Design procedure for vertical drains considering a linear variation of lateral permeability within the smear zone

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A DESIGN PROCEDURE FOR VERTICAL DRAINS CONSIDERING A LINEAR VARIATION OF LATERAL PERMEABILITY WITHIN THE SMEAR ZONE

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Abstract

A system of vertical drains with surcharge preloading is an effective method for promoting radial drainage and accelerated soil consolidation. This study presents a procedure for the design of vertical drains significantly extending the previous technique proposed by the Authors to include; (i) a linear reduction of lateral permeability in the smear zone, (ii) the effect of overlapping smear zones in a closely spaced drain network, and (iii) the gain in undrained shear strength due to consolidation. Design examples are provided for both single stage and multi-stage embankment construction demonstrating the convenient use of the proposed solutions in practical situations.

Key words: Consolidation, Design charts, Smear zone, Vertical drains.
Introduction

Consolidation via vertical drains can be employed to stabilise soft soil by providing a shorter horizontal drainage length, thereby reducing the consolidation time. The theory of consolidation via radial drainage was initially proposed by Carrillo (1942) and Barron (1948). Subsequently, Yoshikini and Nakanodo (1974), Hansbo (1981) and Onoue (1988) extended these solutions considering the effects of smear and well resistance attributed to vertical drains. However, the smear zone characteristics in the aforementioned studies were simplified adopting a reduced but constant horizontal permeability coefficient within the smear zone. The vertical permeability is considered to remain unchanged for both the smear and undisturbed zones. For prefabricated vertical drains (PVDs) driven by a steel mandrel, the smear zone radius is usually in the range of 2 to 3 times the equivalent mandrel radius, and the ratio of undisturbed horizontal soil permeability to that in the smear zone varies from 1 to 8 (Bo et al., 2003; Indraratna and Redana, 2000). However, as observed from large scale laboratory tests conducted by Onoue et al. (1991), Madhav et al. (1993), Indraratna and Redana (1998) and Sharma and Xiao (2000), the horizontal permeability ($k_h$) decreases substantially in a non-linear manner towards the drain within the smear zone. Indraratna and Redana (1998) showed that the horizontal permeability ($k_h$) can be reduced to be the same as vertical permeability ($k_v$) (complete remoulding) very close to the drain, and while the ratio of $k_h/k_v$ decreases sharply as the drain is approached, the vertical permeability on its own remains relatively constant along the radial direction. The same observations have been later confirmed by Sathananthan and Indraratna (2006). Therefore, the effect of disturbance on vertical permeability has not been considered in the analysis. Limited analytical solutions considering different forms of nonlinear variation of horizontal permeability have been cited (e.g. Basu et al. 2006, Walker and Indraratna 2006; Walker and Indraratna 2007). Walker and Indraratna (2007) showed that the difference in degree of consolidation obtained
between linear and nonlinear variation of horizontal permeability are insignificant as long as the undisturbed horizontal coefficient of permeability and the minimum horizontal coefficient of permeability within the smear zone approach the same value. Walker and Indraratna (2007) while capturing the role of reduced horizontal permeability distribution, showed that the overlapping smear zone due to the reduction of drain spacing can further affect the drain performance.

In design, a number of iterations have to be performed before obtaining the appropriate drain spacing. Rujikiatkamjorn and Indraratna (2007) proposed a new design method to avoid the cumbersome trial and error approach for determining the drain spacing. However, in this approach, the effects of quasi-linear variation of horizontal permeability in the smear zone and the possibility of overlapping smear zones at close drain spacing were not considered. In this paper, the design approach proposed by Rujikiatkamjorn and Indraratna (2007) have been significantly extended to consider the above effects, as well as to predict the increase in undrained shear strength during multi-stage embankment construction. Illustrative design examples are provided to the benefit of the practitioners when applying to real-life situations.

**Theoretical Background**

Vertical drains, installed in a square or triangular pattern, are usually modelled analytically by considering an equivalent axisymmetric system. Pore water flows radially from a soil cylinder to a single central vertical drain with simplified boundary conditions. A detailed mathematical solution for radial consolidation considering both linear and constant smear zone permeability has been derived by Walker and Indraratna (2007). Only a summary of the theoretical background is presented below for the benefit of the readers, thus making this article stand alone.
Figure 1 shows a unit cell with an external diameter $d_e$ with vertical drain diameter $d_w$.

According to Rujikiatkamjorn and Indraratna (2007), the average degree of consolidation, $U_i$, considering both vertical and horizontal drainage at time $t$ is:

$$
U_i = 1 - \sum_{m=1}^{\infty} \frac{8}{(2m+1)^3 \pi^2} \exp \left( - \left[ \left( \frac{2m+1}{2} \right)^2 \frac{1}{c_{mh} L^2} + \frac{8}{\mu} \right] T_h \right)
$$

where, the relevant dimensionless parameters are given by:

$$
c_{mh} = c_h / c_v = k_h / k_v
$$

$$
L = l / d_e
$$

$$
T_h = c_h t / d_e^2
$$

$$
c_h = k_h / m_v \gamma_w \text{ and } c_v = k_v / m_v \gamma_w
$$

where, $n = d_e / d_w$, $s = d_s / d_w$, $\kappa = k_h / k_s$, $k_h =$ undisturbed horizontal coefficient of permeability, $k_s =$ minimum horizontal coefficient of permeability in the disturbed zone, $\kappa =$ ratio of undisturbed permeability to permeability at the drain/soil interface, $l =$ drain length, $d_e =$ the diameter of soil cylinder dewatered by a drain, $d_s =$ the diameter of the smear zone, $d_w =$ the equivalent diameter of the drain, $\gamma_w =$ the unit weight of water and $m_v =$ the coefficient of soil compressibility.

For most modern PVDs where the discharge capacity exceeds 150 m$^3$/year, the well resistance can be neglected (Indraratna and Redana 2000). Under these circumstances, $\mu$ for a linear reduction in horizontal permeability towards the drain with constant soil compressibility assumption (Figure 1b) is given by (Walker and Indraratna 2007):

$$
\mu_{L} = \ln \left( \frac{n}{s_L} \right) - \frac{3}{4} + \frac{s_L (s_L - 1)}{s_L - \kappa_L} \ln \left( \frac{s_L}{\kappa_L} \right)
$$

For the case when $s_L = \kappa_L$ the $\mu$ parameter is:
If the smear zones are overlapping, $\mu$ can be calculated based on:

$$\mu_X = \begin{cases} 
\mu_L(n, s_L, \kappa_L) & n \geq s_L \\
\frac{\kappa_L}{\kappa_{LX}} \mu_L(n, s_{LX}, \kappa_{LX}) & 2n - s_L \geq 1, \text{ and, } s_L > n \\
\frac{\kappa_L}{\kappa_{LX}} \mu_L(n) & 2n - s_L < 1 
\end{cases}$$

$$\kappa_{LX} = 1 + \frac{\kappa_L - 1}{s_L - 1}(s_{LX} - 1)$$

$$s_{LX} = 2n - s_L$$

$$\mu_i = \ln(n) - 0.75$$

For a smear zone with constant horizontal permeability (Figure 1a), value of $\mu$ is given by Hansbo (1981):

$$\mu_C = \ln(n/s_C) + (\kappa_C)\ln(s_C) - 0.75$$

In the preceding, the subscripts $C$ and $L$ represent constant horizontal permeability and linear variation of horizontal permeability in the smear zone, respectively.

### Undrained Shear Strength Gain due to Consolidation

Bjerrum (1972) showed that the undrained shear strength of soft soil can be predicted using the undrained shear strength gain ratio, $\frac{\Delta s_u}{\Delta \sigma_v'}$, where, $\Delta s_u$ = gain in undrained shear strength and $\Delta \sigma_v'$ = increase in effective vertical stress. Subsequently, this ratio was incorporated in some designs procedures for soft soil stability, e.g. Stress History and Normalised Soil Engineering Properties, i.e. SHANSEep method (Ladd and Foott, 1974). For vertical drain design, it is
assumed that all sub-soil layers are normally consolidated and subjected to 1D consolidation. An increase in the undrained shear strength ($\Delta s_u$) can be estimated as follows:

\[ \frac{\Delta s_u}{\Delta \sigma_v} = \bar{\alpha} \]

where, $\bar{\alpha}$ is almost constant for a given normally consolidated soil (Table 1). Mesri (1989) and Wang et al. (2008) among others has provided extensive discussions on the relatively constant value of $\bar{\alpha}$ for a variety of soft soils.

An increase in the effective vertical stress ($\Delta \sigma_v'$) due to embankment loading can be determined based on elastic solution (Poulos and Davis, 1974), which can be expressed by:

\[ \Delta \sigma_v' = q_{\max} I_q U_t \]

It can be seen that the stress increments vary from one location to another beneath the embankment. Therefore, the influence factor ($I_q$) would be determined according to the location (Fig. 2), by the following equation:

\[ I_q = \frac{1}{\pi} \left( \theta + \frac{x\phi}{a} - \frac{y}{R_t^2} (x-b) \right) \]

**Design Procedures for Linear Variation of Horizontal Permeability in the Smear Zone**

Most design procedures for vertical drains use horizontal time factor ($T_h$) vs. degree of consolidation curves ($U_h$) to determine the drain spacing ($S$) (e.g. Hansbo 1981). Several design procedures to directly determine the drain spacing have been developed, (e.g. Zhu and Yin 2001; Rujikiatkamjorn and Indraratna 2007) Usually, a number of calculations have to be reiterated to obtain essential parameters such as $n$. In practice, the commercially available shapes and dimensions of prefabricated vertical drains (PVDs) are limited in choice, hence, Rujikiatkamjorn and Indraratna (2007) have established the design charts for constant horizontal permeability in the smear zone using the equivalent drain diameter ($d_e$) as a known variable, in order to
determine the drain spacing \((d_e \text{ or } S)\). A similar procedure will be used to develop the design curves for linear lateral permeability reduction in the smear zone.

Rearranging Equation (1) gives:

\[
\gamma = -\frac{8\tau'_h}{\ln \left( \frac{1-U_L}{U_*} \right)}
\]

where,

\[
\tau'_h = c_h t / d_w^2
\]

\[
u^* = \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp \left( -\left( \frac{2m+1}{2} \right)^2 T_v \right)
\]  \(\text{Figure 3}\)

\[
T_v = c_v t / l^2
\]

If the reduced horizontal permeability in the smear zone is constant, Rujikiatkamjorn and Indraratna (2007) have shown that \(\gamma = n^2 \mu = n^2 \left[ \ln (n) + \xi - 0.75 \right]\) can be rearranged as:

\[
n = \exp (\alpha \ln \gamma + \beta);
\]

where,

\[
\xi_c = (\kappa_C - 1) \ln (s_C)
\]  \(\text{Figure 4}\)

\[
\alpha = 0.3938 - 9.505 \times 10^{-4} \xi^{1.5} + 0.03714 \xi^{0.5}
\]  \(\text{Figure 5}\)

\[
\beta = 0.4203 + 1.456 \times 10^{-3} \xi^2 - 0.5233 \xi^{0.5}
\]  \(\text{Figure 5}\)

For linear variation of horizontal permeability in the smear zone with the same \(\alpha\) and \(\beta\) parameters, it can be shown that:

\[
\xi_L = \frac{\kappa_L (s_L - 1)}{s_L - \kappa_L} \ln \left( \frac{s_L}{\kappa_L} \right) - \ln (s_L)
\]  \(\text{Figure 6}\)

Equation (24) cannot be used to calculate \(\xi_L\), when \(s_L = \kappa_L\). The \(\xi\) parameter for the special case is given by:
Once either $\xi_L$ or $\xi_c$ is determined, the parameters $n$, $\alpha$ and $\beta$ can be calculated from Equations (21), (22) and (23), respectively.

**The design steps for embankment with a simplified single stage loading:**

(i) In-situ and soil laboratory testing to obtain relevant soil properties. Determine the depth of installation ($l$), and the time ($t$) required for the consolidation process;
(ii) Determine the required degree of consolidation $U_i$ for surcharge loading only;
(iii) Based on the value of $c_v$, $t$ and $l$, determine $u^*$ using Equation (18) or Fig. 3;
(iv) Choose the size of the prefabricated vertical drains and then calculate the equivalent drain diameter, $d_w$ using the expression $d_w = 2(a+b)/\pi$;
(v) Determine $\tau'_h$ from Equation (17);
(vi) Determine $\gamma$ from Equation (16);
(vii) Determine the diameter and permeability of the smear zone based on the vertical drain installation procedure, the size of mandrel and the type of soil using large-scale laboratory testing (Indraratna and Redana 1998; Bo et al. 2003);
(viii-a) For a smear zone having a constant lateral permeability, calculate $\xi_c$ by Equation (21) or Fig. 4
(viii-b) For a smear zone having a linear lateral permeability variation, calculate $\xi_L$ by Equations (24) and Figure 6, or by Equation (25) and Figure 7;
(ix) Determine $n$ from $\xi$ using Equation (20) and Fig. 5;
(x) If overlapping of smear zones occur ($s_{L} > n$), the required consolidation time has to be recalculated based on Equations (1) and (8);
(xi) Determine the zone of influence from $d_e = nd_w$,
(xii) Calculate the drain spacing \((d)\) from either \(d = d_e/1.05\) or \(d = d_e/1.128\) for a triangular or square grid pattern, respectively and;

(xiii) Undrained shear strength gain is determined based on Equations (13) and (14).

**Worked-Out Example for Single Stage Construction**

The above methodology is illustrated by the following example. The required input parameters are assumed to be:

\(U_t = 90\%\), \(l = 10m\) (one way drainage to the surface), \(d_w = 0.06\) m, \(c_h = 1.0m^2/\text{year}\), \(c_v = 0.5m^2/\text{year}\), \(\kappa_L = 3\), \(s_L = 18\), \(t = 1.2\) year, \(q_{\text{max}} = 80\) kPa, \(\alpha = 0.22\). Ignoring the well resistance, the following calculation demonstrates how the drain spacing \((S)\) is determined.

**Design steps:**

**Step 1.** \(T'_{\tau} = 0.5 \times 1.2/10^2 = 0.006\).

**Step 2.** Determine \(u^*\) using Equation (18) or Fig. 3, Hence, \(u^* = 0.91\).

**Step 3.** \(\tau'_{h} = c_h t / d_w^2 = 1.0 \times 1.2 / 0.06^2 = 333.33\) (i.e. using Equation (17)).

**Step 4.** \(\gamma = -\frac{8 \tau'_{h}}{\ln \left(\frac{1-U_t}{u^*}\right)} = -\frac{8 \times 333.33}{\ln \left(\frac{1-0.9}{0.91}\right)} = 1207.57\) (i.e. using Equation (16)).

**Step 5.** Use Fig. 6 or Equation (24) to get \(\xi_L = 2.60\).

**Step 6.** Use Fig. 5 or Equations (22) and (23) to determine \(\alpha\) and \(\beta\). For this example, \(\alpha = 0.450\) and \(\beta = -0.414\).

**Step 7.** From Equation (20), \(n = \exp(\alpha \ln \gamma + \beta) = \exp(0.450 \times \ln 1207.57 - 0.414) = 16\).

**Step 8.** As \(n < s_L\), overlapping of smear zone occurs, hence, the required consolidation time to achieve the desired degree of consolidation increases. Therefore, the new required
consolidation time based on Equation (1) and (8) is 1.3 years using
\[
\mu_x = \frac{\kappa_L}{\kappa_{LX}} \mu_L \left( n, s_{LX}, \kappa_{LX} \right) \text{ when } 2n - s_L \geq 1, \text{ and, } s_L > n.
\]

**Step 9.** Determine \( d_e \) from \( d_e = n d_w = 16 \times 0.06 = 0.96 \) m.

**Step 10.** Therefore, the drain spacing (\( S \)) = 0.85 m or 0.91 m for square (\( s=d_e/1.13 \)) or triangular pattern (\( s=d_e/1.05 \)), respectively.

**Step 11.** Assuming that at the centreline of the embankment, the average increase in vertical effective stress = \( 80 \times 0.9 = 72 \) kPa. Therefore, the increased undrained shear strength \( \Delta s_u = \bar{\alpha} \Delta \sigma = 0.22 \times 72 = 15.84 \) kPa.

**Design Methodology for a Multi-Staged Embankment Construction**

For multi-stage construction, the height of embankment and the duration of rest period have to be determined to obtain the optimum drain spacing and to ensure embankment stability. During construction, embankment performance should be carefully monitored using field instrumentation such as settlement plates, inclinometers and piezometers, etc. Any gain in strength needs to be confirmed using in-situ vane shear test, CPT or SPT before proceeding to the next stage of loading.

**Design Considerations for Staged Embankment Construction**

The procedures for constructing a staged embankment stabilised with PVDs are as follows:

i. For a given embankment slope and width, the maximum surcharge load (\( q_{\text{max}} \)) may be determined by Bishop’s limit state theory based on undrained shear strength analysis (Ladd, 1991). The factor of safety for embankment slope stability should typically be more than 1.5.
ii. If \( q_{\text{max}} \) is more than the required surcharge load (\( q_{\text{req}} \)), a single stage construction can be carried out following the design steps given in the previous section. If \( q_{\text{max}} < q_{\text{req}} \), then a multi-stage construction is desirable as described below.

iii. For the first stage of construction, the maximum surcharge pre-loading to prevent embankment instability (\( q_{\text{max}} \)) can be applied based on step (i) to maintain minimum safety factor due to undrained slope failure. For a given period of time (\( t \)), the drain spacing can be calculated using the design steps for a single stage loading given in the earlier section. The average degree of consolidation at the end of the first stage (\( U_t \)) should be at least 70\%, as consolidation occurs faster at the beginning (Hartlen and Wolski 1996; Indraratna et al. 2005).

iv. By assuming that the gain in undrained shear strength is attributed to the increase in the vertical effective stress, an increase in the average shear strength at the end of the first stage of construction can be determined by Equations (13) and (14). It is recommended that the soil under embankment loading should be divided into at least 3 zones (i.e. beneath embankment centreline, slope and in the unimproved zone), in order to determine the effective vertical stress increase due to consolidation.

v. The factor of safety for embankment stability of the second construction stage can be calculated using the initial shear strength plus the shear strength increased during the first stage of consolidation. If the safety factor is less than 1.5 for the required surcharge load \( q_{\text{req}} \), Steps iv-v should be repeated for additional stage loading. Figure 8 shows a flow chart summarising the construction methods selected.

**Worked-Out Example for Multi-Stage Embankment Construction**

The example in this section demonstrates the geotechnical design procedure for a multi-stage embankment construction based on the method described above. In this calculation, the design
parameters except for the undrained shear strength, were assumed to be constant through all stages of construction.

Table 2 shows the selected design soil parameters and surcharge fill properties. A 40m wide embankment with a side slope of 2:1 (H:V) is considered. The permanent service load \( q_{\text{req}} \) is assumed to be 70 kPa. Each wick drain is 10 m long, 100 mm wide and 4 mm thick. This gives an equivalent drain diameter \( d_w \) of 0.066m. The values of \( \kappa_L \) and \( s_L \) for this case study are assumed to be 3 and 10, respectively (Bo et al. 2003). PVDs are installed in a square pattern. The groundwater table is assumed to be located at the surface. Effects of secondary consolidation are neglected.

**Design steps:**

**Step 1.** Maximum surcharge \( q_{\text{max}} \) can be determined using the slope stability analysis described earlier (Figure 9). For a safety factor of 1.6, \( q_{\text{max}} \) is 45 kPa (i.e. 2.5m height of surcharge fill having a unit weight of 18 kN/m\(^3\)).

**Step 2.** As \( q_{\text{req}} = 70 \text{ kPa} \), \( q_{\text{max}} < q_{\text{req}} \). Therefore, a multi-stage construction is required. For the first stage, the selected height of the embankment based on the stability analysis is 2.5m (45 kPa). The time required to attain a 70% degree of consolidation for the first stage is about four months. The drain spacing for a square pattern installation is determined using the procedure for a single stage loading described in the previous section. A drain spacing of 1.05m is chosen to be installed in a square pattern.

**Step 3.** The soil under embankment loading is divided into 3 zones (i.e. beneath embankment centreline, slope and in the unimproved zone), in order to determine the effective vertical stress increase due to consolidation, hence the corresponding enhanced undrained shear strength. The increased shear strengths for each zone after consolidation in Stage 1 are shown in Table 3, calculated using Equations (13-15). Using the increased shear strength for each soil zone, the
safety factor obtained for the second stage of construction from Bishop’s method is more than 1.5 (Table 3 and Fig. 10). Therefore, no further staged construction is required.

**Step 4.** It is assumed that the total surcharge load in Stage 2 is the combination of the remaining of excess pore pressure in Stage 1 and the surcharge load applied in Stage 2. The required degree of consolidation for Stage 2 can be calculated based on

\[
U_{stage2} = \frac{U_s \times \text{Total surcharge load} - U_{stage1} \times \text{surcharge load in stage 1}}{(1-U_{stage1}) \times \text{surcharge load in stage 1} + \text{surcharge load in stage 2}}
\]

Based on Eq. (26), a degree of consolidation for stage 2 of 82% is required to achieve 90% overall degree of consolidation. Based on Eqs. (1) and (8), the time required to achieve 82% degree of consolidation in the second stage is 5.5 months.

**Conclusions**

A system of vertical drains is an effective method for accelerating soil consolidation. Design charts provide a convenient practical means for avoiding tedious mathematical iterations or numerical analyses. In this study, design charts published by the Authors (Rujikiatkamjorn and Indraratna, 2007) were further extended to include the linear horizontal permeability variation in the smear zone and the effect of overlapping of adjacent smear zones. The drain design procedures for both single stage and multi-stage construction were established and then demonstrated capturing the gain in undrained shear strength due to consolidation. The proposed design can also be adopted for vacuum-assisted consolidation as the degree of consolidation versus time factor is independent of vacuum pressure ratio (vacuum pressure/surcharge pressure). As expected, when smear zones overlap, the required consolidation time to achieve the desired degree of consolidation increases. The proposed design method is most beneficial to the practitioner as a preliminary tool for design of embankments stabilized by prefabricated vertical drains, where both soil and drain properties are captured in detail.
References


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Table 1  $\alpha$ value for various soils (adopted from Mesri et al. 1989 and Wang et al., 2008)

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<tr>
<th>Friction angle ($\phi'$, degree)</th>
<th>$\bar{\alpha}$</th>
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<tr>
<td>20-25</td>
<td>0.204</td>
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<tr>
<td>25-30</td>
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<td>30-35</td>
<td>0.269</td>
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Table 2 Selected soil parameters for embankment design

<table>
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<th>Parameters</th>
<th>Soil layers</th>
<th>Surcharge fill</th>
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<tr>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Depth (m^2/yr)</td>
<td>0.0-2.0</td>
<td>2.0-8.5</td>
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<tr>
<td>$c_h$</td>
<td>2.8</td>
<td>2.8</td>
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<td>$c_v$ (m^2/yr)</td>
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<td>0.9</td>
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<tr>
<td>$\gamma$ (kN/m^3)</td>
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<td>16</td>
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<td>OCR</td>
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<td>1</td>
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<tr>
<td>$s_{uu}$ (kPa)</td>
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<td>12</td>
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<tr>
<td>$\overline{\alpha}$</td>
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<td>$c'$ (kPa)</td>
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<tr>
<td>$\phi'$ (degrees)</td>
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Table 3 Shear strength development after stage 1 construction

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>$S_{ui}$ (kPa)</th>
<th>$\bar{a}$</th>
<th>$\Delta\sigma'_v$ (kPa)</th>
<th>$S_u$ (after consolidation) (kPa) (Eq. 13)</th>
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<td>(Ref. Fig. 10)</td>
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<td>(Eqs. 14 and 15)</td>
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<td>1 (zone 1, beneath</td>
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<tr>
<td>2 (zone 1)</td>
<td>12</td>
<td>0.22</td>
<td>28.88</td>
<td>18.35</td>
</tr>
<tr>
<td>3 (zone 1)</td>
<td>14</td>
<td>0.22</td>
<td>25.40</td>
<td>19.59</td>
</tr>
<tr>
<td>1 (zone 2, beneath</td>
<td>15</td>
<td>0.22</td>
<td>15.65</td>
<td>18.44</td>
</tr>
<tr>
<td>embankment slope)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 (zone 2)</td>
<td>12</td>
<td>0.22</td>
<td>15.18</td>
<td>15.34</td>
</tr>
<tr>
<td>3 (zone 2)</td>
<td>14</td>
<td>0.22</td>
<td>14.58</td>
<td>17.21</td>
</tr>
</tbody>
</table>

Note: $\Delta\sigma'_v$ was calculated at the mid point of each zone.

For Zone 3 (outside the improvement area), soil shear strength is assumed to be the same as the initial soil shear strength.
Fig. 1. Unit cell of vertical drain, (a) constant horizontal permeability in smear zone and (b) linear horizontal permeability variation in smear zone
Fig. 2. Diagram showing the location and parameters for calculating the factor of influence
Fig. 3. Relationship between $T_v$ and $u^*$ (Rujikiatkamjorn and Indraratna 2007)
Fig. 4. Contour plot of $\xi_C$ for constant horizontal permeability in the smear zone based on Equation (21) (Rujikiatkamjorn and Indraratna 2007)
Fig. 5. Relationships of $\xi$, $\alpha$ and $\beta$ (Rujikiatkamjorn and Indraratna 2007)
Fig. 6. Contour plot of $\xi_L$ for linear horizontal permeability variation in the smear zone based on Equation (24)
Fig. 7. Contour plot of $\xi_L$ for linear horizontal permeability variation in the smear zone based on Equation (25) when $s_L=\kappa_L$
Preliminary Design
Determine initial design parameters for:
- Slope stability of embankment
- Maximum height of embankment
- Required PVDs spacing

Stability, settlement and other constraints satisfactory?

Yes → Perform single stage construction

No → Consider alternative construction methods

- Embankment Geometry Modification:
  - Berms
  - Reinforcement
  - Lightweight materials

- Multi-stage construction

- Ground Improvement beneath embankment:
  - Vacuum preloading
  - Sub-soil replacement

Define suitable combined methods

Stability, settlement and other constraints satisfactory?

Yes → Establish additional requirements for combined methods

Perform cost comparison between selected methods

Select most suitable combined methods

No → Define suitable combined methods

Fig. 8. Procedure for the selection of construction method
Fig. 9. Slope stability analysis for the first stage embankment loading to determine $q_{max}$. 

Soil layer 1 $s_u = 15kPa$

Soil layer 2 $s_u = 12kPa$

Soil layer 3 $s_u = 14kPa$

Factor of Safety = 1.6
Soil layer 1
\[ s_{ui} = 15 \text{kPa} \]
Soil Layer 2
\[ s_{ui} = 12 \text{kPa} \]
Soil Layer 3
\[ s_{ui} = 14 \text{kPa} \]

1\textsuperscript{st} stage
\( q_f = 45 \text{kPa} \)

2\textsuperscript{nd} stage
\( q_f = 25 \text{kPa} \)

Zone 1
\[ s_{ui} = 18.44 \text{kPa} \]
\[ s_{ui} = 15.34 \text{kPa} \]
\[ s_{ui} = 17.21 \text{kPa} \]
\[ s_{ui} = 21.86 \text{kPa} \]
\[ s_{ui} = 18.35 \text{kPa} \]
\[ s_{ui} = 19.59 \text{kPa} \]

Zone 2

Zone 3

Fig. 10. Undrained shear strength of each soil layer for slope stability analysis for the second stage loading to determine \( q_{max} \)