Investigation on Some Factors Affecting Prefabricated Vertical Drain Behavior

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1. INTRODUCTION

The theoretical solution for vertical drain consolidation was first proposed for unit cell condition (i.e., a single drain surrounded by a soil cylinder) by Barron (1948). However, in many cases, ground deformation patterns do not represent a unit cell condition. Some techniques have been developed by Chai et al. (1995) for simulating the vertical drain effect in two dimensional finite element analysis. Field engineers, however, often face a problem that the expected (or theoretical) effect of vertical drain could not be achieved in the field, especially when comparing the behavior of improved subsoil with unimproved case at the same site. In order to improve the design method, it is useful to investigate the main influencing factors on vertical drain behavior and compare the laboratory data with back-calculated field performance values.

The Asian Institute of Technology (AIT) was engaged by the Airport Authority of Thailand (AAT) to undertake the full scale test of PVD for the SBIA project. Three full scale test embankments were constructed on PVD improved soft Bangkok clay at Nong Ngua Hao, Thailand. The test site is located approximately 30 km east of the capital city of Bangkok, Thailand. A test embankment was also constructed in Saga Airport, about 13 km south of Saga, Japan. Some factors affecting the behavior of vertical drains were studied in the laboratory, and a back–analysis was performed to evaluate its field performance. A comparison was then made between laboratory test data and corresponding back–calculated field values of the parameters related to drain behavior. Finally, suggestions were made on determining design
values for vertical drain improvement and the methods for predicting the behavior of the improved soil.

2. SITE AND SOIL CONDITIONS

The generalized soil profile and soil properties at the Nong Ngu Hao test site are shown in Figs. 1 and 2. The soil profile is relatively uniform consisting of a layer of weathered crust (2 m thick) overlying very soft to soft clay approximately 10 m thick. Underlying the soft clay layer is a medium clay layer of about 4 m thickness followed by a stiff clay layer extending down to 22 m depth which is in turn underlain by a layer of dense sand.
Fig. 3 Cross section and plan view of the Saga airport test embankment.

At the Saga Airport test site, the soft layer is about 25 m deep consisting of 3 clay layers and 2 sand layers. Top weathered crust (B) is about 1.0 m thick followed by the first clay layer, Ac1, with a thickness of 2.5 to 3.0 m. A sand layer, As1, of 1.1 to 1.5 m thick underlies Ac1. The main clay layer, Ac2, below As1 is very soft with a thickness of 13.5 to 15.5 m. Under Ac2 is a sand layer, As2, which has a thickness of 0.5 to 2.7 m. The third clay layer, Ac3, is soft to middle stiff and has a thickness of 1.3 to 3.6 m underlying a thick and dense sand layer (DS). The soil profile is indicated in Fig. 3.

3. SELECTION OF PREFABRICATED VERTICAL DRAIN (PVD)

The PVD identified from the published information and economic considerations suitable for Nong Ngu Hao clay are: Amerdrain (408), Castle Board (CSI), Colbond (CX-1000), Flodrain (FD4-EX), Geodrain (L-type) and Mebra (MD7007). Tests were performed on these PVD to check their suitability for safe installation and their
post installation performance (Bergado et al., 1996b). Three PVD types were selected for installation in three test embankments, namely: Mebra (MD7007), Castle Board (CS1) and Amerdrain (408).

A systematic laboratory test program was carried out at the Institute of Lowland Technology in Saga University to identify the important influencing factors on discharge capacity (Miura et al., 1998). The main findings from the tests were that the discharge capacity of the drain confined in clay during one week (short term) was only about 20% of the corresponding value of confinement by rubber membrane. The long term test (lasted for about 5 months) indicated that discharge capacity reduced significantly with elapsed time. Air bubbles trapped in the drainage path of the drain reduced the discharge capacity by about 20%. The folding (no kinking) of drain, however, with a vertical strain up to 20% did not have much effect on discharge capacity, which supports the conclusion drawn by Hansbo (1983). These results indicated that the confinement condition (by clay or by rubber membrane) and test duration for the case of clay confinement are two key factors affecting the discharge capacity of a PVD.

The results of the two long term discharge capacity tests are shown in Fig. 4. The corresponding values for confining the drain by rubber membrane are also indicated in the figure for comparison under a confining pressure of 49 kPa and hydraulic gradient of 0.08. This confining pressure approximately represents the lateral earth pressure in subsoil under a 5 m high embankment, or at 10 to 15 m depth of natural subsoil. A lower hydraulic gradient (0.08) was adopted because in the field, the average value during the consolidation process is not high. For both tests, the discharge capacities continuously reduced with elapsed time, and the lowest value was about 4% of the value of confined drain by rubber membrane. For a drain, it is normally expected to work for at least half a year. Therefore, in design, the long term behavior of drain should be taken into account. Linearly converting the data to hydraulic gradient of 1.0, the lowest discharge capacity was 75 m3/year for PVD(A) and 126 m3/year for PVD(B). PVD(B) had a larger discharge capacity than PVD(A) due to a larger initial drainage channel. The factors considered for causing the discharge capacity reduction with time under clay confinement are the creep deformation of the filter under constant confining pressure and the clogging effect by very fine clay particles entering the drainage path of the drain.

To quantify the creep effect, creep tests for the drain filter were conducted. On a 200 mm wide by 400 mm long sample. The filter of the drain was tested in the direction corresponding to the transverse direction of the drain. To avoid necking, a clamp was fixed in the middle of the sample. The creep test results are given in Fig. 5. The filter of PVD(B) is weaker than that of PVD(A). For the two long term tests, the cross-sectional area reduction due to deformation (including creep) of the filter was 4% and 17% for PVD(A) and PVD(B), respectively.
Fig. 4  Result of long term discharge capacity test.

Fig. 5  Creep test results.

Fig. 6  Test embankment TS3 (4.2m height).

Fig. 7  Settlement of layers of increasing thickness from the ground surface.

Fig. 8  Comparison of computed (FEM) and measured settlement with time for embankments TS1, TS2 and TS3 with PVD.
4. TEST EMBANKMENT CONSTRUCTION AND STAGE LOADING

At the Nong Ngu Hao test site, the original ground was cleared of grass roots and excavated to −0.3 m MSL. A sand blanket of 1.0 m thick was laid on the excavated ground. After the PVD installation, the thickness of the sand blanket was increased to 1.5 m. Then, clayey sand was used to raise the embankment to 4.2 m (i.e., 75 kPa of surcharge) in stages. The Mebra drains, Castle Board drains and Flodrains (replaced Amerdrains) were inserted to a depth of 12 m on a square pattern with 1.0 m, 1.2 m and 1.5 m spacing for TS3, TS2 and TS1, respectively. The test embankments (TS1, TS2 and TS3) were 40 m x 40 m in plan dimensions with 3H:1V side slopes and finished height of 4.2 m (Fig. 6). Construction commenced in April 1994 and was completed 9 months later. The fill material was compacted to an average bulk unit weight of 18 kN/m$^3$.

At the Saga Airport test site, located 13 km south of Saga City, Japan on a reclamation land close to Ariake sea, a test embankment was constructed on the soil consisting mainly of soft and highly compressible Ariake clay. The embankment had a fill thickness of 3.5 m, base dimensions of 71 m by 71 m, and top dimensions of 25 m by 25 m. The filling speed was about 0.03 m/day. The PVDs were installed in a square pattern with a 1.5 m spacing to around 25 m deep with an area of 45 m by 45 m. Figure 2 also shows the geometry of the embankments, the main instrumentation points and the pattern of drain installation.

5. FIELD PERFORMANCE OF NONG NGU HAO TEST EMBANKMENTS

A field monitoring program was followed at the Nong Ngu Hao site to measure surface and subsurface settlements, lateral movements and excess pore pressures. A clear trend of settlement magnitudes emerged in Fig. 7 taken from test embankment TS-3. A comparison of surface settlements of test embankments TS1, TS2 and TS3 with PVD spacing of 1.5, 1.2, and 1.0m, respectively, is given in Fig. 8. Most of the compression occurred in the uppermost 8 m. In Fig. 8, the FEM predictions for surface settlements are also included. FEM using CRISP program by Britto and Gunn (1987) was used to predict the settlements. FEM also made good predictions for lateral movements and pore pressures.

In each embankment, two slope indicators labeled I1 and I2 were installed. The inclinometer I1 was located at the outermost edge of the embankment at a distance of 20 m from the center while the inclinometer I2 was installed at the shoulder at maximum fill height. Most of the lateral movements occurred in the very soft to soft clay layer. The hydraulic piezometers PH2, PH4, PH6, PH10 and PH14 were installed at depths of 2.0 m, 4.0 m, 6.0 m, 10.0 m and 14.0 m, respectively. Although the rate was slow and delayed, the dissipation of excess pore pressures definitely occurred.

Figure 9 illustrates the reduction of water content with depth for TS3 in February 1996 after 660 days of preloading compared to the mean values of the water content
Fig. 9 Water contents before and after preloading with PVD.

Fig. 10 Field vane shear strength measured in embankment TS3.

Fig. 11 Comparison of $c_{u} - q_{w}$ relation of three test embankments TS1, TS2 and TS3 using the same value of smear ratio, $k_{s}/k_{a} = 5$.

Fig. 12 Comparison of settlement – time curves for embankment on PVD improved subsoil.
measured in February 1994. Those determined in 1973 are also included as dotted lines. Similar water content reductions are noted for TS2 and TS3. For the very soft clay from 2 to 6 m depth, the water content reduction is consistent, i.e., higher reduction corresponding to smaller PVD spacing in TS3. The reduction in water content in TS3 is more than 20% which agreed with the back-calculated water contents from settlements.

The increase in undrained shear strength was predicted by the SHANSHEP technique (Ladd, 1991). The predicted increases in undrained shear strengths are indicated by solid lines in Fig. 10. The corrected undrained shear strengths measured by field vane shear tests in February 1994, May 1995 and March 1996 are also plotted for comparison. There is excellent agreement between the measured and predicted data with regards to the increase in undrained shear strength due to preconsolidation and drainage.

Using the equation of Hansbo (1979) for consolidation of PVD and the approach of Asaoka (1978), the back-calculated $C_h$ values from the hydraulic piezometer against the increase in effective stress. The $C_h$ values decreased consistently with the increase in effective stress (with the progress of consolidation) for all depths and in all three embankments. The weathered crust (2m) has the highest $C_h$ value while the weakest soil at 6 m depth has the lowest $C_h$ value. The $C_h$ value at 4 m and 10 m depths are higher than those at 6 m depth. The average values of $C_h$ (for 12 m improved ground) calculated from three embankments TS1, TS2 and TS3 at the end of construction is about 3 m²/year. The comparison of $C_h$ versus discharge capacity, $q_w$, of the 3 test embankments using the smear ratio ($k_h/k_s$) of 5 is shown in Fig. 11. The $q_w$ varied from 30 to 100 m³/yr.

6. BACK ANALYSIS OF TEST EMBANKMENTS IN SAGA AIRPORT

The field performance of improved subsoil is influenced by several factors, such as the properties of subsoil, the effect of drain, the properties of sand mat, etc. The laboratory test usually underestimates the value of field hydraulic conductivity ($C_r > 1.0$). Two parameters are needed to characterize the smear effect, namely: the diameter of the smear zone (ds) and the hydraulic conductivity ratio ($k_h/k_s$), i.e., the value in the undisturbed zone ($k_h$) over that in smear zone ($k_s$). Several investigations have been made on these factors (Jamiolkowski and Lancellotta, 1981; Jamiolkowski et al., 1983; Hansbo, 1987; Miura et al., 1993). The diameter of smear zone, ds, it can be estimated as (2 to 3)dm where dm is the equivalent diameter of the cross-sectional area of mandrel.

There are many uncertainties regarding the $k_h/k_s$ value. Hansbo (1987) proposed that $k_s$ can be same as the hydraulic conductivity of natural soil in vertical direction, $k_v$ based on laboratory test results. Laboratory tests may be a correct way for determining the value of $k_s$, but it generally underestimates the hydraulic conductivity of field deposits because of sample disturbance and sample size effect. The value of
$k_h/k_s$ can vary from 1 to 15 (Jamiolkowski et al., 1983). It is suggested that $k_h/k_s$ can be expressed as:

$$\frac{k_h}{k_s} = \left(\frac{k_h}{k_s}\right)_{C_f}$$

(1)

where subscript 1 represents the value determined in laboratory, and $C_f$ is the hydraulic conductivity ratio between field and laboratory values. For a homogeneous deposit, the $C_f$ value can be close to 1.0, but for stratified deposits, even those with thin sand layers and sand seams which cannot be clearly identified from the borehole record, the $C_f$ value can be much larger than 1.0. Experimental results indicate that the degree of disturbance and, therefore, the coefficient of consolidation in smear zone varies with the radial distance from drain (Onoue, 1991; Madhav et al., 1993). Based on this experimental evidence, a unit cell consolidation theory which considers the hydraulic conductivity in smear zone linearly or bi-linearly varying has been proposed by Chai et al. (1997).

In back-analysis, a one-dimensional (1-D) drainage element was used to simulate the drain effect. A matching procedure based on well resistance (Chai et al., 1995) was adopted to simulate the axisymmetric drainage condition in plane strain analysis not only to match the average degree of horizontal consolidation, but also to yield a more realistic excess pore pressure distribution in the horizontal direction. The equation for discharge capacity matching is as follows (Chai et al., 1995):

$$q_{wp} = \frac{4k_hl^2}{3D_e^2\left[ln\left(\frac{D_e}{d_w}\right) + \left(\frac{k_h}{k_s} - 1\right)ln\left(\frac{d_s}{d_w}\right) - 3 \cdot \frac{1}{4} + \frac{2l^2\pi k_h}{3q_w}\right] - 2B}$$

(2)

where $D_e$ is the diameter of unit cell, $l$ is drainage length, $q_w$ is the discharge capacity of drain, $q_{wp}$ is equivalent discharge capacity in plane strain and $B$ is half spacing between drainage elements in plane strain analysis. The 1-D drainage element and the proposed matching procedure as well as the hydraulic conductivity variation with void ratio function were incorporated into a finite element program, CRISP (Britto and Gunn, 1987), verified by Chai et al. (1995), and formed the analysis program for the current study.

In order to fit the measured data, either the discharge capacity of the drain or the hydraulic conductivity in smear zone was varied. Fixing smear effect case, a discharge capacity of about 85 m³/year was obtained, which almost matches the laboratory long term value. If fixing the discharge capacity at 150 m³/year, a value of $k_h/k_s = 11$ resulted. The comparison on settlement is presented in Fig. 12. The analysis simulated the field performance. The analysis slightly underpredicted the settlement during the construction period and slightly overestimated the settlement after construction period. One of the possible reasons for the discrepancies is the continuous variation (reduction) on the discharge capacity of the PVD with elapsed time.
7. CONCLUSIONS

Three full scale test embankments (TS1, TS2, TS3) were constructed in stages on soft Bangkok clay, at the proposed site of the Second Bangkok International Airport (SBIA) in Thailand and another test embankment was constructed at the Saga Airport in Saga, Japan. Prefabricated vertical drains were installed at the Thailand site to a depth of 12 m in a square pattern with spacing of 1.0m, 1.2m, 1.5m corresponding to TS3, TS2 and TS1, respectively. The PVDs at Saga Airport site were installed to a depth of 25 m in a square pattern with a spacing of 1.5 m. The water content reductions from field measurements were in good agreement with the computed values from consolidation settlements. The undrained shear strength with depth due to preconsolidation and drainage as measured in the field is in agreement with values calculated from the SHANSHEP technique. From back-analyses at SBIA site, the $C_h$ value was obtained as 3 m$^3$/yr and the corresponding discharge capacity, $q_{aw}$, varied from 30 to 100 m$^3$/yr. The CRISP FEM program incorporating Biot's theory of consolidation and Critical State Soil Mechanics yielded good predictions on the vertical and lateral movements of these test embankments. Based on the back analyses results from the Saga Airport study, suggestions are made on determining the design parameters for vertical drain improvement. For the discharge capacity test, confining the drain in clay is essential. Due to the creep of filter and the clogging caused by the fine particles entering the drainage channel, the long term discharge capacity is significantly smaller than the short term one, and this should be considered in design. For smear effect, a new equation is proposed for determining the hydraulic conductivity ratio of natural subsoil to smear zone.

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