PVD IMPROVEMENT OF SOFT BANGKOK CLAY WITH COMBINED VACUUM AND REDUCED SAND EMBANKMENT PRELOADING

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ABSTRACT

The proposed site of the Second Bangkok International Airport (SBIA) comprising a total land area of 8.0 km by 4.0 km, is situated on a 16.0 m thick very soft to soft Bangkok clay. At the proposed site, a previous study has been successfully conducted involving the use of prefabricated vertical drain (PVD) with conventional preloading using sand embankment surcharge. Vacuum-assisted consolidation provides an alternative in reducing the length of preloading period. In this method, the soft clay foundation is preloaded by reducing the pore pressures through the application of vacuum pressure in combination with reduced amounts of sand surcharging. Two full scale and fully instrumented test embankments each with base area of 40 m by 40 m were constructed. In Embankment 1, hypernet drainage system combined with 15.0 m PVD length were used. For Embankment 2, perforated and corrugated pipes combined with nonwoven heat-bonded geotextiles were used as drainage system combined with 12.0 m PVD length. Among the foundation instrumentation, vibrating wire piezometers were installed in the foundation subsoil at varying depths to measure both negative and positive pore pressures. The undrained shear strength obtained after improvement was found to be 1.5 to 2.0 times higher than before improvement. Embankment 2 indicated higher drainage efficiency demonstrating 20 to 30 percent accelerated settlement rate compared to Embankment 1. After 45 days of vacuum pressure application, the test embankments were raised to a maximum height of 2.50 m. The surface settlements in Embankments 1 and 2 were 0.74 m and 0.96 m, respectively, after 140 days. Finite element methods (FEM) was utilized to investigate the influence factors. First, the vacuum preloading was simulated numerically by obtaining reasonable fit in the settlement values. Then, the effects of vacuum preloading was investigated by (a) simulating the field conditions, (b) maintaining higher vacuum pressures, and (c) no vacuum loading. The results of FEM analysis demonstrated the efficiency of combined vacuum preloading and reduced sand surcharging. Finally, the performance of Embankment 2 as compared to the previous studies using conventional surcharging, demonstrated a 60 percent acceleration in the rate of settlement and 4 months reduction in the preloading time.

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INTRODUCTION

At the site of the proposed Second Bangkok International Airport (SBIA), the presence of 16.0 m thick, weak, and compressible soft Bangkok clay pose a lot of foundation problems (AIT, 1995). For economic utilization of the proposed site, ground improvement techniques are necessary. In this regard, ground improvement with prefabricated vertical drain (PVD) has been studied successfully using conventional sand surcharge (Bergado et al, 1997). Since sand materials are increasingly expensive being sourced from far distances, vacuum-assisted preloading can be viable alternative. In this method, instead of increasing the effective stresses in the soil mass by increasing the total stresses, vacuum preloading relies on increasing the effective stresses by decreasing the pore pressures. Thus, vacuum preloading combined with reduced sand surcharging can shorten the consolidation period considerably without endangering the stability of the test embankment.

In this study, two full scale embankments, namely: Embankment 1 and Embankment 2, were constructed at the SBIA site with PVD lengths of 15.0 m combined with hypernet drainage system and 12.0 m with corrugated pipe drainage system, respectively. The PVDs consisted of Mebra Drains and were installed at 1.0 m spacing in triangular pattern. The test embankments with base dimensions of 40 m by 40 m were constructed in stages up to a height of 2.50 m in order to provide surcharge and combined with a vacuum pressure of -60 kPa continuously for a period of 5 months.

PRINCIPLES OF VACUUM CONSOLIDATION TECHNIQUE

Vacuum consolidation was proposed in early 1950s by Kjellman (1952). Isolated studies of vacuum induced consolidation continued for the next two decades (Holtz, 1975). Vacuum-assisted consolidation provides an effective alternative to surcharging for preloading soils. Instead of increasing the effective stress in the soil mass by increasing the total stress by means of conventional mechanical surcharging, vacuum assisted consolidation preloads the soil by reducing the pore pressure while maintaining constant total stress. Figure 1 presents a typical layout of a vacuum-assisted consolidation with PVD. Figure 2 graphically portrays the initial total stress in the ground and pore pressure induced due to a) conventional surcharge and b) vacuum loading applied at the ground surface assuming 100% efficiency (vacuum pressure of -100 kPa). Vacuum-assisted consolidation with PVD was tested in China and was presented by Choa (1989) with approximately 70 to 80 percent efficiency. Jacob et al (1994) reported an average vacuum efficiency of 40 to 50 percent compared to a target value of 70 percent for a test section with PVD on a hydraulic landfill area.

SITE DESCRIPTION AND SOIL PROFILE

The proposed site of the Second Bangkok International Airport (SBIA) is located at Nong Ngu Hao in the Central Plain of Thailand. Figure 3 shows the project area of about 8 km by 4 km situated about 25 km east of Bangkok Metropolis.

The soil profile at the site can be divided into 5 sublayers as shown in Fig. 4. It consists of a 1.0 m thick weathered clay layer overlying very soft to soft dark gray layer which extends from 1.0 m to 10.5 m depth. Underneath the soft clay layer, a 2.50 m thick medium clay layer can be found. The light-brown stiff clay layer can be encountered at 14.0 m to 21.0 m depth. The undrained shear strength of the very soft to soft clay layer increased from 13 to 27 kPa with depth. The groundwater level was found at about 0.50 m depth. The initial piezometric level is lower than the theoretical hydrostatic pressure below 6.0 m depth due to the excessive withdrawal of groundwater causing ground subsidence (see Fig. 10).

FULL SCALE TEST EMBANKMENT

Two full scale test embankments each with base area of 40 m by 40 m with different drainage systems were constructed on soft Bangkok clay with PVD. In Embankment 1 (TV 1), hypernet drainage system with 15 m PVD length were used. For Embankment 2 (TV 2), perforated and corrugated pipes combined with nonwoven geotextiles with 12 m length PVD were utilized. The plan of the test area is given in Fig. 5. Also shown in Fig. 5 are the locations of boreholes and field vane tests as well as the dummy area. The working platforms which also served as drainage blankets were constructed with thickness of 0.30 m for Embankment 1 and 0.80 m for Embankment 2.

The PVDs were installed from the working platforms to a depth of 15 m for TV1 and 12 m for TV2. As shown in Fig. 6, the PVDs were installed in triangular pattern with 1.0 m spacing. The parameters related to the behavior of PVD are listed in Table 1.

The cross-sections of TV1 and TV2 are shown in Figs. 7 and 8, respectively. The drainage layers of the test embankments are also shown in Figs. 7 and 8 consisting of, respectively, hypernet for TV1 and perforated and corrugated pipes covered with geotextiles for TV2. The geotextile consisted of 136 g/m^2 nonwoven spunbonded polypropylene with high modulus. The hypernet (or geonet) consists of a grid of HDPE threads which are melted together at their intersections. The hypernet has discharge capacity of 8×10^{-3} m^3/s per meter width. One layer of hypernet was placed over the whole area. The perforated and corrugated pipes consists of 5 pieces of Mebra tubes with 80 mm diameter and 297 g/m weight. On top of the drainage system, a water and air tight LLDPE geomembrane liner was placed. The geomembrane liner was sealed by placing the edges at the bottom of the perimeter trench and covered with 300 mm layer of sand-bentonite and submerged underwater.

COMBINED VACUUM PRESSURE AND SURCHARGE PRELOADING

In each embankment, the water collection system was connected to vacuum pump capable of supplying -70 kPa vacuum pressure continuously. A back-up pump was also provided. After applying the vacuum pressure for 45 days, the embankments were raised in stages up to a height of 2.50 m. Embankment 1 and, similarly, Embankment 2 were raised from 0.30 m and 0.80 m height, respectively. The stage loading diagrams with time are shown in Fig. 11 for both Embankments 1 and 2.

FIELD INSTRUMENTATIONS

The field instrumentations for monitoring of embankment behavior include surface settlement plates, subsurface multipoint extensometers, vibrating wire electrical piezometers, and inclinometers. In the dummy area, the instrumentations include standpipe piezometers, surface settlement plates (or benchmarks), and observation wells. Figures 5, 7 and 8 show the typical layout of instrumentations for the test embankments. The vibrating wire piezometers were installed under the test embankments at 3.0 m depth intervals together with the sensors for the multipoint extensometers. The surface settlement plates were placed directly on top of the geomembrane liner. The inclinometers were placed at the edges of each test embankment. At the dummy area, observation wells, standpipe piezometers and a benchmark were also installed.

FEM ANALYSES OF VACUUM CONSOLIDATION

Considering that most finite element codes used in practice do not include special drainage element, a simple approximate method for modeling the effect of PVD has been proposed by Chai and Miura (1997). From the macro point of view, PVD increases the mass permeability in the vertical direction. Consequently, it is possible to establish a value of the vertical permeability which approximately represents the combined vertical permeability of the natural subsoil and the radial permeability towards the PVD. This equivalent vertical permeability *(K_{ve})* is derived based on equal average degree of consolidation together with the following assumptions:

1. The deformation mode of PVD improved subsoil is close to one-dimensional. Thus, one-dimensional consolidation theory can be used to represent the consolidation in the vertical direction and the unit cell theory of Hansbo (1979) for radial consolidation is applicable.

2. The total degree of consolidation is the combination of vertical and radial consolidation by using the relationship proposed by Scott (1963).

In order to obtain a one-dimensional expression for the equivalent vertical permeability, an approximate equation for consolidation in vertical direction is proposed as follows:

$$U_v = 1 - \exp(-3.54)\ T_v \quad (1)$$

where U_v is the vertical degree of consolidation and T_v is the dimensionless time factor. The equivalent vertical permeability, K_{ve}, can be expressed as:

$$K_{ve} = \left(1 + \frac{2.261^2}{F\,D_e^{\,2}}\,\frac{K_h}{K_v}\right) K_v \quad (2)$$

where:

$$F = \ln\left(\frac{D_e}{d_w}\right) + \left(\frac{K_h}{K_s} - 1\right) \ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \frac{\pi\, 2\, L^2\, K_h}{3\, q_w} \quad (3)$$

where D_e is the equivalent diameter of a unit PVD influence zone, d_s is the equivalent diameter of the disturbed zone, d_w is the equivalent diameter of PVD, K_h and K_s are the undisturbed and disturbed horizontal permeability of the surrounding soil, respectively*,* L is the PVD length for one-way drainage, and q_w is the discharge capacity of PVD. The effects of smear and well-resistance have been incorporated in the derivation of the equivalent vertical permeability.

For numerical modeling, the ground was divided into 5 sublayers and represented by modified Cam clay model (Roscoe and Burland, 1968). The adopted model parameters are listed in Table 2. Part of the values in Table 2 were evaluated based on laboratory consolidation test results and part of them were determined empirically. The values of permeability were determined by referring to the back-calculated data of previous test embankments in the adjacent area (Chai et al, 1996). The estimated initial stresses water pressure and the size of yield locus are given in Table 3. The factor of hydraulic pressure drawdown due to excessive pumping of groundwater was considered for evaluating the initial stresses.

With the soil parameters in Table 2, and the compression modulus corresponding to the yielding stress (size of yielding locus in Table 3), for a point 5.0 m below ground surface (middle of very soft to soft clay layer), a coefficient of consolidation of about 7 m^2/yr can be obtained. This value is comparable with the field value obtained from dissipation tests using the piezocone apparatus in Fig. 13 (Hanh et al, 1998). This indicates that the piezocone dissipation test is useful for determining the field coefficient of consolidation.

SIMULATION OF VACUUM CONSOLIDATION

The analyses were conducted under plane strain condition. The vacuum consolidation was simulated by fixing the excess pore pressure at ground surface of the test area. There are discrepancies between the measured vacuum pressure in the sand mat and at the ground surface. The adopted values are based on the measured values at ground surface with adjustment on the vacuum pressure at the early stage (<20 days). This is because the measurement yielded low vacuum pressures at the early stages but there were considerable settlements.

In this study, first the vacuum consolidation were simulated numerically. After obtaining a reasonable fitting of settlement magnitudes, the distribution as well as the variation of vacuum pressure in the ground was studied. Then, the effect of vacuum was studied using higher vacuum (-60 kPa) and no vacuum. Figures 9a,b show the adopted vacuum pressure-time curves for both test embankments. The higher vacuum cases are also shown in the figures. The measured total and excess pore pressures in the subsoils at Embankment 2 are shown in Fig. 10. The loading histories of the test embankments (Fig. 11) according to the field record were also used in the simulations.

CONSOLIDATION SETTLEMENTS

Figure 11 illustrates the construction stages of both test embankments together with the settlement-time curves at varying depths. In Embankment 1, the maximum settlement after 144 days at the ground surface, 3 m, 6 m, and 9 m depths were 0.74 m, 0.48 m, 0.26 m and 0.09 m, respectively. The corresponding values in Embankment 2 were 0.97 m, 0.70 m, 0.35 m and 0.11 m, respectively.

Using FEM analysis, the calculated settlements are compared with the observed values in Figs. 12a, b. Although there are slight discrepancies, it is considered that FEM analysis simulated the measured data reasonably well.

BENEFICIAL EFFECTS OF VACUUM CONSOLIDATION

To quantify the beneficial effects of vacuum consolidation, Figs. 14a,b compare the surface settlements at the center of the test embankment obtained from the FEM analyses with the corresponding measured data assuming no vacuum pressure, with vacuum pressure that simulates the field conditions, and at higher vacuum pressure. For Embankment 1 in Fig. 14a, at about 140 days and no vacuum case, a settlement of 0.43 m can be obtained. With vacuum pressure as indicated by solid line in Fig. 9a, the settlement was 0.73 m. If the high vacuum pressure of -60 kPa (dashed line in Fig. 9a) was maintained, the settlement would be 1.30 m. The beneficial effects of vacuum consolidation have been demonstrated.

Figure 14b compares the numerical results of no vacuum, with vacuum, and higher vacuum cases with the measured settlement data for Embankment 2. The trends are similar to Embankment 1. For no vacuum, the slight difference in loading history of Embankments 1 and 2 does not have significant effects on foundation settlements. The vacuum consolidation is effective if the vacuum preloading can be maintained for longer periods and if leaks are prevented. The results have indicated that even with PVD installation, high vacuum pressure need to be maintained for 4 to 5 months to achieve higher degree of consolidation.

LATERAL DEFORMATIONS

The lateral displacements obtained from FEM analyses are compared to the measured values after 45 days of vacuum application in Figs. 15a,b for both embankments. For both embankments, especially Embankment 1, the simulated data are comparable to the measured data below 2.0 m depth. However, for both embankments, the measured data do not agree with the simulations near the ground surface. An explanation for these discrepancies is the possible disturbances of the inclinometer casings near the ground surface. When applying vacuum pressure at the ground surface, the effective stress increment in the top soil layer will be approximately the same for both horizontal and vertical directions. Therefore, based on the principles of soil mechanics, the vacuum pressure application would induce an inward lateral deformation, if the effect of the embankment fill surcharge is neglected.

EXCESS PORE PRESSURES

Figure 16a,b shows the contours of simulated excess pore pressures for both embankment foundations at the end of high vacuum application (45 days after pumping started). For Embankment 1, the vacuum pressure affected down to 15.0 m depth (depth of PVD installation). However, the vacuum pressure at 15.0 m depth is only -15 kPa which is quite small compared to -60 kPa vacuum pressure at the ground surface. For Embankment 2, the vacuum pressure at 12.0 m depth (depth of PVD installation) was -30 kPa. This higher value is considered to be the main reason for larger settlements in Embankment 2.

The simulated and measured excess pore pressures are compared in Figs. 17a,b for both embankments. At 45 days for both embankments, the lower measured excess pore pressures in the lower depths maybe explained as follows: (a) the piezometers might be close to the PVD, (b) the initial total pore pressures might be higher due to the recharge effects of PVD (see Fig. 10), and (c) the effects of ground subsidence due to excessive withdrawal of groundwater which greatly reduced the excess pore pressures (piezometric drawdown) at lower depths (refer to Fig. 10). For Embankment 2 at 140 days in Fig. 17b, the predicted excess pore pressure is lower than the measured data. This is because to fit the measured settlements in the simulation, the surface vacuum pressure was maintained at -20 kPa.

COMPARISON WITH PREVIOUS STUDIES ON PVD

The settlement of Embankment 2 with PVD spaced at 1.0 m in triangular pattern and 12.0 m long with vacuum preloading was compared with the results of previous studies with conventional sand surcharge. Embankment TS3 in the previous test embankment had PVD spaced at 1.0 m in triangular pattern and 12.0 m long preloaded with conventional sand embankment. Figure 18 shows the loading and settlement records of both embankments. Embankment TS3 indicated a total settlement of 1.60 m after 400 days under a maximum sandfill surcharge of 4.2 m high. However, under a sandfill height of 2.5 m surcharge, greater settlements were indicated for Embankment 2 with vacuum preloading than Embankment TS3. Moreover, an acceleration in the rate of settlement of 60% was recorded. Furthermore, under vacuum pressure, the preloading period to obtain the same amount of settlement is also reduced by 4 months.

If leakages did not occur, the combined 2.50 m high sandfill and vacuum preloading for Embankment 2 is supposed to have the same level of surcharge as that using conventional surcharging in Embankment TS3 with 4.2 m high sandfill. However, the final settlement of 0.97 m for Embankment 2 is lower than the settlement of 1.60 m for Embankment TS3. For the high vacuum case, the numerical results yielded a settlement of 1.30 m at 145 days which is much larger than the corresponding value of TS3. In addition, under vacuum pressure, the lateral deformations near the ground surface can be less than the corresponding values when subjected to sand fill loading. The lower lateral movements can reduce the settlements.

CONCLUSIONS

At the proposed site of the Second Bangkok International Airport (SBIA), two full scale and fully instrumented test embankments, each with 40 m by 40 m base area, was constructed on 16 m thick soft Bangkok clay improved with prefabricated vertical drain (PVD). In this site, ground improvement with PVD subjected to conventional sand surcharging has already been studied successfully. Vacuum assisted consolidation with reduced sand surcharging provides cheaper and faster alternative. In this study, vacuum preloading in combination with reduced amount of sand surcharging were applied. The performances of the 2 test embankments with different drainage systems are described and analyzed. In Embankment 1, hypernet (geonet) drainage system combined with 15.0 m PVD length were used as drainage system with 0.3 m thick sand blanket. In Embankment 2, perforated corrugated pipes combined with nonwoven geotextiles were used as drainage system in combination with 0.8 m thick sand blanket at the top of 12 m long PVDs with spacing of 1.0 m in triangular pattern. After 45 days of vacuum loading, the sand surcharge was raised to 2.5 m high. Finally, finite element method (FEM) was utilized to investigate the efficiency of the field study. Based on the measurements and subsequent analyses, the following conclusions can be made:

1) The final settlement of Embankment 1 and Embankment 2 amounted to 0.74 m and 0.96 m. Although some leakages occurred, the effectiveness of vacuum-assisted consolidation has been demonstrated.

2) The finite element method (FEM) illustrated the beneficial effects of vacuum preloading combined with reduced sand surcharging by comparison of the simulated results using the (a) actual field loading conditions, (b) by maintaining higher vacuum loading, and (c) by no vacuum loading. The numerical results indicated that even with PVD, the vacuum pressure needs to be maintained for more than 4 to 5 months in order to achieve higher degree of consolidation.

3) The performance of Embankment 2 with vacuum preloading, when compared to previous studies using conventional sand surcharging showed an acceleration in the rate of settlement by about 60% and a reduction in the period of preloading by about 4 months.

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Table 1 Parameters of Vertical Drain

Spacing, S	1.0 m (triangular pattern)
Diameter of drain, d_w	50 mm
Diameter of smear zone, d_s	300 mm
Ratio of K_h/K_s	10
Drainage length, 1	12 m
Discharge capacity, q_w (per drain)	50 m^3/year

Table 2 Soil Parameters

Depth m	e_o	λ	κ	ν	M	K_h 10^{-4} m/day	K_v 10^{-4} m/day	γ kN/m^3
0-1.0	1.8	0.3	0.03	0.3	1.2	26	26	16.0
1.0-8.5	2.8	0.73	0.08	0.3	1.0	11	5.5	14.5
8.5-10.5	2.4	0.5	0.05	0.25	1.2	5.2	2.6	15.0
10.5-13.0	1.8	0.3	0.03	0.25	1.4	2.2	1.1	16.0
13.0-18.0	1.2	0.1	0.01	0.25	1.4	0.52	0.26	18.0

Table 3 Initial Stresses

Depth m	σ_{ho} kPa	σ_{vo} kPa	μ_o kPa	Size of Yield, Locus (ρ_o') kPa
0.0	5.0	5.0	-5.0	57.5
0.5	8.0	8.0	0.0	52.7
1.0	11.7	11.0	5.0	42.0
2.0	13.2	15.5	15.0	40.0
3.0	15.6	20.75	25.0	39.7
8.5	35.3	54.75	70.0	75.0
10.5	39.9	79.75	75.0	80.0
13.0	49.3	114.75	80.0	105.5
18.0	88.0	204.75	80.0	188.3

Table 4 Soil Parameters Used in Settlement and Stability Analysis

Zone	Depth (m)	Z_1 (m)	Γ (kN/m^3)	σ_{vo} (kPa)	σ_p (kPa)	OCR	CR	RR	C_h	S_u (kPa)
1	0.3-2.0	0.85	16.0	12.1	75	6.20	0.30	0.030	10	12.5
2	2.0-5.0	4.2	14.5	28.5	50	1.75	0.55	0.055	3	10.0
	5.0-7.0									10.5
3	7.0-9.0	8.7	14.5	48.7	65	1.35	0.045	0.045	4	14.0
	9.0-11.0									17.5
4	11.0-13.0	11.7	16.0	64.7	87	1.35	0.035	0.035	4	23.0
5	13.0-15.0	13.7	16.5	77.2	105	1.35	0.030	0.030	4	30.0

Table 2 Soil Parameters Used in the F.E.M. Analyses

Depth (m)	λ	κ	M	ν	K_v 10^{-4} (m/day)	K_h 10^{-4} (m/day)	e_{cs}
0-2	0.34	0.07	1.2	0.25	25.9	25.9	2.80
2-7	0.90	0.18	0.9	0.30	5.9	10.1	5.90
7-12	0.50	0.10	1.0	0.25	2.6	5.2	4.00
12-15	0.34	0.07	1.2	0.25	1.0	2.1	3.00
15-22	0.10	0.02	1.2	0.20	0.3	0.5	1.30