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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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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, Yoshikuni 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_t , considering both vertical and horizontal drainage at time t is:

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

where, the relevant dimensionless parameters are given by:

$$[2] \quad c_{vh} = c_h / c_v = k_h / k_v$$

$$[3] \quad L = l / d_e$$

$$[4] \quad T_h = c_h t / d_e^2$$

$$[5] \quad c_h = k_h / m_v \gamma_w \text{ and } c_v = k_v / m_v \gamma_w$$

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, κ = 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, γ_w = the unit weight of water and m_v = the coefficient of soil compressibility.

For most modern PVDs where the discharge capacity exceeds 150 m³/year, the well resistance can be neglected (Indraratna and Redana 2000). Under these circumstances, μ for a linear reduction in horizontal permeability towards the drain with constant soil compressibility assumption (Figure 1b) is given by (Walker and Indraratna 2007):

$$[6] \quad \mu_L = \ln\left(\frac{n}{s_L}\right) - \frac{3}{4} + \frac{\kappa_L(s_L - 1)}{s_L - \kappa_L} \ln\left(\frac{s_L}{\kappa_L}\right)$$

For the case when $s_L = \kappa_L$ the μ parameter is:

$$[7] \quad \mu_L = \ln\left(\frac{n}{s_L}\right) - 1.75 + s_L$$

If the smear zones are overlapping, μ can be calculated based on:

$$[8] \quad \mu_X = \begin{cases} \mu_L(n, s_L, \kappa_L) & n \geq s_L \\ \frac{\kappa_L}{\kappa_{LX}} \mu_L(n, s_{LX}, \kappa_{LX}) & 2n - s_L \geq 1, \text{ and, } s_L > n \\ \frac{\kappa_L}{\kappa_{LX}} \mu_I(n) & 2n - s_L < 1 \end{cases}$$

$$[9] \quad \kappa_{LX} = 1 + \frac{\kappa_L - 1}{s_L - 1} (s_{LX} - 1)$$

$$[10] \quad s_{LX} = 2n - s_L$$

$$[11] \quad \mu_I = \ln(n) - 0.75$$

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

$$[12] \quad \mu_C = \ln(n/s_C) + (\kappa_C) \ln(s_C) - 0.75$$

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, Δs_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. SHANSEP 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 (Δs_u) can be estimated as follows:

$$[13] \quad \frac{\Delta s_u}{\Delta \sigma_v'} = \bar{\alpha}$$

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:

$$[14] \quad \Delta \sigma_v' = q_{\max} I_q U_t$$

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:

$$[15] \quad I_q = \frac{1}{\pi} \left(\theta + \frac{x\omega}{a} - \frac{y}{R_1^2} (x-b) \right)$$

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_w) as a known variable, in order to

determine the drain spacing (d_e 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:

$$[16] \quad \gamma = -\frac{8\tau'_h}{\ln\left(\frac{1-U_t}{u^*}\right)}$$

where,

$$[17] \quad \tau'_h = c_h t / d_w^2$$

$$[18] \quad u^* = \sum_{m=1}^{\infty} \frac{8}{(2m+1)^2 \pi^2} \exp\left(-\left(\frac{2m+1}{2}\right)^2 \pi^2 T_v\right) \quad (\text{Figure 3})$$

$$[19] \quad T_v = c_v t / l^2$$

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

$$[20] \quad n = \exp(\alpha \ln \gamma + \beta); \quad \text{where,}$$

$$[21] \quad \xi_c = (\kappa_c - 1) \ln(s_c) \quad (\text{Figure 4})$$

$$[22] \quad \alpha = 0.3938 - 9.505 \times 10^{-4} \xi^{1.5} + 0.03714 \xi^{0.5} \quad (\text{Figure 5})$$

$$[23] \quad \beta = 0.4203 + 1.456 \times 10^{-3} \xi^2 - 0.5233 \xi^{0.5} \quad (\text{Figure 5})$$

For linear variation of horizontal permeability in the smear zone with the same α and β parameters, it can be shown that:

$$[24] \quad \xi_L = \frac{\kappa_L (s_L - 1)}{s_L - \kappa_L} \ln\left(\frac{s_L}{\kappa_L}\right) - \ln(s_L) \quad (\text{Figure 6})$$

Equation (24) cannot be used to calculate ξ_L , when $s_L = \kappa_L$. The ξ parameter for the special case is given by:

$$[25] \quad \xi_L = s_L - 1 - \ln(s_L) \quad (\text{Figure 7})$$

Once either ξ_L or ξ_c is determined, the parameters n , α and β 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_t 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 τ'_h from Equation (17);
- (vi) Determine γ 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 ξ_c by Equation (21) or Fig. 4
- (viii-b) For a smear zone having a linear lateral permeability variation, calculate ξ_L by Equations (24) and Figure 6, or by Equation (25) and Figure 7;
- (ix) Determine n from ξ 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:

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

Design steps:

Step 1. $T_v = 0.5 \times 1.2 / 10^2 = 0.006$.

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 α and β . 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

$$\mu_x = \frac{\kappa_L}{\kappa_{LX}} \mu_L(n, s_{LX}, \kappa_{LX}) \text{ when } 2n - s_L \geq 1, \text{ and, } s_L > n.$$

Step 9. Determine d_e from $d_e = nd_w = 16 \times 0.06 = 0.96$ 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 $\Delta s_u = \bar{\alpha} \bullet \Delta \sigma'_v = 0.22 \times 72 = 15.84$ 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 ($q_{\max}$) 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 $q_{\max}$ is more than the required surcharge load (q_{req}), a single stage construction can be carried out following the design steps given in the previous section. If $q_{\max} < q_{req}$, 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 ($q_{\max}$) 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_r) 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_{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_{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 κ_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_{max}) can be determined using the slope stability analysis described earlier (Figure 9). For a safety factor of 1.6, q_{max} is 45 kPa (i.e. 2.5m height of surcharge fill having a unit weight of 18 kN/m³).

Step 2. As q_{req} = 70 kPa, $q_{max} < q_{req}$. 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

$$[26] \quad U_{stage2} = \frac{U_t \times \text{Total surcharge load} - U_{stage1} \times \text{surcharge load in stage 1}}{(1 - U_{stage1}) \times \text{surcharge load in stage 1} + \text{surcharge load in stage 2}}$$

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

- Barron, R. A. 1948. The influence of drain wells on the consolidation of fine-grained soils. Diss., Providence, US Eng. Office.
- Basu, D., Basu, P. and Prezzi, M. (2006). Analytical solutions for consolidation aided by vertical drains. *Geomechanics and Geoengineering: An International Journal*, 1(1): 63-71.
- Bo, M.W., Chu, J., Low, B.K. and Choa, V. 2003. Soil improvement; prefabricated vertical drain techniques. Thomson Learning, Singapore, 341 p.
- Bjerrum, L. 1972. Embankments on soft ground. *Proceedings, Specialty Conference on Performance of Earth and Earth-supported Structures*, Lafayette, Indiana, Vol. 2, pp. 1-54. (Reprinted in Norwegian Geotechnical Institute Publication No. 95, 1973, 27 p.)
- Carrillo, N. 1942. Simple two - and three-dimensional cases in the theory of consolidation of soils. *Journal of Mathematical Physics*, 21: 1-5.
- Hansbo, S. 1981. Consolidation of fine-grained soils by prefabricated drains. In *Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Balkema, Rotterdam. 3. pp. 677-682.
- Hartlen, J. and Wolski, W. 1996. *Embankments on Organic Soils*. Elsevier, 424 p.
- Indraratna B., and Redana I.W. 1998. Laboratory determination of smear zone due to vertical drain installation. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 124(2): 180-184.
- Indraratna B., and Redana I.W. 2000. Numerical modeling of vertical drains with smear and well resistance installed in soft clay. *Canadian Geotechnical Journal*. 37: 133-145.

- Indraratna, B., Rujikiatkamjorn C., and Sathananthan, I. 2005. Radial consolidation of clay using compressibility indices and varying horizontal permeability. *Canadian Geotechnical Journal*, 42: 1330-1341.
- Ladd, C. C., and Foott, R. 1974. New Design Procedure for Stability of Soft Clays. *Journal of the Geotechnical Engineering*, 100(GT7): 763-786.
- Madhav, M.R., Park, Y.-M. and Miura, N., 1993. Modelling and study of smear zones around band shaped drains. *Soils and Foundations*, 33(4): 135-147.
- Mesri, G. 1989. A re-evaluation of $\sigma_{u(mob)} = 0.22 \sigma'_p$ using laboratory shear tests. *Canadian Geotechnical Journal*. 26(1): 162-164.
- Ladd, C. 1991. Stability evaluation during staged construction. *Geotechnical Engineering Journal*, 117(4): 540-615.
- Onoue, A. 1988. Consolidation by vertical drains taking well resistance and smear into consideration. *J. Soils and Foundations*, 28(4): 165-174.
- Onoue, A., Ting, N.-H., Germaine, J.T. and Whitman, R.V., 1991. Permeability of disturbed zone around vertical drains, in *Geotechnical Engineering Congress, Proceedings of the Congress of the Geotechnical Engineering Division, 1991*, 879--890 (American Society of Civil Engineers: New York).
- Poulos, H. G., and Davis, E. H. 1974. *Elastic solutions for soil and rock mechanics*, Wiley and Sons, NY.
- Poulos, H. G. and Davis, E. H. 1974, *Elastic Solutions for Soil and Rock Mechanics*,. New York: John Wiley & Sons.
- Rujikiatkamjorn, C. and Indraratna, B. 2007. Analytical solutions and design curves for vacuum-assisted consolidation with both vertical and horizontal drainage. *Canadian Geotechnical Journal*, 44 (2), 188-200.
- Sathananthan, I. and Indraratna, B. 2006. Laboratory Evaluation of Smear Zone and Correlation between Permeability and Moisture Content, *J. of Geotechnical & Geoenvironmental*

- Engineering, ASCE, 132(7): 942-945.
- Sharma, J.S. and Xiao, D., 2000. Characterization of a smear zone around vertical drains by large-scale laboratory tests. Canadian Geotechnical Journal, 37(6), 1265-1271.
- Walker, R. and Indraratna, B. 2006. Vertical Drain Consolidation with Parabolic Distribution of Permeability in Smear Zone. J. of Geotechnical & Geoenvironmental Engineering, ASCE, 132(7), 937-941.
- Walker, R. and Indraratna, B., 2007. Vertical drain consolidation with overlapping smear zones. Geotechnique, 57 (5), 463-467.
- Wang, L.Z., Shen, K. and Ye, S.H., 2008. Undrained shear strength of K_0 consolidate soft soils. International Journal of Geomechanics, ASCE. 8(2), 105-113.
- Yoshikuni, H., and Nakanodo, H. 1974. Consolidation of fine-grained soils by drain wells with finite permeability. Japan Soc. Soil Mech. and Found. Eng. 14(2): 35-46.
- Zhu, G. F., Yin, J. H. 2001. Design charts for vertical drains considering construction time. Canadian Geotechnical Journal. 38(5): pp1142-1148

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Fig. 7. Contour plot of ξ_L for linear horizontal permeability variation in the smear zone based on Equation (25) when $s_L = \kappa_L$

Fig. 8. Procedure for the selection of construction method

Fig. 9. Slope stability analysis for the first stage embankment loading to determine q_{max}

Fig. 10. Slope stability analysis for the second stage loading to determine q_{max}

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

Friction angle (ϕ' , degree)	$\bar{\alpha}$
20-25	0.204
25-30	0.239
30-35	0.269

Table 2 Selected soil parameters for embankment design

Parameters	Soil layers			
	Surcharge fill	1	2	3
Depth		0.0-2.0	2.0-8.5	8.5-10.0
c_h (m ² /yr)		2.8	2.8	2.8
c_v (m ² /yr)		0.9	0.9	0.9
γ_i (kN/m ³)	18	16	16	16
OCR		1	1	1
s_{ui} (kPa)		15	12	14
$\overline{\alpha}$		0.22	0.22	0.22
c' (kPa)	10			
ϕ' (degrees)	29			

Table 3 Shear strength development after stage 1 construction

Soil layer (Ref. Fig. 10)	S_{ui} (kPa)	$\bar{\alpha}$	$\Delta\sigma'_v$ (kPa) (Eqs. 14 and 15)	S_u (after consolidation) (kPa) (Eq. 13)
1 (zone 1, beneath embankment)	15	0.22	31.17	21.86
2 (zone 1)	12	0.22	28.88	18.35
3 (zone 1)	14	0.22	25.40	19.59
1 (zone 2, beneath embankment slope)	15	0.22	15.65	18.44
2 (zone 2)	12	0.22	15.18	15.34
3 (zone 2)	14	0.22	14.58	17.21

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 ξ_C for constant horizontal permeability in the smear zone based on Equation (21) (Rujikiatkamjorn and Indraratna 2007)

Fig. 5. Relationships of ξ , α and β (Rujikiatkamjorn and Indraratna 2007)

Fig. 6. Contour plot of ξ_L for linear horizontal permeability variation in the smear zone based on Equation (24)

Fig. 7. Contour plot of ξ_L for linear horizontal permeability variation in the smear zone based on Equation (25) when $s_L = \kappa_L$

Fig. 8. Procedure for the selection of construction method

Fig. 9. Slope stability analysis for the first stage embankment loading to determine q_{max}

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