SYNOPSIS: The proposed site of the Second Bangkok International Airport (SBIA), consisting of 8.0km by 4.0 km, is founded on a 16.0m thick very soft to soft Bangkok clay. In the proposed site, ground improvement using prefabricated vertical drain (PVD) has previously been studied successfully with full scale sand embankment surcharge. Vacuum-assisted consolidation provides an alternative to surcharging for preloading with PVD in the clay subsoils. In this study, the soil is preloaded by reducing the pore pressures through the application of vacuum pressures and combined with reduced sand surcharging. Two full scale and fully instrumented test embankments each with base area of 40m by 40m and with different drainage systems were constructed. For Embankment 2, perforated corrugated pipes combined with nonwoven geotextiles were used as drainage system in conjunction with the 0.8m thick sand blanket at the top of 12.0m long PVDs. The maximum total surface settlement for Embankment 2 amounted to 0.96m after about 120 days. The undrained shear strength obtained after improvement was 1.5 to 2.0 times higher than before improvement. The back-calculated Ck values agreed with the measured values from piezocone test. Finally, finite element method (FEM) was utilized to investigate the influence factors. First, the vacuum preloading was simulated numerically by obtaining reasonable fitting 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 with reduced sand surcharging.

INTRODUCTION

The major problems at the proposed location of the Second Bangkok International Airport (SBIA) are the low strength and high compressibility of the 16.0m thick very soft to soft Bangkok clay which underlie the project site. The area is mostly covered by paddy fields and fishponds. In the proposed site, ground improvement with prefabricated vertical drain (PVD) has been studied successfully with sand embankment surcharge (Bergado et al., 1997). Drainage systems using PVD with surcharge and vacuum-assisted preloading can be effective alternative to conventional sand surcharging since sand materials are expensive being sourced at far distances. Instead of increasing the effective stresses in the soil mass by increasing the total stresses as in conventional sand surcharging, vacuum-assisted preloading relies on increasing the effective stresses by decreasing the pore pressures. Vacuum preloading with surcharge combined with PVD can shorten the preloading period considerably without endangering the stability of the embankment.

Vacuum consolidation was proposed in the early 1950s by Kjellman (1952). Isolated studies of vacuum-assisted consolidation continued in next two decades (Holtz, 1975). Vacuum-assisted consolidation with vertical drains was tested in China and was presented by Choa (1989) with 70 to 80 percent efficiency.

Two full scale test embankments, namely: Embankment 1 and Embankment 2, were constructed at SBIA site with PVD lengths of 15.0m and 12.0m, respectively. The PVDs consists of Mebra drains and were installed at 1.0m spacing in triangular pattern. The test embankments with base dimensions of 40m by 40m were constructed in stages up to a height of 2.5m in order to provide surcharge combined with a vacuum pressure of -60 KPa for a period of about 3 months. The details and performance of Embankment 2 is described in this paper.

SITE DESCRIPTION AND SOIL PROFILE

The proposed site of the Second Bangkok International Airport (SBIA) is located at Nong Ngou Hao, about 25 km east of Bangkok metropolis. The project area consists of about 8 by 4 km and is situated in the Chao Phraya (Central) Plain of Thailand.

The soil profile at the site can be divided into sublayers (Fig. 1). It consists of a 2.0m thick weathered clay layer overlying very soft to soft, dark gray, clay layer with shells and organic matter which extends from 2.0 to 12.0m depth. Below the soft clay layer, a 3.50m thick medium clay layer can be found. The light-brown stiff clay layer is encountered at 15.5 to 21.0m depth. The undrained shear strength from field vane shear tests increased from 13 to 27 KPa with depth. The groundwater
level was found at 0.60m depth. Note that the initial piezometric level is lower than the theoretical hydrostatic pressure below 6.0m depth due to the excessive withdrawal of groundwater causing ground subsidence.

FULL SCALE TEST EMBANKMENT

The working platform which also served as the drainage blanket for Embankment 2 was constructed to 0.80m thick with the base area of 40m by 40m. The PVD installation to 12.0m depth was done from the working platform. The PVD was installed in triangular pattern with 1.0m spacing. The drainage layer at the sand blanket for Embankment 2 consisted of a geotextile combined with a system of perforated corrugated pipe 80mm in diameter. The geotextile was placed directly on top of the vertical drains which were cut off 0.15m below the working platform surface. The geotextile used consisted of 136 g/m² nonwoven spunbonded polypropylene with high modulus. On top of the drainage layer, a water and air tight VLDPE geomembrane liner was laid out. The ends of the liner were placed on the bottom of the trench and covered with 0.30m thick sand-bentonite layer. The water level in the trench was maintained at constant level. The cross-section of Test Embankment 2 is given in Fig. 2. The drainage system of Embankment 2 was connected to one vacuum pump. The vacuum pump was capable of generating -100 kPa pressure. After testing the liner for airtightness and applying the vacuum for about 45 days, a surcharge fill with density of 1.8 t/m³ was placed on top of the liner up to a total height of 2.50m.

The monitoring instrumentation includes piezometers, surface settlement plates, multipoint extensometers, inclinometers, observation wells and benchmarks. The locations of these instrumentations are also shown in Fig. 2. Vibrating wire piezometers were placed under the embankment at 3.0m intervals. Standpipe piezometers were placed on the dummy area at 1.50m intervals to measure the initial pore pressures with depth. Surface settlement plates were placed on top of the geomembrane liner. Multipoint extensometers were installed at 3.0m intervals to monitor the subsurface settlements below the embankment. An inclinometer was installed at the edge of the embankment but not at the same side as the vacuum pump. An observation well was installed at the dummy area to monitor the change in the groundwater level. A benchmark for each embankment was located at a distance of 10.0m from the perimeter trench. Another benchmark was installed at the dummy area.

FEM ANALYSIS 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 (kₚₑ) 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 (1981) 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 simple expression for the equivalent vertical permeability, an approximate equation for the average degree of consolidation is proposed as follows:

\[ U_s = 1 - \exp(-3.54)T_s \]  

(1)

where \( U_s \) is the vertical degree of consolidation and \( T_s \) is the dimensionless time factor.

The equivalent vertical permeability, \( K_{ve} \), can be expressed as:

\[ K_{ve} = \left( 1 + \frac{2.261^2 K_h}{FD_s} \right) K_v \]  

(2)

where:

\[ F = \ln \left( \frac{D_s}{d_w} \right) + \left( \frac{K_h}{K_v} \right) - \left( \frac{3}{4} \right) + \frac{\pi 2l^2 K_h}{3q_w} \]  

(3)

where \( D_s \) is the equivalent diameter of a unit PVD influence zone, \( d_w \) is the equivalent diameter of PVD, \( K_h \) and \( K_v \) 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. Consequently, the effects of smear and well-resistance have been incorporated in the analysis.

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 1. Part of the values in Table 1 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 a test embankment at neighboring area (Chai et al., 1996). The deposit is in a lightly-overconsolidated state. The estimated initial stresses, water pressure, and the size of yield locus are given in Table 2. The factor of hydraulic pressure drawdown due to excessive pumping of groundwater was considered for evaluating the initial stresses. The smear effect and discharge capacity parameters were evaluated empirically (Chai et al., 1997).

The analyses were conducted under plane strain condition. The vacuum consolidation was simulated by fixing the excess

<table>
<thead>
<tr>
<th>Table 1 Soil Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth m</td>
</tr>
<tr>
<td>----------</td>
</tr>
<tr>
<td>0-1.0</td>
</tr>
<tr>
<td>1.0-8.5</td>
</tr>
<tr>
<td>8.5-10.5</td>
</tr>
<tr>
<td>10.5-13.0</td>
</tr>
<tr>
<td>13.0-18.0</td>
</tr>
</tbody>
</table>
Table 2 Initial Stresses

<table>
<thead>
<tr>
<th>Depth m</th>
<th>$\sigma_{\text{w}}$ kPa</th>
<th>$\sigma_{\text{uw}}$ kPa</th>
<th>$\mu_{\text{w}}$ kPa</th>
<th>Size of Yield Locus ($\sigma_{\text{loc}}$) kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>5.0</td>
<td>5.0</td>
<td>-5.0</td>
<td>57.5</td>
</tr>
<tr>
<td>0.5</td>
<td>8.0</td>
<td>8.0</td>
<td>0.0</td>
<td>52.7</td>
</tr>
<tr>
<td>1.0</td>
<td>11.7</td>
<td>11.0</td>
<td>5.0</td>
<td>42.0</td>
</tr>
<tr>
<td>2.0</td>
<td>13.2</td>
<td>15.5</td>
<td>15.0</td>
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<td>3.0</td>
<td>15.6</td>
<td>20.75</td>
<td>25.0</td>
<td>39.7</td>
</tr>
<tr>
<td>8.5</td>
<td>35.3</td>
<td>54.75</td>
<td>70.0</td>
<td>75.0</td>
</tr>
<tr>
<td>10.5</td>
<td>39.9</td>
<td>79.75</td>
<td>75.0</td>
<td>80.0</td>
</tr>
<tr>
<td>13.0</td>
<td>49.3</td>
<td>114.75</td>
<td>80.0</td>
<td>105.5</td>
</tr>
<tr>
<td>18.0</td>
<td>88.0</td>
<td>204.75</td>
<td>80.0</td>
<td>188.3</td>
</tr>
</tbody>
</table>

Pore pressure at ground surface of test area. There are discrepancies between the measured vacuum pressure in the sand 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 gave a low vacuum pressure at the early stage but there was considerable settlement. Figure 3 shows the adopted vacuum pressure - time curves for Embankment 2. The higher vacuum cases are also indicated in the figure. The loading history of Embankment 2 was simulated according to the field record.

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 (a) higher vacuum (60 KPa) and (b) no vacuum.

GEOTECHNICAL PARAMETERS

The soil properties before and after improvement is also plotted with depth in Fig. 1. The total unit weights are higher than the previous values before improvement. The natural water content, liquid limit, and plasticity index decreased after improvement. The maximum past pressures also increased after improvement. The undrained shear strength obtained from field vane shear tests improved to substantially higher values. The shear strengths indicated considerable increase after improvement by a factor of 1.5 to 2.0 times in the very soft clay layer at 2 to 8m depth (Hanh et al., 1998).

CONSOLIDATION SETTLEMENTS

Figure 4 illustrates the construction stages of Embankment 2 together with the settlement-time curves at varying depths in

the subsoil. It is indicated that after 144 days, the maximum settlement at the ground surface, 3m, 6m and 9m depths were 0.97m, 0.70m, 0.35m, and 0.11m, respectively. The calculated settlements, using FEM analysis, are compared with the corresponding measured data in Fig. 5. Although there are some slight discrepancies, it is considered that the FEM analysis simulated the measured data reasonably well.

Asaka (1978) proposed a graphical method to determine the final settlements based on the observed data. The observed time-settlement curve plotted in arithmetic scale was divided into equal time intervals, $\Delta t$. The settlement, $S_i$, corresponding to time, $t_i$, is plotted with $S_{i+1}$ corresponding to time, $t_{i+1}$, with $\Delta t$ taken as 10 days. A straight line is fitted through the points with slope $\beta$. The final settlement for Embankment 2 was calculated as 1.01m which is close to the observed value of 0.97m.

HORIZONTAL COEFFICIENT OF CONSOLIDATION, $C_h$

Using the method of Asaka (1978) for radial consolidation with PVD, the horizontal coefficient of consolidation, $C_h$, can be estimated as follows:

$$C_h = \frac{(1 - \beta) D_h^2 F}{8(\Delta t) \beta}$$  \hspace{1cm} (4)

where the terms have been defined previously. Assuming the compression modulus in the vertical direction, $m_v$, is equal to the corresponding value in the horizontal direction, the following expression can be derived:

$$K_h = m_v C_h \gamma_w$$  \hspace{1cm} (5)

Thus, the value of $C_h$ can be obtained assuming the values of $K_h/K_v$, $d/d_w$, and $\gamma_w$. Bergado et al (1996) has found that the $C_h$ value is little affected by $\gamma_w$ when $\gamma_w$ is greater than 50m$^2$/yr. In this analysis, $\gamma_w$ is assumed to be 50m$^2$/yr. As suggested by Hansbo (1987) and reconfirmed for soft Bangkok clay by Bergado et al (1991), the ratio of $d/d_w$ can be assumed to be 2.0. A number of trials can be made on the basis of $K_h/K_v$ of 5 and 10. Figure 6 shows the comparison between the back-calculated using the method of Asaka (1978) and the measured $C_h$ values from piezoecone tests.

LATERAL DEFORMATION

The calculated and measured lateral deformations are compared in Fig. 7 at the end of Embankment 2 construction. The measured data indicated that down to 6m depth, the lateral displacements are close to uniform. The calculated data were obtained from FEM analysis. There is discrepancy between the calculated and measured data. Also shown in Fig. 7 is the possible lateral deformation assuming high vacuum pressure.

When vacuum is applied at the ground surface, the effective stress increment in the topmost soil layer will be approximately the same for both horizontal and vertical directions. Based on
Fig. 3 Vacuum Pressure Loading at the Ground Surface

Fig. 4 Time and Settlement Plot at Different Depths

Fig. 5 Calculated (FEM) and Measured Settlements

Fig. 6 Comparison of Computed and Measured $C_h$ Values

Fig. 7 Calculated (FEM) and Measured Lateral Deformation

Fig. 8 Comparison of FEM Simulation and Measured Settlements
the general principle of soil mechanics, if the effects of the fill material is not considered, the vacuum pressure will certainly induce inward lateral deformation. The presence of 2.5m high embankment fill and possible disturbance in the inclinometer are possible explanation of the discrepancy.

EFFECTS OF VACUUM CONSOLIDATION

Figure 8 compares numerical results of no vacuum pressure with vacuum pressure that simulate the field condition, and higher vacuum pressure. Results show that at the time of 144 days, the surface settlements at the center of Embankment 2 for no vacuum pressure, with vacuum pressure that simulate field condition, and with higher vacuum maintained at -60kPa, are 0.50m, 0.95m and 1.3m, respectively. The beneficial effect of vacuum pressure application have been shown. It also demonstrated the beneficial effects of maintaining higher vacuum pressure at longer periods for possible 4 to 5 months.

CONCLUSIONS

At the proposed site of the Second Bangkok International Airport (SBIA), two full scale and fully instrumented test embankments, each with 40m by 40m base area, was constructed on 16m thick soft Bangkok clay improved with prefabricated vertical drain (PVD). Vacuum preloading in combination with reduced amount sand surcharging were applied. 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. The performance of Embankment 2 is described and analyzed. In this embankment, perforated corrugated pipes combined with nonwoven geotextiles were used as drainage system in combination with 0.8m thick sand blanket at the top of 12m long PVDs with spacing of 1.0m in triangular pattern. After 45 days of vacuum loading, the sand surcharge was raised to 2.5m 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 2 amounted to 0.96m in 120 days and the undrained shear strength improved to 1.5 to 2.0 times.

2) The back-calculated coefficient of horizontal consolidation, $C_h$, agreed with the measured values from piezocene tests.

3) The finite element method (FEM) illustrated the efficiency of the combined vacuum preloading with reduced sand surcharging by comparison of the simulated results using the actual field loading conditions, by maintaining higher vacuum loading, and by no vacuum loading.

REFERENCES


