showed the monitoring of excess pore water pressure from piezometers PE-A2-6, PE-A2-7, and PE-A2-8 at a depth of 2 m, 5 m, and 8 m of Apron 2 area, respectively compare to the analysis results with various conditions. To consider the excess pore water pressure at 2 m depth, the monitored data from PE-A2-6 is nearly the same with the case analysis of smear effect and k equal to 5.1 m/day. The excess pore water pressure of monitoring from PE-A2-7 and PE-A2-8 at 5 m and 8 m depths are nearly the same with the case analysis of smear effect and k equal to 2.0 m/day. Figure 12 showed the consolidation settlement of ground surface at Apron 2 for settlement plate SP7 and the analysis results. The settlement of the monitoring and analysis are nearly the same which analyzing, taking into account in smear effect and well resistance by setting k equal to 25.5 m/day.

Figure 13 showed the monitoring of excess pore water pressure in the field from piezometers PE-RF-6, PE-RF-7, and PE-RF-8 at depths of 2 m, 5 m, and 8 m of Reference

area, respectively compare to the analysis results. The behavior of increasing and decreasing excess pore water pressure, measured in the field and analysis results are consistent according to the construction stages. To consider the excess pore water pressure at 2 m depth, the monitoring in the field from PE-RF-6 is nearly the same with the case analysis of smear effect and k equal to 15.3 m/day.

The excess pore water pressure of monitoring in the field from PE-RF-7 and PE-RF-8 at 5 m and 8 m depths are nearly the same with the case analysis of smear effect and k

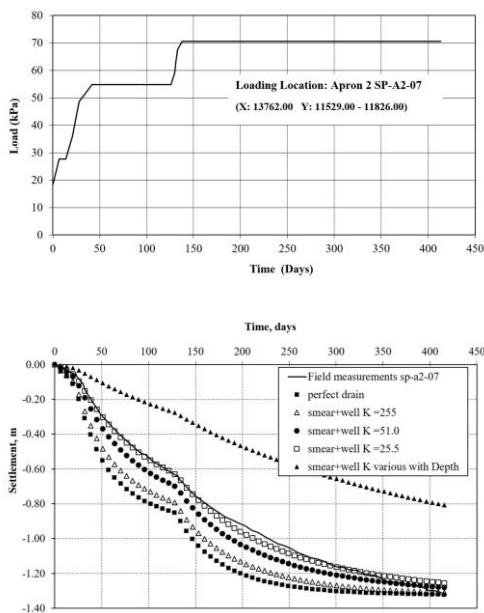


Figure 12. Loading and consolidation settlement at ground surface of Apron 2 form settlement plate SP7 with various time (X: 13762.00, Y: 11529.00 - 11826.00)

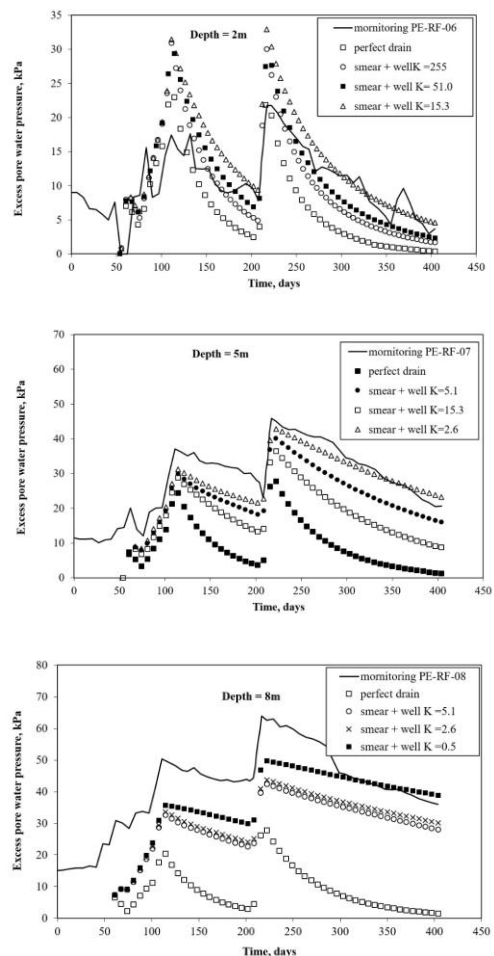


Figure 13. Variation of excess pore water pressures at Reference area for Piezometer at 2, 5, and 8 m below ground level. (X : 13639.00 Y : 11894.00)

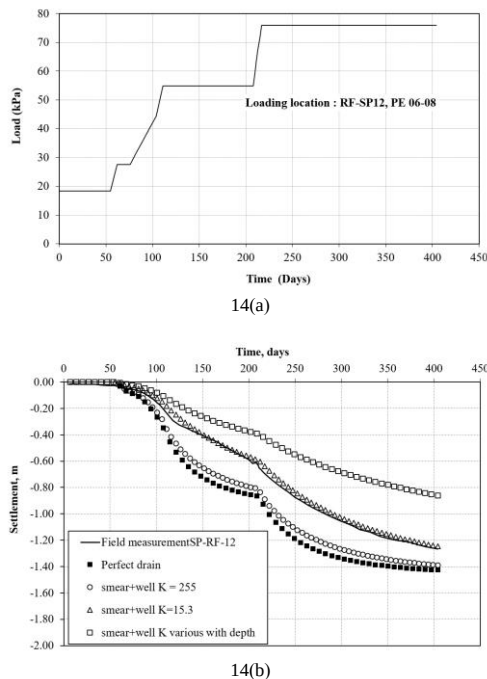


Figure 14. Loading and consolidation settlement at ground surface of Apron 2 form settlement plate SP7 with various time (X: 13762.00, Y: 11529.00 - 11826.00)

equal to 2.6 m/day. Figure 14 showed the consolidation settlement at ground surface of Reference Area (RF) by the settlement plate SP12 and the analysis results. The settlements of the monitoring and analysis are nearly the same when analyzing, taking into account in smear effect and well resistance by setting k equal to 15.3 m/day.

From the finite element analysis, it found that the settlement values from the analysis are close to the measured value in the field when performing analytical thinking, the effects of smear and well resistance with determining the k value of PVD in the range 15.3 to 25.5 m/day. If calculating the k value to discharge capacity (q_w), they will be equal to 284 to 474 m^3 /year.

When considering the excess pore water pressure analysis results, it was found that the k values changed according to the types and depths of the soil layers. In the weather crust

layer at depth 0.2 m, q_w are in the ranges of 284 to 474 m^3 /year same as the results of back analysis from settlement data. In the very soft clay to soft clay layers at depth 2-10 m, the values of q_w are in the ranges of 37 to 46 m^3 /year which result in consistent reported with previous researches. Hansbo (1987) presents the appropriate value of q_w in the range of 50-100 m^3 /year. In addition, Bergado *et al.* (1996) present the value from back analysis of the q_w in the range of 30-100 m^3 /year.

Conclusions

From the analysis results of factors that influence in the behavior of PVDs by finite element, the conclusion can be drawn as following:

1. The settlement of filed monitoring is close to the finite element when analyzed with the effects of smear and well resistance with the discharge capacity of PVD in the rage of 284 to 474 m^3 /year.
2. Form the back analysis of excess pore water pressure, It found that the discharge capacity of PVD in the layers of weather crust and soft clay are in the range of 284 to 474 m^3 /year and 37 to 46 m^3 /year, respectively.
3. The excess pore water pressure in the depth of 2 meters that was analyzed was the closest to the field monitoring value but at deeper layer, the monitored value is lower than the analysis value.

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References

- Bergado, D.T., Asakami H., Alfaro, M.C. and Balasubramaniam, A.S. (1991). Smear effects of vertical drain on Soft Bangkok clay. *Journal of Geotechnical Engineering, ASCE*, 117(10):1,509-1,530.
- Bergado, D.T., Anderson, L.R., Miura, N., and Balasubramaniam, A.S. (1996). *Soft ground*

- improvement in lowland and other environment, ASCE, New York.
- Chai, J.C. and Miura, N. (1999). Investigation of factors affecting vertical drain behavior. *J. Geotech. Geoenviron. Eng.*, ASCE 125(3):216-226.
- Freeze, R.A. and Cherry, J.A. (1979). *Groundwater*. Prentice-Hall, p. 32-34.
- Hansbo, S. (1979). Consolidation of clay by band-shaped prefabricated drains. *Ground Engng.*, 12(5):16-27.
- Hansbo, S. (1981). Consolidation of fine grained soil by prefabricated drains. *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, (3):677-682.
- Hansbo, S. (1987). Design aspects of vertical drains and lime column installation. *Proceedings of the 9th Southeast Asian Geotech. Conf.* Bangkok, Thailand, (2):8-12.
- Hird, C.C., Pyrah, I.C., and Russell, D. (1992) Finite element modelling of vertical drains beneath embankments on soft ground. *Geotechnique*, 42(3):499-511.
- Hird, C.C., Pyrah, I.C., Russell, D., and Cinicioglu, F. (1995). Modelling the effect of vertical drains in two- dimensional finite element analyses of embankments on soft ground. *Canadian Geotech. J.*, (32):795-807.
- Indraratna, B. and Redana, I.W. (2000). Numerical modeling of the vertical drains with smear and well resistance installed in soft clay. *Canadian Geotech. J.*, (37):132-145.
- Jamiolkowski, M., Lancellotta, R., and Wolski, W. (1983). Pre-compression and speeding up consolidation, General Report, Special Session 6. *Proceedings of 8th Europe Conf. Soil Mech. and Found. Eng'g.*, Helsinki, (3):1,201-1,226.
- Lin, D.G., Kim, H.K., and Balasubramaniam, A.S. (2000). Numerical modeling of prefabricated vertical drain. *Geotech. Eng. J.*, 31(2):109-125.
- Mesri, G. and Lo, D.O.K. (1991). Field performance of prefabricated vertical drains. *Proc. Int. Conf. On Geotech. Eng'g. for Coastal Development - Theory to Practice*, Yokohama, (1):231-236.
- Seah T.H., Burut T., and Pana W. (2004). Horizontal coefficient of consolidation of soft bangkok clay. *Geotech. Test. J.*, 27(5).