SIMPLE METHOD OF MODELING PVD-IMPROVED SUBSOIL

By Jin-Chun Chai,1 Shui-Long Shen,2 Norihiko Miura,3 and Dennes T. Bergado4

ABSTRACT: On a macroscale, the effect of installing prefabricated vertical drains (PVDs) in a subsoil is to increase the mass hydraulic conductivity of the subsoil in the vertical direction. Based on this concept, a simple method for modeling PVD improved subsoils is proposed, in which an equivalent vertical hydraulic conductivity $k_{ve}$ for the PVD improved subsoil is explicitly derived. With the proposed simple method, analysis of PVD improved subsoil is the same as that of the unimproved case. The theoretical verification of the simple method was made under 1D condition. The calculated average degree of consolidation and excess pore pressure distribution in the vertical direction using the simple method are compared with existing theoretical solutions (combination of Terzaghi’s consolidation theory and Hansbo’s solution for PVD consolidation). It has been proved theoretically that, in terms of average degree of consolidation, in the case of one layer and ignoring the vertical drainage of natural subsoil, the maximum error of the proposed method is 5%. The multilayer case was analyzed by FEM method, and the proposed simple method is compared with that of using 1D drainage elements. Then, 2D finite-element analyses were conducted for three case histories of embankments on PVD improved subsoils. One case is discussed in detail. The analyses using both the simple method and 1D drainage elements, were conducted. It is shown that for all three cases, the simple method yielded results as good as those using 1D drainage elements.

INTRODUCTION

The commonly used consolidation theory for designing prefabricated vertical drain (PVD) improvement is the 1D unit cell solution [e.g., Barron (1948) and Hansbo (1981)]. Because the solutions considering both vertical and radial drainage are complicated, the solutions used in practice are those that ignore the effect of vertical drainage, such as Barron’s and Hansbo’s solutions. However, in some cases, the vertical drainage has a considerable effect on the degree of consolidation of PVD improved subsoil. Furthermore, for most actual cases, the subsoil is not uniform, and also the deformation of PVD improved subsoil is not always in 1D conditions. Thus, numerical methods, typically FEM, are required for designing PVD improvement or predicting the behavior of PVD improved subsoil. Existing methods for modeling PVD improved subsoil either employ a 1D drainage element (Hird et al. 1992; Chai et al. 1995) or adopt a special formulation of FEM program (Sekiguchi et al. 1986). Because of these particularities, conducting the FEM analysis for PVD improved subsoil is very time-consuming (Olson 1998) and inconvenient. Consequently, a simple way for analyzing the PVD improved subsoil is desirable.

A simple approximate method for analyzing PVD improved subsoil is proposed in this study, which analyzes the improved subsoil in the same way as that for the unimproved case. Verification of the simple method was made by comparing the calculated average degree of consolidation and excess pore pressure distribution, using the proposed method with theoretical results under the 1D condition. The factors affecting the accuracy of the proposed method are discussed. Finally, the proposed method is applied to analyze three test embankments on PVD improved subsoil at different soft clay deposits.

SIMPLE METHOD FOR ESTIMATING $k_{ve}$

From a macro point of view, vertical drains increase the mass hydraulic conductivity of subsoil in the vertical direction. Therefore, it is logical to try to establish a value of vertical hydraulic conductivity, which approximately represents both the effect of vertical drainage of natural subsoil and the effect of radial drainage due to existence of PVD. Under this condition, the PVD improved subsoil can be analyzed in the same way as the unimproved case. This equivalent value of vertical hydraulic conductivity $k_{ve}$ is derived based on the equal average degree of consolidation under the 1D condition. Carrillo’s theoretical solution (1942) is used to combine the vertical and radial drainage effects

$$U_{ve} = 1 - (1 - U_r)(1 - U_t)$$

where $U_{ve}$ = average degree of consolidation of PVD improved subsoil; $U_r$ = average degree of consolidation due to radial drainage; and $U_t$ = average degree of consolidation due to vertical drainage. The value of $U_r$ is calculated by Hansbo’s solution (1981), which was derived based on equal vertical strain assumption and neglected the vertical drainage of natural subsoil

$$U_r = 1 - \exp \left( - \frac{8}{\mu} T_h \right)$$

where $T_h$ = time factor = $C_v t / D^2_r$, in which $C_v$ = coefficient of consolidation in horizontal direction, $D_r$ = diameter of unit cell, and $t$ = time. The value of $\mu$ can be expressed

$$\mu = \ln \frac{n s^3}{s k_s} \ln(s) - \frac{3}{4} + \pi \frac{2l k_h}{3g_v}$$

where $n = D_s / d_s$ ($d_s$ = diameter of drain); $s = d_r / d_s$ ($d_r$ = diameter of smear zone); $k_h$ and $k_s$ = horizontal hydraulic conductivities of the natural soil and smear zone, respectively; $l$ = drainage length; and $g_v$ = discharge capacity of PVD.

To obtain a simple expression for the equivalent vertical hydraulic conductivity $k_{ve}$, an approximation of Terzaghi’s solution for the average degree of vertical consolidation is proposed.
\[ U_v = 1 - \exp(-C_d T_v) \]  

(4)

where \( T_v \) = time factor for vertical consolidation = \( C_v t/H^2 \), in which \( C_v \) = coefficient of consolidation in the vertical direction and \( H \) = vertical drainage length (in this study \( H = l \)); and \( C_d \) = constant. To determine the value of \( C_d \), the following factors are considered:

- Eq. (4) is used to obtain the \( k_v \) value only. After \( k_v \) is determined, the degree of consolidation of PVD improved subsoil is calculated using Terzaghi’s theory for the 1D case and Biot’s theory (1941) for 2D or 3D problems.
- As shown in Fig. 1, under the condition that, at \( U_v = 50\% \), (4) can yield the same result as that of Terzaghi’s theory, \( C_d \) can be assumed as 3.54. In this case, for \( U_v < 50\% \), (4) underestimates, and, for \( U_v > 50\% \), it overestimates the average degree of consolidation with a maximum error of <10\%. Therefore, for converting the PVD effect into \( k_v \), if one ignores the effect of vertical drainage of natural subsoil, \( C_d = 3.54 \) is the best value.
- By considering the effects of both vertical drainage of natural subsoil and radial drainage of PVD, the maximum error on the average degree of consolidation can be reduced. However, with \( C_d = 3.54 \), the range of underestimation will be more than half.
- The best value of \( C_d \) is a function of relative importance of vertical and radial drainage, which varies from case to case. In this study, \( C_d = 3.2 \) is proposed. With \( C_d = 3.2 \), in terms of average degree of consolidation, for the cases studied here, the maximum error of the proposed method is within 5\%. In this case, ignoring the vertical drainage of natural subsoil, the maximum error between (4) and Terzaghi’s solution is 10\%. Because (4) takes the same form as (2) (Hansbo’s solution), the maximum error between the proposed method (converting PVD effect to an equivalent vertical hydraulic conductivity) and Hansbo’s solution (PVD consolidation) is 10\% also. Then, the equivalent vertical hydraulic conductivity \( k_v \) can be expressed

\[ k_v = \left( 1 + \frac{2.5l^2 k_v}{\mu D^2 k_h} \right) k_v \]  

(5)

where \( k_v \) = hydraulic conductivity in the vertical direction. Other parameters are defined previously.

For multilayer conditions, when using (5) to calculate the \( k_v \) value of each layer, it is simply assumed that the drainage length \( l \) is the same as the total thickness \( H \) of the PVD-improved zone for one-way drainage (\( l = H \)) and \( l = H/2 \) for two-way drainage. One-way or two-way drainage conditions should be applied based on the condition of the PVD improved zone only. In the case where a clay deposit is in two-way drainage conditions but the PVDs are not penetrated through the whole thickness of the layer, then for the PVD-improved zone, one-way drainage condition should be applied, as illustrated in Fig. 2.

It needs to be emphasized that the 1D condition is used to obtain the \( k_v \) value, which does not mean that the proposed method can only be used in 1D analysis. A PVD-improved zone with a vertical hydraulic conductivity of \( k_v \) and horizontal value of \( k_h \) can be analyzed in 1D, 2D, or 3D, depending on the requirements.

**VERIFICATION OF PROPOSED METHOD**

The proposed method has been tested by comparing the average degree of consolidation and the excess pore pressure distribution in the vertical direction with those calculated by the combination of Terzaghi’s 1D consolidation theory and Hansbo’s solution for radial consolidation (Hansbo 1981) (referred to as the theoretical solution hereinafter) for a uniform subsoil case. For a multilayer subsoil condition, the analyses were conducted by FEM and the results of the proposed method have been compared with those using drainage elements. The comparisons are made under the 1D condition.

**FIG. 1.** Effect of \( C_d \) on Degree of Consolidation

**FIG. 2.** One- or Two-way Drainage Conditions
### TABLE 1. Assumed Subsoil and Drain Parameters

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</tr>
<tr>
<td>$n$</td>
<td>$D_w$</td>
</tr>
<tr>
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<td>$(m)$</td>
</tr>
<tr>
<td>$k_h$</td>
<td>$d_s$</td>
</tr>
<tr>
<td>$(10^{-8}\text{ m/s})$</td>
<td>$(m)$</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>$k_h/k_s$</td>
</tr>
<tr>
<td>$(\text{kN/m}^3)$</td>
<td></td>
</tr>
<tr>
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<td>5.0</td>
</tr>
<tr>
<td>18.0</td>
<td>100</td>
</tr>
</tbody>
</table>

### Uniform Subsoil Case

The assumed 1D conditions are illustrated in Fig. 3 (left side). The assumed subsoil and drain parameters are listed in Table 1. It needs to be mentioned that the absolute values of the involved parameters have no effect on the tendency of comparison. With these conditions, (5) yields an equivalent vertical hydraulic conductivity of $11.94 k_v$.

The comparison of the average degree of consolidation is shown in Fig. 4. It can be seen that the maximum difference between the simple approximate method and the theoretical solution is about 5%. The excess pore pressure distribution in the vertical direction is compared in Fig. 5 at the average degree of consolidation of 50%. The proposed method indicates a faster excess pore pressure dissipation at the upper part and a slower dissipation at the lower part of the subsoil. This is because the simple approximate method converts effects of both vertical and radial drainages into vertical drainage. The radial drainage condition tends to result in a close to uniform excess pore pressure distribution with depth if the well resistance of PVD is not significant.

### Multilayer Subsoil

The uniform subsoil condition is rarely encountered in engineering practice. A common situation is that there is a weathered crust with higher hydraulic conductivity underlain by soft layers. A two-layer subsoil model is also indicated in Fig. 3 (right side). The values of hydraulic conductivities of Layer 1 are $k_{v1} = 2 \times 10^{-8} \text{ m/s}$ and $k_{h1} = 2k_{v1}$, and the elastic modulus of Layer 1 $E_1$ is 8,000 kPa. Other soil parameters for Layers 1 and 2 and the drain parameters are the same as those tabulated in Table 1. Consequently, the coefficient of consolidation of Layer 1 $C_{v1}$ is four times that of Layer 2 $C_{v2}$.

For the multilayer subsoil, a 1D rigorous closed-form solution is not available and FEM was used for calculation. The base FEM solutions used for comparison with FEM solutions using the simple approximate method are calculated by a method in which the radial drainage due to PVD is modeled by 1D drainage elements (Chai et al. 1995).

Fig. 6 compares the average degree of consolidation, which shows that the maximum difference is about 5%. In Fig. 6, the time factor $T_{ve}$ was calculated by using the weighted (by the layer thickness) average value of the coefficient of consolidation in the vertical direction. The distribution of excess pore pressure with depth is given in Fig. 7. Because of the higher hydraulic conductivity of Layer 1, the difference in Fig. 7 is smaller than that of the uniform case (Fig. 5).

### Discussion

Regarding the accuracy of the simple approximate method, a theoretical conclusion is that the error on the average degree of consolidation is <10% in the 1D condition. The above comparisons are made for the assumed cases, and the relative er-
The error is affected by the following factors:

- Relative significance of vertical to radial drainage effects — The smaller the effect of vertical drainage of natural subsoil, the larger the error will be.
- Effect of well resistance — The effect of well resistance tends to be given in a nonuniform excess pore pressure distribution with depth. Therefore, the larger the well resistance, the smaller the error will be. Field evidence showed that, because of the clogging effect, the well resistance of PVD is quite high (or the discharge capacity of PVD is small) (Chai and Miura 1999).
- Vertical drainage length — The larger the vertical drainage length \( l \) and the smaller the drain spacing \( S \) (larger \( l/S \)), the larger the error will be.
- The uniformity of subsoil — As shown in the above comparisons, for multilayer and topsoil with a higher hydraulic conductivity case, the error of the simple approximate method can be reduced. This is because the higher hydraulic conductivity of the upper layer increases the effect of vertical drainage. Actually, for most natural deposits, the hydraulic conductivity of the top layer is higher than that of the underlying layer.

**FIG. 6.** Comparison of Average Degree of Consolidation (Two-Layer Case)

**FIG. 7.** Comparison of Excess Pore Pressure Distribution (Two-Layer Case)

**APPLICATION OF PROPOSED METHOD TO CASE HISTORIES**

The proposed method was applied to analyze three case histories of embankment on PVD improved subsoil under the plane strain condition (2D), in which only one case is presented in detail and the other two are presented briefly. Case 1 is a test embankment at Hangzhou-Ningbo (HN) Expressway in eastern China, which is discussed in detail. The other two briefly discussed cases consist of test embankments at Saga Airport, Japan and Nong Ngu Hao, Thailand. The analyses using drainage elements (Chai et al. 1995) to represent the effect of the PVD were also conducted. It is worth mentioning that, using the concept of equivalent hydraulic conductivity proposed in this study, FEM analysis can be conducted using standard programs. The results obtained using the proposed simple method are compared with those obtained using drainage elements in terms of settlement curves, excess pore pressures, and lateral displacement profiles. Both FEM analysis results are compared to measured data also.

**Modeling Methods**

In the following analyses, the behavior of clay layers was modeled by a modified Cam-clay model (Roscoe and Burland 1968). All soil parameters of clay layers were estimated based on test results, except for Poisson’s ratio and hydraulic conductivity of subsoil. The hydraulic conductivity was back-calculated from the test embankments on natural subsoil at the same site or near the test sites (Chai and Miura 1999; Shen et al. 2000). During consolidation, the hydraulic conductivity is varied with the void ratio according to Taylor’s equation (1948). The parameter \( C_1 \) in Taylor’s equation was taken as 0.4\( e_o \), to 0.5\( e_o \) (\( e_o \) is initial void ratio). Sand layers and embankment fill materials were treated as elastic materials, and the Young’s modulus \( E \) and Poisson’s ratio \( v \) were assumed empirically.

The parameters related to the behavior of PVD were evaluated by the methods proposed by Chai and Miura (1999). The methods can be briefly described as follows:

- Discharge capacity of PVD — The discharge capacity \( q_w \) of PVD needs to be determined by a long-term confined in-clay test. If there are no test data available, based on the writers’ experience, \( q_w = 100 \text{ m}^3/\text{year} \) is suggested for preliminary design.
- Smear-zone diameter — The smear zone diameter \( d_s \) can be estimated

\[
d_s = 3d_m\tag{6}
\]

where \( d_m \) = area equivalent diameter of mandrel for installing PVD.
- The ratio of \( k_h/k_s \) — An equation for evaluating the field value of \( k_h/k_s \) is as follows:

\[
\left( \frac{k_h}{k_s} \right)_f = C_f \cdot \left( \frac{k_h}{k_s} \right)_L
\]

where \( C_f \) = ratio of field hydraulic conductivity \( k_h \) over the corresponding laboratory value \( k_h \). A laboratory test normally underestimates the field hydraulic conductivity, and \( C_f > 1.0 \). The value of \( C_f \) needs to be determined by back-analysis or field and laboratory hydraulic conductivity tests. The value of \( (k_h/k_s) = \) laboratory value of the hydraulic conductivity ratio.

**Test Embankment on Soft Mucky Clay Deposit in Eastern China**

The (HN) expressway is located at the southern coast of Hangzhou Bay, as shown in Fig. 8. It starts from Hangzhou,
the capital of Zhejiang Province, to Ningbo, the biggest harbor city of the same province. The total length of the HN expressway is 145 km, of which about 92 km is passing through a soft clay deposit (in China it is called mucky clay). To get reliable data and experience to guide the design and construction, 12 field full-scale test embankments with a total length of 3.15 km were constructed and investigated (Wang et al. 1998). The PVD improved section, which is analyzed in this study, was one of the test embankments.

Description of Subsoil Condition and Test Embankment

The generalized soil profile and soil properties of the soft deposit at the test site are shown in Fig. 9. The thickness of the soft layers is about 23 m. A thin weathered crust (TC) has a thickness of 1–1.5 m overlying a silty clay (SC1) approximately 4 m thick. The third layer is a very soft mucky clay (MC) with a thickness of approximately 10 m. The fourth layer is a soft clay called mucky-silty clay (MSC) and has a thickness of approximately 4 m. The fifth layer is a medium-to-stiff silty clay layer (SC2) 3–5 m thick. Below the fifth layer, it is a clayey sand layer. The subsoils are in lightly overconsolidated to normally consolidated states. The overconsolidation ratio at the top crust is about 5. The soft silty clay and mucky clay have a water content greater than the corresponding liquid limit, low hydraulic conductivity, and lower shear strength. The groundwater level is about 1.5 m below the ground surface.

Fig. 10 shows the geometry of the embankment on PVD improved subsoil and the main instrumentation points. A 0.5-m-thick sand mat was placed on top of the soft ground. Decomposed granite was laid out and compacted in layers to a unit weight of about 20 kN/m³. The height of the embankment was 5.88 m. PVDs were installed to 19-m depth in a triangular pattern with a spacing of 1.5 m.

FEM Modeling and Model Parameters

The modeled subsoil was 29 m below the ground surface, and 120 m horizontally from the embankment centerline (120 m is about four times the embankment half-width, which is considered enough to substantially reduce the boundary effects). The displacement boundary conditions were (1) at bottom, both vertical and horizontal displacements were fixed; and (2) for left (under the embankment centerline) and right (away from the centerline) vertical boundaries, the horizontal displacement was fixed but vertical movement was allowed. The adopted drainage boundary conditions were that the ground surface and bottom line (sand layer) were drained. The left and right boundaries were undrained, but for the right boundary (away from the centerline), at the locations of sand layers, they were drained.

Fig. 11 shows the mesh for this case. The element used is an eight-node quadrilateral element. In the analysis with drainage elements, the 1D drainage elements were set one at every other vertical line in the PVD improved zone, as indicated in Fig. 11. The determined model parameters for the subsoil are listed in Table 2. Based on the back-analysis of an embankment on natural subsoil at the same site, the value of \( C_f \) is 6 (Shen et al. 2000). The parameters related to PVD behavior are listed in Table 3, which were determined by using the method proposed by Chai and Miura (1999). The embankment construction history was simulated closely.

Comparing Simulated Results

Fig. 12 compares the settlement curves. The proposed method resulted in a slightly slower consolidation rate compared with that of using drainage elements. The maximum difference on settlement is <5% (or 0.1 m). Fig. 13 compares the excess pore pressures, and it indicates that the proposed method yielded slightly higher excess pore pressures than those using drainage elements, which is consistent with the results for settlement. When comparing with the measured data, the FEM analyses resulted in a faster excess pore pressure dissipation rate. For lateral displacement, the two methods yielded an almost identical result (Fig. 14). From these comparisons, it can be observed that, practically, the proposed
method can give results as good as that of using drainage elements.

**Limitation on Predicting Excess Pore Pressure**

In the field, a single PVD works in a close to axisymmetric drainage condition (3D). In 2D analysis, the equivalence is obtained by the average degree of consolidation. As a result, local pore pressure distribution (horizontal direction) is different between 3D and 2D cases. In 2D analysis using drainage elements, it tends to overpredict the pore pressure at a point between two PVDs (the point considered) (Chai et al. 1995) and it cannot be the reason for underprediction. Although the reason for the poor prediction of excess pore pressure is not clear yet, the following points may provide a partial explanation of why the predicted excess pore pressure dissipation rate is faster:

- Possible errors in the measured data—For some embankments on soft subsoils [such as on Muar clay, Malaysia (Indraratna et al. 1994); Porto Tolle, Italy (Hird et al. 1995); Saga Airport, Japan (Chai and Miura 1999); and the HN Expressway, China (Shen et al. 2000)], with progressing of settlement, there were little excess pore pressure dissipations. This phenomenon has been observed for embankments both on natural subsoil and on PVD improved subsoil. One of the possible reasons is the clogging of the piezometer with elapsed time, wherein a certain amount of excess pore pressures are “locked in” the

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<table>
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<tr>
<th>Layer</th>
<th>E (kPa)</th>
<th>ν</th>
<th>k</th>
<th>λ</th>
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<th>e₀</th>
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<th>kₕ (10⁻⁸ m/s)</th>
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**FIG. 10.** Cross Section of Embankment and Field Instrumentation

**FIG. 11.** Finite-Element Mesh in Analysis

**FIG. 12.** Comparison of Settlement Curves

**TABLE 3.** Parameters Related to Behavior of PVD

<table>
<thead>
<tr>
<th>Item</th>
<th>Symbol</th>
<th>Values</th>
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<tr>
<td>Drain diameter (mm)</td>
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<td>Unit cell diameter (m)</td>
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<td>Smear zone diameter (mm)</td>
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<tr>
<td>Ratio of field kₑ value over laboratory value</td>
<td>Cₑ</td>
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</tr>
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</table>

(Chai et al. 1995); (Indraratna et al. 1994); (Hird et al. 1995); (Chai and Miura 1999); (Shen et al. 2000)); with progressing of settlement, there were little excess pore pressure dissipations. This phenomenon has been observed for embankments both on natural subsoil and on PVD improved subsoil. One of the possible reasons is the clogging of the piezometer with elapsed time, wherein a certain amount of excess pore pressures are “locked in” the
piezometer. For test embankments on Muar clay, Malaysia, some piezometers showed no excess pore pressure dissipation during the consolidation period. Upon installation of new piezometers at adjacent points, close to the predicted values of excess pore pressures were measured [Malaysia Highway Authority (MHA) 1989].

- Discharge capacity $q_w$ reduction of PVD with elapsed time—In FEM analysis, a constant $q_w$ was adopted whereas a laboratory long-term discharge capacity test with clay confinement showed that $q_w$ was significantly reduced with elapsed time (Chai and Miura 1999). The constant $q_w$ tends to predict a faster excess pore pressure dissipation rate at a later stage of consolidation. However, at present, it is difficult to formulate a general expression for $q_w$ variation with time.

- Limitation of soil model—A modified Cam-clay model (Roscoe and Burland 1968) was used to model the behavior of soft clay in this study. The model employed a volumetric strain hardening function using a linear $e$-$\ln p'$ relationship ($e$ is the void ratio and $p'$ is the consolidation stress). In this case, the coefficient of consolidation $C_v$ (or $C_h$) can be expressed as follows:

$$C_v = \frac{(1 + e)p' \cdot k}{\lambda \cdot \gamma_w}$$  \hspace{1cm} (8)$$

where $k$ = hydraulic conductivity of soil; $\gamma_w$ = unit weight of water; and $\lambda$ = slope of virgin compression curve in $e$-$\ln p'$ plot.

For an Ariake clay sample, considering $k$ both as constant and as varying with the void ratio (Taylor 1948), the calculated $C_v$ values using (8) are compared with measurements in Fig. 15 (Chai 2001). Based on test results, an initial hydraulic conductivity $k_o = 7.5 \times 10^{-7}$ m/s, $\lambda = 0.544$, and initial void ratio $e_o = 3.4$ were adopted in the calculation. The measurement shows that, when $p' > 100$ kPa, there was little increase of $C_v$ with increase of $p'$. But (8) gives a close to linear increase of $C_v$ with $p'$ (double logarithm scale). Even for $k$ varying with the void ratio case, (8) gives a higher increase rate of $C_v$ values than the measured data. Faster increase of the $C_v$ value will yield a faster excess pore pressure dissipation rate. Although further research is needed to clarify this point, it is possible that the adopted model may not exactly represent the behavior of the subsoil.

**Verification by Other Field Cases**

**Test Embankment on Soft Ariake Clay Deposit at Saga Airport, Japan**

The test embankments at Saga Airport were reported by Chai and Miura (1999), and a brief description is given here. The site is located 13 km south of Saga City on a reclaimed land close to the Ariake Sea. The deposit mainly consists of soft and highly compressible Ariake clay. The compression index $C_s$ is 1.0–2.0 and natural water content is 80–120%.

The soft deposit is about 25 m thick, consisting of three alluvial clay layers and two thin alluvial sand layers. The top crust is about 1.0 m thick and is in an overconsolidated state with an overconsolidation ratio of about 4.0. Other clay layers
are in a normally to lightly overconsolidated state. The groundwater level is about 1.0 m below the ground surface.

The test embankments were constructed on both natural and PVD improved subsoils to investigate the effect of PVD improvement. The embankment fill thickness was 3.5 m (including a 0.5-m sand mat) and filling speed was about 0.03 m/day. The base and top dimensions were 71 × 71 m and 25 × 25 m, respectively. The PVDs were installed to around 25 m depth with a spacing of 1.5 m (square pattern). The improved zone was under the center of the embankment with an area of 45 × 45 m.

Test Embankment on Bangkok Clay Deposit, Thailand

The test embankments on PVD improved subsoil at Nong Ngu Hao, Thailand, were reported by Bergado et al. (1997). The embankment analysis had base dimensions of 54 × 54 m. After the fill thickness reached 1.5 m, a 7-m berm was added at four sides and the embankment was built to a final fill thickness of 4.2 m. The average filling speed was about 0.02 m/day. The top dimension was 14.8 × 14.8 m. At the test site, the soft clay deposit extends 12–15 m. For soft layers, the compression index $C_v$ is 0.8–2.0 and the water content is 80–100%. The soft deposit is in a normally consolidated state except for the surface weathered crust. The groundwater level is about 1.0 m below the ground surface. The PVD improved zone had a base width of 40 m. The PVDs were installed in a square pattern to 12 m deep, with a spacing of 1.5 m. In this site, the hydraulic conductivity of the subsoil below 12-m depths was low and one-way drainage was adopted for the PVD-improved zone.

For the above two cases, although the embankments were in a square pattern, on the centerline, a plain strain condition was assumed and analyzed by 2D FEM.

The FEM analysis results of settlements, excess pore pressures, and lateral displacements using the proposed method as well as the drainage elements method were compared together with field data. The tendencies are similar to those of Case 1, the test embankment at the HN expressway, China.

The above three cases represent a wide range of natural clay deposits. The results of the proposed method compared well with those of a more sophisticated method using drainage elements. Considering the potential errors involved in determining the soil parameters, the error introduced by the simple method is not significant. The proposed method provides an easy way to analyze and predict the behavior of PVD-improved subsoil.

**CONCLUSIONS**

- A simple approximate method for analyzing PVD improved subsoils is proposed in this paper. It represents the effect of vertical hydraulic conductivity of natural subsoil and the effect of radial drainage due to PVD, by using an equivalent vertical hydraulic conductivity $k_{eq}$. With the proposed method, the analysis of PVD-improved subsoil becomes the same as that for the unimproved case.
- It has been proved theoretically that, in terms of the average degree of consolidation, in the case of one layer and ignoring the vertical drainage of natural subsoil, the maximum error of the proposed method is 10% compared with Hansbo’s solution for PVD consolidation. For the cases of one layer or multilayers and considering both vertical and radial drainages with the parameters adopted here, the maximum error of the proposed method is 5%. The multilayer cases were analyzed by FEM, and the FEM solutions using the proposed method are compared with those using 1D drainage elements. The main factor influencing the accuracy of the proposed method is the relative importance of vertical-to-radial drainage. The shorter the vertical drainage length, the higher the vertical hydraulic conductivity, and the larger the well resistance of PVD, the smaller the error will be.
- The method was applied to analyze test embankments on PVD-improved subsoils in China, Japan, and Thailand. All these cases, compared with the results of using the drainage elements method as well as measured data, indicate that the proposed method yielded acceptable results in terms of settlement, excess pore pressure, and lateral displacement. It is recommended that the proposed simple approximate method is a useful tool for engineering practice.

**REFERENCES**


