INVESTIGATION OF FACTORS AFFECTING VERTICAL DRAIN BEHAVIOR

By Jun-Chun Chai¹ and Norihiko Miura²

ABSTRACT: Some influencing factors on vertical drain behavior were investigated by laboratory tests as well as by back-analyses of test embankments on vertical drain improved subsoil at Saga Airport, Saga, Japan. Based on the results from this study, suggestions are made on determining the design parameters for vertical drain improvement. For the discharge capacity test of a prefabricated vertical drain, confining the drain in clay is essential. Also, due to the creep of the 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 ratio of the hydraulic conductivity of the natural subsoil to that of the smear zone, which considers the fact that laboratory tests normally underestimate the hydraulic conductivity of natural deposits. Regarding the effect of the sand mat, the numerical analysis results in this study show that if the hydraulic conductivity of sand is larger than 10⁻⁴ m/s, the assumption of a free drainage condition in the sand mat may not result in significant error. Finally, a methodology of predicting the behavior of vertical drain improved subsoil is proposed.

INTRODUCTION

Installing a drainage material vertically into the ground can shorten the drainage path of soft clay deposits significantly, and, combined with preloadings, it can improve the stiffness and strength of the ground substantially in a short period. In the past few decades, vertical drain improvement techniques have been widely applied in soft soil engineering, such as in embankment construction on soft ground.

The theoretical solution for vertical drain consolidation was first proposed for the unit cell condition (i.e., a single drain surrounded by a soil cylinder) by Barron (1948). Further studies on unit cell behavior were made by Yoshikuni (1979) and Hansbo (1981). Since, in many cases, ground deformation patterns do not represent a unit cell condition, some techniques have been developed for simulating the vertical drain effect in two-dimensional finite-element analysis (Chai et al. 1995). It can be said that the basic theory for the design of the vertical drain improvement has been established. However, field engineers often face a problem that the expected (or theoretical) effect of a vertical drain could not be achieved in field, especially when comparing the behavior of improved subsoil with an unimproved case at the same site. There could be two reasons: (1) a laboratory test overestimates the drain effect, such as the discharge capacity of drains; and (2) analysis assumptions do not represent the actual field conditions, such as the drainage boundary condition in the field. In order to improve the design method, it is useful to investigate the main factors influencing vertical drain behavior and to compare the laboratory data with back-calculated field performance values. By so doing, both laboratory test methods and analysis assumptions can be improved to represent actual field conditions more closely.

In this paper, firstly, some factors affecting the behavior of vertical drains are studied, followed by the back-analysis of the test embankments at Saga Airport, Saga, Japan, to evaluate the field performance of prefabricated vertical drains (PVDs) as well as sand drains (SDs). Then, a comparison is made between laboratory test data and corresponding back-calculated field values of the parameters related to drain behavior. Finally, suggestions are made on determining design values for vertical drain improvement, and the methods for predicting the behavior of improved subsoil.

FACTORS AFFECTING VERTICAL DRAIN PERFORMANCE

One of the most important parameters affecting the consolidation rate of vertical drain improved subsoil is the radial coefficient of consolidation of subsoil. There is no satisfactory laboratory test method to determine this parameter, and back-analysis from field measurements or field hydraulic conductivity test is recommended for evaluating the design value. This study mainly investigates other important influencing factors. For a given subsoil condition, the effect of vertical drains depends on (1) drain spacing and equivalent drain diameter; (2) well resistance (discharge capacity); (3) smear effect; and (4) drainage boundary condition. In quantifying the influence of these factors, some uncertainties exist, except for the drain spacing. As a general tendency, the larger the equivalent drain diameter, the smaller the well resistance (the larger the discharge capacity), and the smaller the smear effect, the more effective the vertical drain. The amount of the effect of each uncertain factor on the consolidation rate of improved subsoil can be easily calculated by Hansbo’s analytical solution (Hansbo 1981). The following section focuses on how to estimate the design values of these parameters.

Equivalent Drain Diameter

For the equivalent drain diameter \(d_e\) of a band-shaped drain, the first proposal was made on the equal drainage perimeter assumption (Hansbo 1979). Subsequent research indicates that, due to the corner effect, the equivalent drain diameter is less than the value calculated based on an equal drainage perimeter assumption, and a new equation based on the finite-element analysis result has been suggested as (Rixner et al. 1986)

\[
d_e = \frac{w + t}{2}\]

where \(w\) and \(t\) = width and thickness of a band-shaped drain, respectively. Eq. (1) has been widely used in practice and was also verified by the finite-element analysis in this study.

Discharge Capacity of PVD

The discharge capacity of a vertical drain must be determined experimentally. An ideal discharge capacity test should...
simulate the drain installation, the confinement of clay on the filter sleeve of drain, and the deformation of the drain during consolidation. It is obvious that a full-scale test could be expensive if it is ever possible. For a useful small-scale laboratory test, the important influencing factors, such as the confinement condition, must be considered.

A systematic laboratory test program was carried out to identify the important influencing factors on discharge capacity (Miura et al. 1998). The following factors were investigated:

1. Confining the drain by clay (compared with confining the drain by rubber membrane)
2. The effect of possible air bubbles trapped in the drainage path
3. The effect of folding of the drain
4. Long-term discharge capacity

The main findings from the tests were as follows:

1. For the case investigated, the discharge capacity of confining the drain by clay during one week (short-term) was only about 20% of the corresponding value of confinement by rubber membrane. One long-term test (lasting for about five months) indicated that discharge capacity reduced significantly with elapsed time.
2. Air bubbles trapped in the drainage path of the drain reduced the discharge capacity about 20%.
3. The folding (no kinking) of the drain 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. To confirm this point and to identify the reasons for discharge capacity reduction with time in the case of clay confinement, additional discharge capacity tests with clay confinement were conducted in the current study. In order to quantify the creep deformation of a filter under given confining pressure, creep tests were also carried out for the filter of the drain tested.

**Discharge Capacity Tests with Clay Confinement**

The test device is illustrated in Fig. 1. The apparatus mainly consists of a cylindrical cell with a diameter of 200 mm and height of 600 mm. The maximum length of the drain can be tested is up to 400 mm. The test procedure is as follows:

1. Install the drain sample at the lower pedestal, which is connected to the inlet water flow system.
2. Fix the membrane at the lower pedestal, and put the cylindrical mold (two halves) in position with the membrane inside. The inner diameter of the mold is 100 mm.
3. Put the remolded clay (at a water content close to liquid limit) into the mold layer by layer until the desired height is reached. Each layer is compacted by a stick (about 10 mm in diameter) so that air bubbles do not remain in the sample. However, the initial density of the sample is basically controlled by the water content.
4. Install the upper pedestal and connect the drain sample to the outlet water flow system. Fix the membrane to the upper pedestal.
5. Apply a suction of about 10 kPa to the sample and remove the mold.
6. Set up the confining pressure, and gradually release the suction.

**FIG. 1. Illustration of Discharge Capacity Test Device**

7. Set up the desired confining pressure (49 to 392 kPa) and hydraulic gradient (0.08 to 0.8). Then let the clay sample consolidate under a given confining pressure. After the clay sample is consolidated, the discharge capacity is measured. The consolidation of the sample takes about one week. Therefore, the measurement after one week is defined as the short-term value for the confined-in-clay test.

There is a slot on the lower and upper pedestals for inserting the band-shaped drain. The gap between the drain sample and the slot needs to be sealed by plasticine so that the clay particles do not enter the drainage path. Also, during the process of putting the clay into the mold, the top of the drain sample should be covered by a plastic sheet to prevent clay from falling into the drainage channels of the drain. Another point worth mentioning is that this setup is not a unit cell test. It just simulates the clay (smear zone) confinement on the sleeve of the drain.

Table 1 gives the properties of the drains tested. PVD(A) was also used in the field for a test embankment at Saga Airport. The remolded Ariake clay used had a liquid limit of 105.0% and a plastic limit of 42.8%. The soil consisted of 57.0% clay, 41.7% silt, and 1.3% sand particles.

The results of two long-term tests are shown in Fig. 2. The corresponding values for confining the drain by rubber membrane are also indicated in the figure for comparison. The test conditions were the confining capacity of 49 kPa and the 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; this factor will be discussed in a later section. For both tests, the discharge capacities continuously reduced with elapsed time, and the lowest value was about 4% of the value of the drain confined by rubber membrane. A drain is normally expected to work for at least half a year. Therefore, in design, the long-term behavior of a drain should be taken into account. When linearly converting the data to a hydraulic gradient of 1.0, the lowest discharge capacity was 75 m³/yr for PVD(A) and 126 m³/yr for PVD(B). PVD(B) had a larger discharge capacity than PVD(A) due to a larger initial drainage channel (Table 1). The factors considered for causing the discharge capacity reduction with time under clay confinement are (1)
TABLE 1. Physical Properties of PVDs

<table>
<thead>
<tr>
<th>PVD (1)</th>
<th>Size</th>
<th>Drainage Channel</th>
<th>Material</th>
<th>Connection condition between filter and core (10)</th>
<th>Structure (11)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Thick-</td>
<td>Width</td>
<td>Depth</td>
<td>Width</td>
<td>Number of channel/drain</td>
</tr>
<tr>
<td></td>
<td>ness (mm)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(mm)</td>
<td>(6)</td>
</tr>
<tr>
<td>PVD(A)</td>
<td>2.6</td>
<td>94</td>
<td>1.5</td>
<td>1.8</td>
<td>40</td>
</tr>
<tr>
<td>PVD(B)</td>
<td>3.6</td>
<td>97</td>
<td>1.3</td>
<td>2.0 ~ 2.4</td>
<td>64 ~ 66</td>
</tr>
</tbody>
</table>

FIG. 2. Amount of Water Flow versus Elapsed Time for Long Term Discharge Capacity Tests

FIG. 3. Illustration of Setup of Creep Test

FIG. 4. Creep Test Results

The creep deformation of the filter under constant confining pressure; and (2) the clogging effect of fine particles entering the drainage path of the drain.

Creep Test for Drain Filter

To quantify the creep effect, creep tests for the drain filter were conducted. The test device is illustrated in Fig. 3. The filter of the drain was tested in the direction corresponding to the transverse direction of drain. The sample tested was 200 mm in width and 400 mm in length. To avoid the necking of the sample, a clamp was fixed in the middle of the sample. The measuring ruler was fixed on the right side of the frame (Fig. 3). The creep test results are given in Fig. 4. It can be seen that the filter of PVD(B) is weaker than that of PVD(A). To relate the creep deformation of a filter with the reduction of cross-sectional area of a drainage channel, the following assumptions are used:

1. As illustrated in Fig. 5, the deformed shape of a filter is assumed to be a circular arc.
2. The confining pressure is balanced by the component of tension force \( T \) of the filter in the \( y \)-direction (Fig. 5).

By trial and error, the relationship between confining pressure and reduction of the cross-sectional area was obtained as shown in Fig. 6. This figure shows that for the two long-term tests (Fig. 2), the cross-sectional area reduction due to deformation (including creep) of the filter was 4% and 17% for PVD(A) and PVD(B), respectively.

Investigation of Clogging Effect

For the test of PVD(A) in Fig. 2, after about 130 days, the hydraulic shocks (by firmly stepping on the inlet water flow hose) were applied to examine the clogging effect. It was observed that some flocculated fine particles were pressured out (due to being stepped on) of the drainage channel of the drain and deposited on the wall of the outlet hose, i.e., the part of clogging effect was removed by the hydraulic shocks. As shown by the dashed line in Fig. 2, the discharge capacities increased after the shocks.

Also, after the tests, the fine particles in the drainage channel were collected and examined by electron microscopic photographs. It has been confirmed that the size of the particles in the drainage channel was in the range of fine clay particles.
particles were coarser than those in contact with the core (location B in Fig. 5), which indirectly shows that the fine particles entered the drainage channel through the filter. Figs. 7(a and b) show the electron microscopic pictures of the filter at the open channel area (location A in Fig. 5). Fig. 7(a) is for the drainage channel side, and Fig. 7(b) is for the side in contact with the clay. Fig. 7(a) shows some biofilm formed in the drainage channel side. On the side in contact with the clay, there is no biofilm, but some soil particles are inserted in the opening of the filter. It is considered that the formation of biofilm together with the fine particles entering the drainage channel caused the partial clogging of the drainage path, and resulted in the reduction of discharge capacity with time.

**Smear Effect**

In the field, the drain is installed by using a mandrel, which is pushed into ground. Then the mandrel is withdrawn, leaving the drain in subsoil. This process creates a completely disturbed zone around the drain, called the smear zone. The hydraulic conductivity in the smear zone will be reduced significantly, which affects the behavior of the drain. Two parameters are needed to characterize the smear effect, namely, the diameter of the smear zone \(d_s\) and the hydraulic conductivity ratio \(k_h/k_s\), i.e., the value in the undisturbed zone \(k_h\) over that in the 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 the smear zone, \(d_s\), can be estimated as

\[
d_s = (2 \text{ to } 3)d_m
\]

where \(d_m\) = equivalent diameter of the cross-sectional area of the mandrel. In design, if no test data are available for evaluating the smear zone size, the value of \(d_s = 3d_m\) is suggested. Since the shape of the mandrel is rectangular, the shape of the actual disturbed zone is close to a rectangle or an ellipse. In unit cell theory, the smear zone is represented by a circular shape based on the equal area assumption. This factor has been checked by finite-element analysis in this study. A plane strain slice (circular or rectangular) perpendicular to the drain direction was considered for evaluating the effect of smear zone shape. A quarter of the slice was used for a finite-element analysis (Fig. 8). The boundary conditions are also shown in the figure. During the analysis, first a loading of 50 kPa was applied under the undrained condition, and then the consolidation process was simulated. The soil was assumed as elastic with a Young’s modulus of 10,000 kPa, a Poisson’s ratio of
0.3, \( k_s = 10^{-4} \, \text{m/s} \), and \( k_h = k_s/5 \). Fig. 9 compares the average degree of consolidation of a rectangular smear zone with that of a circular one under the equal area condition. It can be seen that the shape does not make a significant difference in the degree of consolidation (\( U \)) versus time factor (\( T \)) curves. The assumption of a circular smear zone is reasonable.

There are many uncertainties regarding the \( k_s/k_h \) value. Since, for most natural deposits, the hydraulic conductivity in the horizontal direction is higher than that in the vertical direction, Hansbo (1987) proposed that \( k_h \) can be the same as the hydraulic conductivity of natural soil in the vertical direction, \( k\). The value of \( k_s/k_h \) can vary from 1 to 15 (Jamiokowski et al. 1983). It is argued here that the assumption of \( k_s = k_h \) is mainly based on laboratory test results. Laboratory tests may be a correct way to determine the value of \( k_s \), but they generally underestimate the hydraulic conductivity of field deposits because of sample disturbance and sample size effect. It is suggested that \( k_s/k_h \) can be expressed as

\[
\frac{k_s}{k_h} = \left( \frac{k_h}{k_s} \right)_{0} \cdot C_f
\]  

(3)

where subscript \( l \) represents the value determined in the laboratory; and \( C_f = \text{hydraulic conductivity ratio between field and laboratory values. In some cases}, \ (k_s/k_h)_{0} = (k_h/k_s)_{0}. The best way to estimate the value of field hydraulic conductivity is to back-analyze local case histories, such as measured embankment settlement, or to measure hydraulic conductivity in the field using a self-boring permeameter if this sophisticated equipment is available (Tavenas et al. 1986). It is considered that the most important factors affecting the value of \( C_f \) are the deposit stratifications. 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 that cannot be clearly identified from the borehole record, the \( C_f \) value can be much larger than 1.0. The \( C_f \) values of a few clay deposits are listed in Table 2 for reference.

For the unit cell theory proposed by Hansbo (1981), a single value of hydraulic conductivity is used in the smear zone. However, experimental results indicate that the degree of disturbance, and therefore the coefficient of consolidation in the smear zone, varies with the radial distance from the drain (Onoue 1991; Madhav et al. 1993). Based on this experimental evidence, a unit cell consolidation theory that considers the hydraulic conductivity in the smear zone varying linearly or bilinearly has been proposed by Chai et al. (1997). It has been demonstrated that with the average value of hydraulic conductivity in the smear zone, the smear effect will be underestimated. The value of hydraulic conductivity at the inner smear zone (a zone adjacent to the drain surface) has a larger effect on the consolidation behavior of the unit cell. It is recommended to determine the hydraulic conductivity in the smear zone by laboratory tests with the soil sample adjacent to the drain surface.

**Effect of Sand Mat**

Part or all of the water collected by the drain will flow to the ground surface first, and then drain out by the outlet system, the sand mat. Since the hydraulic conductivity of sand is considerably higher than that of clay, usually, in analysis, it is assumed that there is no hydraulic resistance in the sand mat. If a thick layer (more than 0.5 m) of clean sand (percentage fines less than 5%) is used, this assumption may not result in much error. However, in some cases, due to the availability of material as well as cost considerations, lower quality sand such as clayey sand is used for the sand mat. In this case, the hydraulic conductivity in the sand mat may have some influence on the rate of consolidation of vertical drain improved subsoil. The amount of resistance in the sand mat is a function of the hydraulic conductivity of the sand as well as the geometry of the embankment. This factor will be discussed further in the following section using a test embankment at Saga Airport as an example.

**BACK-ANALYSIS OF TEST EMBANKMENTS IN SAGA AIRPORT**

**Methodology of Back-Analysis**

The field performance of improved subsoil is influenced by several factors, such as the properties of the subsoil, the effect of the drain, and the properties of the sand mat. In order to back-calculate the field behavior of a PVD and a SD, the properties of the subsoil and the analysis method should be verified first. As mentioned previously, the laboratory test usually underestimates the value of field hydraulic conductivity (\( C_f > 1.0 \)). In back-analysis, if using a value lower than the actual value for the hydraulic conductivity of natural deposit, the effect of the drain will be overestimated. One of the test

<table>
<thead>
<tr>
<th>Deposit Description</th>
<th>( C_f ) value</th>
<th>Method for evaluating field value of hydraulic conductivity</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok clay at Asian Institute of Technology campus (about 100 km from sea)</td>
<td>25</td>
<td>Back-analysis</td>
<td>Chai et al. (1995)</td>
</tr>
<tr>
<td>Bangkok clay at Nong Ngu Hao (close to sea)</td>
<td>4</td>
<td>Back-analysis</td>
<td>Chai et al. (1996)</td>
</tr>
<tr>
<td>Malaysia Muar clay deposit</td>
<td>2</td>
<td>Back-analysis</td>
<td>Chai and Bergado (1993)</td>
</tr>
<tr>
<td>Ariake clay (close to sea area)</td>
<td>4</td>
<td>Self-boring permeameter</td>
<td>Tavenas et al. (1986)</td>
</tr>
<tr>
<td>Louisville (Canada)</td>
<td>About 1°</td>
<td>Self-boring permeameter</td>
<td>Present study</td>
</tr>
<tr>
<td>St-Alban (Canada)</td>
<td>About 3°</td>
<td>Self-boring permeameter</td>
<td>Tavenas et al. (1986)</td>
</tr>
</tbody>
</table>

*Laboratory value was determined by direct measurement. For other cases, laboratory values were deduced from \( C_f \) value (\( C_f \) is coefficient of consolidation).
embankments at Saga Airport was built on natural subsoil, and it was used to calibrate the model parameters as well as the numerical procedure. Then, with fixed soil parameters, the field performance data of the drain were evaluated.

As discussed previously, there are mainly two uncertain factors on drain behavior, namely, smear effect and well resistance. Since these factors are independent, a set of base values was determined based on laboratory values first, then the possible field values were evaluated by varying one parameter at a time.

**Modeling Drain Effect**

In back-analysis, a one-dimensional (1D) 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. The procedure not only matches the average degree of horizontal consolidation, but also yields a more realistic excess pore pressure distribution in the horizontal direction. The equation for discharge capacity matching is as follows (Chai et al. 1995):

\[
q_{wp} = \frac{4k_d l^2}{3D} \ln \left[ \frac{D}{d_r} \right] + \left( \frac{k_s}{k_r} - 1 \right) \ln \left( \frac{d_r}{d_w} \right) - \frac{3}{4} + \frac{2l^2 - \pi k_s}{3q_{wp}} - 2B
\]

where \( D_r \) = diameter of unit cell; \( l \) = drainage length; \( q_w \) = discharge capacity of the drain; \( q_{wp} \) = equivalent discharge capacity in plane strain; and \( B \) = half spacing between drainage elements in plane strain analysis. The 1D 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), and formed the analysis program for current study. The program was verified by Chai et al. (1995).

**Brief Description of Test Embankment in Saga Airport**

The test embankments at Saga Airport were reported by Bergado et al. (1996), and a brief description is given here. Saga Airport is located 13 km south of the city of Saga on reclaimed land close to Ariake Bay. The deposit mainly consists of soft and highly compressible Ariake clay. For verifying the effect of vertical drain improvement for this deposit, three test embankments were constructed on natural, PVD improved, and SD improved subsoil (Bergado et al. 1996). The embankments had the same geometry, with a fill thickness of 3.5 m, base width of 71 by 71 m, and top width of 25 by 25 m. The filling speed was about 0.03 m/day. The PVD and SD were installed to around 25 m deep with an area of 45 by 45 m [Fig. 10(b)]. PVD was installed in a square pattern with a...
spacing of 1.5 m. SD was also installed in a square pattern. At one time, four drains in a group were installed with a spacing between the drains of 1.2 m. The spacing between groups was 2.0 m, so that the average spacing was 1.6 m. Fig. 10 shows the geometry of the embankments, the main instrumentation points, and the pattern of drain installation.

At the test site, the soft layer is about 25 m deep, consisting of three clay layers and two sand layers. The thickness of the soil layers slightly varies at three embankment locations. The top weathered crust (B) is about 1.0 m thick, followed by the first soft 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 for PVD section is indicated in Fig. 10 also.

**Boundary Conditions and Model Parameters**

In finite-element analysis, the plane strain condition was assumed. The modeled range was 30 m deep from ground surface, and horizontally 120 m away from the embankment centerline. The displacement boundary conditions were as follows: at bottom, both vertical and horizontal displacements were fixed, and for left and right vertical boundaries, the horizontal displacement was fixed. The adopted drainage boundary conditions were as follows: the ground surface and bottom line (sand layer) were drained. The left and right boundaries were undrained, but the right boundary (away from the embankment centerline) at the location of the sand layer was drained. Fig. 11 shows the finite-element mesh for the PVD section, and the construction history is also indicated in the figure. For the zone with vertical drains, 1D drainage elements coincide with every other vertical line. The mechanical behavior of the clay layers was represented by the modified Cam clay model (Roscoe and Burland 1968) and the sand layers as well as decomposed granite fill material were assumed to be elastic. The determined model parameters for subsoil are listed in Table 3. For the clay layers, the parameters were determined from a laboratory consolidation test and triaxial test results on undisturbed samples (Bergado et al. 1996), except for Poisson’s ratio and the value of hydraulic conductivity. Poisson’s ratio, \( \nu \), was assumed empirically. For the value of hydraulic conductivity, first the ratio of \( k_h/k_v \) was determined from laboratory test results (Park 1994) as 1.5. Then the values of hydraulic conductivity in the vertical direction were adjusted to fit the observed field data of the embankment on natural subsoil, which are about four times the laboratory values (\( C_f = 4 \)). For the sand layers, Young’s modulus was estimated by referring to standard penetration test results (N value). The values of hydraulic conductivity were assumed. The values of hydraulic conductivity listed in Table 3 were initial ones; during consolidation, they varied with void ratio according to Taylor’s equation (Taylor 1948). Parameter \( C_f \) in the equation was taken as 0.4\( e_0 \) (\( e_0 \) is the initial void ratio). The subsoils are in lightly overconsolidated to normally consolidated states with a maximum overconsolidation ratio (OCR) of about 4 for the top crust. The lateral earth pressures were calculated using the equation proposed by Mayne and Kulhawy (1982). The ground-water level was about 1.0 m below ground surface. The mechanical properties of the fill material were assumed.

![FIG. 11. Typical Finite-Element Mesh](image)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Young’s modulus, ( E ) (kPa)</th>
<th>Poisson’s ratio, ( \nu )</th>
<th>( \kappa )</th>
<th>( \lambda )</th>
<th>( M )</th>
<th>Initial void ratio, ( e_0 )</th>
<th>Unit weight, ( \gamma ) (kN/m(^3))</th>
<th>( k_h ) (10(^{-8}) m/s)</th>
<th>( k_v ) (10(^{-8}) m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>—</td>
<td>0.25</td>
<td>0.025</td>
<td>0.25</td>
<td>1.3</td>
<td>2.00</td>
<td>15.0</td>
<td>11.45</td>
<td>7.60</td>
</tr>
<tr>
<td>Ac1</td>
<td>10,000</td>
<td>0.30</td>
<td>0.044</td>
<td>0.44</td>
<td>1.2</td>
<td>2.00</td>
<td>14.5</td>
<td>5.7</td>
<td>3.8</td>
</tr>
<tr>
<td>As1</td>
<td>15,000</td>
<td>0.30</td>
<td>0.087</td>
<td>0.87</td>
<td>1.2</td>
<td>2.50</td>
<td>14.5</td>
<td>2.64</td>
<td>1.76</td>
</tr>
<tr>
<td>Ac2</td>
<td>30,000</td>
<td>0.30</td>
<td>0.030</td>
<td>0.30</td>
<td>1.3</td>
<td>1.75</td>
<td>16.0</td>
<td>2.64</td>
<td>1.76</td>
</tr>
</tbody>
</table>

Note: \( \lambda \) = virgin loading slope in \( e - \ln(p') \) plot (\( p' \) is effective mean stress); \( \kappa \) = reloading/unloading slope in \( e - \ln(p') \) plot; and \( M = \) slope of failure line in \( p' \) versus \( q \) plot (\( q \) is deviator stress).
TABLE 4. Parameters Related to Drain Behavior

<table>
<thead>
<tr>
<th>Item</th>
<th>Symbol</th>
<th>Unit</th>
<th>PVD</th>
<th>SD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drain diameter</td>
<td>$d_w$</td>
<td>mm</td>
<td>48.3</td>
<td>120</td>
</tr>
<tr>
<td>Unit cell diameter</td>
<td>$D_e$</td>
<td>m</td>
<td>1.7</td>
<td>1.8</td>
</tr>
<tr>
<td>$D_e/d_w$</td>
<td>$n$</td>
<td>---</td>
<td>35.2</td>
<td>15</td>
</tr>
<tr>
<td>Smear zone diameter</td>
<td>$d_s$</td>
<td>mm</td>
<td>300</td>
<td>412</td>
</tr>
<tr>
<td>Hydraulic conductivity ratio</td>
<td>$k_h/k_s$</td>
<td></td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>$d_s/d_w$</td>
<td>$s$</td>
<td>---</td>
<td>6.2</td>
<td>3.4</td>
</tr>
<tr>
<td>Discharge capacity</td>
<td>$q_w$</td>
<td>m$^3$/yr</td>
<td>75–150</td>
<td>35–265</td>
</tr>
</tbody>
</table>

as follows: Young’s modulus of 15,000 kPa and Poisson’s ratio of 0.2. The unit weight of the fill material was 20 kN/m$^3$.

The parameters for PVD and SD are listed in Table 4. The equivalent drain diameter for PVD was calculated by (1), and the diameter of the smear zone was estimated as 3.0 times the equivalent mandrel diameter (Miura et al. 1993). The smallest value of hydraulic conductivity in the inner smear zone for Ariake clay was reported as 1/5 of that undisturbed zone (Madhav et al. 1993). If assuming a linear variation of hydraulic conductivity in the smear zone, from laboratory test results, a single value of representative hydraulic conductivity in the smear zone (equal smear effect) (Chai et al. 1997) is 0.4 and 0.43 times that of the undisturbed zone for the PVD and SD cases, respectively. Since the ratio between field and laboratory hydraulic conductivity, $C_f$, is 4, according to (3), $k_h/k_s$ is about 10 for the PVD case and 9 for the SD case. The base value of discharge capacity for PVD was from laboratory long-term test results of confining the drain in clay, and the value for SD was computed using the laboratory value of hydraulic conductivity of the sand used ($1.0$ to $7.4 \times 10^{-4}$ m/s).

Analysis of Embankment on Natural Subsoil

The purpose of analyzing the embankment on natural subsoil is to verify the model parameters as well as the numerical procedure. The initial values of hydraulic conductivity in Table 3 were adjusted by comparing the numerical results with the measured data of the embankment on natural subsoil. The simulated results are compared with measured data in Figs. 12–14 for settlement, lateral displacement, and excess pore pressure, respectively. As can be seen from Fig. 12, the analysis simulated the settlement curve well. The comparisons for lateral displacement and excess pore pressure are fair (Figs. 13 and 14). For the piezometer point in the Ac2 layer, after embankment construction, the measured excess pore pressure continuously increased, and the reason is not clear. These comparisons indicate that the analysis simulated the behavior of the natural subsoil reasonably well. In back-analyzing the drain behavior, only the parameters related to drain behavior were adjusted to fit the measured data. In addition, the effect of the sand mat was investigated using the embankment on PVD improved subsoil as an example.

Analysis of Embankment on PVD Improved Subsoil

1. Back-calculated values. In order to fit the measured data, either the discharge capacity of the drain or the hydraulic conductivity in smear zone was varied, and the back-calculated values are listed in Table 5. It can be seen that for fixing the smear effect case, a discharge capacity of about 85 m$^3$/yr was obtained, which almost matches the laboratory long-term value (Fig. 2). The back-estimated field discharge capacity is evaluated under the assumption that the discharge capacity is a constant. However,
the laboratory test indicates that the discharge capacity reduces with elapsed time. As will be discussed for comparing the settlement curves, the discharge capacity of PVD also seems to reduce with time in field. If fixing the discharge capacity as 150 m³/yr (value at 100 day in Fig. 2 for PVD(A)), then \(k_h/k_s = 11\).

2. Comparison between measured and simulated values. The comparisons on settlement, lateral displacement, and excess pore pressure are presented in Figs. 15–17 to show how well the analysis simulated the field performance. Since the target in back-analysis was the settlement, the numerical results simulated the field data well (Fig. 15). However, checking carefully, one will notice that the analysis slightly underpredicted the settlement during the construction period and slightly overestimated the settlement for the period after construction. One possible reason is the continuous variation (reduction) of discharge capacity of PVD with elapsed time, i.e., the initial discharge capacity might be larger than 85 m³/yr, and the value might be less than 85 m³/yr in a later consolidation period. The simulated excess pore pressure is lower than the measured one (Fig. 17), and the reason is not clear. The lateral displacement is larger than that of the embankment on natural subsoil because the location of the inclinometer casing was different. For the embankment on PVD improved subsoil, the inclinometer casing was 9 m away from the embankment centerline and had a larger lateral displacement. If comparing at the same location (5 m away from the embankment centerline), numerical results indicated that improved subsoil had a smaller lateral displacement. Another interesting fact is that, for the improved subsoils, the lateral displacement at the ground surface is very small. This is partially because the effect of the drain increased the rate of consolidation, and therefore the stiffness, of the soil near the ground surface.

3. Calculated hydraulic gradient \((i)\) in the drain. Hydraulic gradient in the drain also has an influence on discharge capacity of drain. The higher the \(i\) value, the less the effect of trapped air bubbles, and the less fine clay particles can deposit on the wall of the drainage channel. The laboratory test should be conducted with an \(i\) value close to the field condition. For the PVD section, the numerical result indicates that the highest \(i\) value of 0.3 corresponded to the end-of-construction condition near the drainage boundary. At deeper levels, the \(i\) value was about 0.1.

4. Effect of sand mat. As mentioned previously, in some cases, the sand mat under the embankment may have certain hydraulic resistance if lower-quality materials, such as clayey sand, are used. The hydraulic conductivity of the sand mat was assumed to vary from \(10^{-2}\) m/s to \(10^{-4}\) m/s to investigate its effect on the behavior of improved subsoil. Fig. 18 gives the results of settlement,
and shows that when the hydraulic conductivity of the sand mat is less than $10^{-4} \text{ m/s}$, it will affect the behavior of improved subsoil. The sand mat used for the test embankments had a hydraulic conductivity of about $10^{-4} \text{ m/s}$, so it may be sufficient for functioning as a free drainage path.

**Analysis of Embankment on SD Improved Subsoil**

1. Back-calculated SD behavior. For the SD case, the back-calculated discharge capacity is within the range of the laboratory value (Table 5). This result suggests that for the SD method, the laboratory value of the hydraulic conductivity of sand can be directly used for design by considering the possible reduction of hydraulic conductivity caused by (1) the nonuniformity of sand used; and (2) the partial saturation effect in field. If fixing $q_w$ as 265 m$^3$/yr (upper value), a value of $k_s/k_s = 10$ was obtained, which is not different much from the base value in Table 4.

2. Comparison between measured and simulated values. Since the tendency is the same as the PVD case, only the settlement is compared in Fig. 19. For this case, the analysis simulated the settlement curve very well. Comparing with the PVD case, there is no overprediction after construction for the SD case, which indicates that the drainage capacity of SD might not change much during the consolidation of subsoil.

3. Calculated hydraulic gradient in the drain. Just as in the PVD section, a maximum hydraulic gradient of about 0.3 was obtained near the drainage boundary. The hydraulic gradient in the drain is a function of construction speed, embankment height, soil profile, and the discharge capacity of the drain. Therefore, it is difficult to suggest a single value for a laboratory discharge capacity test. Two more cases of embankment on PVD improved subsoil were analyzed by the same method. One is the test embankment on Malaysia Muar clay (case A), and another is the test embankment on Bangkok clay (case B) (Chai et al. 1996). For case A, the embankment had a fill thickness of 4.7 m and was constructed in about 150 days. For this case, a drain without a filter (the small holes on the core for drainage) was used. It seems that the drain did not work well, and a maximum hydraulic gradient of about 1.0 was obtained near the drainage boundary, and about 0.2 for a deeper location. For case B, the embankment had a fill thickness of 4.2 m and the construction period was about 210 days. Maximum hydraulic gradients of about 0.7 near the drainage boundary and 0.15 for deeper locations were calculated. Considering the factor that after construction, the hydraulic gradient in the drain will gradually reduce, it is suggested that for most cases, an average hydraulic gradient during the process of consolidation will be far below 1.0. This can be used as a reference for determining the value of hydraulic gradient for the laboratory discharge capacity test of PVD and the hydraulic conductivity test of sand for SD improvement.

**Discussion**

The foregoing comparison indicates that the laboratory confined-in-clay discharge capacity tests are useful in determining the parameters related to drain behavior. Then, with correct information about the properties of natural subsoil, the behavior of vertical drain improved subsoil can be predicted. It is emphasized that at present, the laboratory test has a limitation on determining the value of hydraulic conductivity of natural deposit, and it is recommended to back-evaluate from a test embankment on natural subsoil or an existing case history.

In many cases, the test embankments were only constructed on improved subsoil, and the actual effect of the drain cannot be correctly evaluated. Based on the results of this study, a method of predicting the behavior of vertical drain improved subsoil is proposed as follows:

1. Evaluate the properties of a natural deposit using a test embankment on natural subsoil or an existing case history, or use verified field and laboratory tests, such as a field hydraulic conductivity test.

2. Determine the discharge capacity of the vertical drain by the long-term confined-in-clay test, and evaluate the $k_s/k_s$ value by (3).

3. Predict the behavior of vertical drain improved subsoil by a proper method, such as the unit cell method for predicting the settlement versus time curve under the embankment centerline. If it is available, finite-element analysis can also be adopted.

**SUMMARY AND CONCLUSIONS**

Some factors affecting the behavior of a vertical drain were investigated by laboratory tests as well as back-analysis of field performance data. Based on the results from this study, the following suggestions are made on determining the design parameters for vertical drain improvement:

1. Evaluate the properties of a natural deposit using a test embankment on natural subsoil or an existing case history, or use verified field and laboratory tests, such as a field hydraulic conductivity test.

2. Determine the discharge capacity of the vertical drain by the long-term confined-in-clay test, and evaluate the $k_s/k_s$ value by (3).

3. Predict the behavior of vertical drain improved subsoil by a proper method, such as the unit cell method for predicting the settlement versus time curve under the embankment centerline. If it is available, finite-element analysis can also be adopted.
1. The discharge capacity of the drain is one of the main influencing factors on vertical drain behavior. The laboratory discharge capacity test for PVD should be conducted by confining the drain in clay. Also, if time allows, it is suggested to conduct the long-term test. For a sand drain, it is suggested that the laboratory value of hydraulic conductivity can be used for design by considering the possible reduction of hydraulic conductivity in the field due to the nonuniformity of the sand, and the partial saturation condition in field. For both the hydraulic conductivity test of sand and the discharge capacity test of PVD, an upper hydraulic gradient of 1.0 is suggested.

2. The smear zone has a significant effect on the consolidation rate of the drain. The diameter of the smear zone ($d_s$) can be estimated as two to three times the equivalent mandrel diameter ($d_m$). If there are no test data, $d_s = 3d_m$ is suggested. On determining the hydraulic conductivity ratio between natural subsoil and the smear zone, the fact that a laboratory test usually underestimates field hydraulic conductivity should be considered. A new equation, (3), is proposed. Also, for the hydraulic conductivity test of the smear zone, it is recommended to use the soil sample adjacent to the drain surface.

3. It is normally assumed that a sand mat placed on the top of the drain is a free drainage path. The limited numerical results indicate that if the hydraulic conductivity of sand is larger than $10^{-4}$ m/s, the free drainage path assumption is acceptable.

A methodology for predicting the behavior of vertical drain improved subsoil is also proposed. The first step in the method is to evaluate the properties of natural deposit by back-analysis of a test embankment on natural subsoil or an existing case history. The second step is to determine the drain parameters with the methods already described. Then the behavior of improved subsoil can be predicted by a proper method, such as the unit cell method or finite-element method.

ACKNOWLEDGMENTS

The writers wish to express their appreciation to D. T. Bergado at the Asian Institute of Technology, Bangkok, Thailand, and Yudhbir at the Indian Institute of Technology, Kanpur, India, for their valuable comments during preparation of this paper. Sincere thanks are due to K. Toyoda and E. Takeshita, graduates of Saga University, Japan, for conducting part of the laboratory tests and assisting with preparing figures.

APPENDIX. REFERENCES

