Application of Hybrid Drained-Undrained Model for Analyzing the Stability of Reinforced Soil Structures Over Soft Foundations with Prefabricated Vertical Drains

David Espinoza, *Senior Principal, Geosyntec Consultants, Columbia, Maryland, USA; email: despinoza@geosyntec.com*
Chunling Li, *Senior Engineer, Soil and Land Use Technology, Inc., Glen Burnie, Maryland, USA; email: cli@salutinc.com*
Lucas, de Melo, *Senior Principal, Geosyntec Consultants, Columbia, Maryland, USA; email: ldemelo@geosyntec.com*
Ranjiv Gupta, *Senior Engineer, Freeport-McMoRan, Phoenix, Arizona, USA; email: rgupta2@fmi.com*

ABSTRACT: *Prefabricated Vertical Drains (PVDs) are typically used for embankment construction over saturated soft cohesive soil deposits to accelerate consolidation and reduce construction time in the field. PVDs accelerate consolidation of thick soil deposits by reducing the drainage path from tens of meters to 1-2 meters depending on PVD spacing in the field. Current design methodologies typically consider the increase of shear strength due to accelerated consolidation, but still use undrained shear strength for the entire cohesive soil layer even after PVD's are installed. However, for cases in which PVDs are closely spaced, which allows excess pore water pressure to dissipate relatively fast, the assumption of undrained conditions for design may be overly conservative and, in some cases, this assumption may render an embankment construction unfeasible, unless additional ground improvement techniques are used to significantly enhance the foundation strength. This paper presents a Hybrid Drained-Undrained (HDU) model for construction of embankments over soft soils that accounts for the improved soil drainage conditions after installation of PVDs in the assessment of the shear strength used for design. A field case study is presented where the HDU methodology was used for the design of a 2.4-km long MSE berm constructed over a PVD-improved soft soil site, allowing for significant cost savings. The HDU approach was implemented using limit equilibrium models during the design stages to analyze the global stability of the MSE berm at different stages. Finite element models calibrated using field monitoring data collected during construction showed factors of safety comparable with that calculated using the HDU approach, which further supports the suitability of the HDU approach for PVD design.*

KEYWORDS: Hybrid Drained-Undrained model, slope stability, prefabricated vertical drain, limit equilibrium, shear strength

SITE LOCATION: Geo-Database

INTRODUCTION

Prefabricated vertical drains (PVDs) can be used to accelerate consolidation of thick deposits of poorly draining materials. PVDs are installed relatively close to each other to reduce the drainage distance from tens of meters to 1 to 2 meters, thus reducing the consolidation time by orders of magnitude. Because PVDs provide conduits for the excess pore pressure to dissipate, their installation improves the drainage characteristics of the low permeability material increasing the consolidation speed during loading (Atkinson and Eldred 1981; Hansbo 1981; Holtz 1987; Holtz et al. 1991). As a result, the undrained shear strength of low permeability soils increases at a faster rate (i.e., higher undrained shear strengths are achieved sooner) due to the accelerated consolidation. In general, standard analysis of low-permeability soils with PVDs only considers its ability to speed up the consolidation process and the associated undrained shear strength increase, and neglects the significantly lower excess pore water pressure generation potential near the installed PVDs.

Submitted: 03 October 2018; Published: 01 April 2020
Reference: Espinoza, D., Li, C., de Melo, L. and Gupta, R. (2020). Application of Hybrid Drained-Undrained Model for Analyzing the Stability of Reinforced Soil Structures Over Soft Foundations with Prefabricated Vertical Drains, International Journal of Geoengineering Case Histories, Vol.5, Issue 3, p. 138 - 151. doi: 10.4417/IJGCH-05-03-01

It is well known that shear strength and soil deformation behavior are governed by effective stress parameters. In theory, excess pore pressures generated by changes in compressive and shear stresses can be predicted at any time during loading and the response of soils to the applied loads could be modeled using effective stress analysis (ESA) along with effective stress (i.e., drained) shear strength parameters, independently of the soil permeability. Hence, highly permeable material (e.g., sands, gravels) that dissipates any excess pore pressures generated during construction immediately as well as low permeable soils (e.g., clays) that need time for excess pore pressures to dissipate could be modeled using ESA. In practice, highly permeable materials are analyzed using ESA effective stress parameters because all excess pore pressures generated during loading are readily dissipated. Because excess pore pressures can be at times difficult to predict reliably and expeditiously for a variety of field loading conditions, low permeable materials are typically analyzed using total stress analysis (TSA) along with total stress (i.e., undrained) shear strength parameters since predicting the magnitude of excess pore pressures during loading is not necessary in this type of analysis.

The indiscriminate use of TSA for low-permeable soils without consideration of the speed of construction and ability of low permeability soils to drain excess pore pressures can lead to overly conservative designs, in particular for cases where the drainage characteristics of the thick fine-grained deposits have been modified by the installation of highly permeable media such as PVDs.

Conventionally, slope stability analysis for staged construction is conducted using either the ESA or TSA. The ESA is typically conducted using field measured piezometer data to account for the excess pore water pressure generated during construction. For ground improved with PVD, the ESA is difficult because the excess pore water pressures change rapidly for zones close to the PVD and zones away from the PVD. Ladd (1991) presented an example for analyzing the stability of an embankment constructed over ground improved with PVD using the TSA. In this example, the discussion focused on the selection of undrained shear strength based on the Stress History and Normalized Soil Engineering Properties (SHANSEP) approach. Various factors affecting the performance of PVD were also discussed. This analysis approach essentially only considered the undrained shear strength increase due to the accelerated consolidation by the installed PVDs, but still used undrained shear strength for the entire soil layer. This approach conservatively neglects the fact that PVDs also create a drainage interface that allows a zone of soils around it to dissipate excess pore water pressure during loading, similar to a coarse-grained free draining material.

In what follows, the basis for the development of a Hybrid Drained-Undrained (HDU) methodology that takes into consideration the change of the drainage characteristics in the vicinity of PVDs is presented and its implementation in a standard limit-equilibrium analysis is discussed. This proposed approach allows the actual construction pace to be taken into account in a standard limit equilibrium analysis without performing sophisticated numerical analysis. Also presented in this paper is a field case study where this new method was implemented.

HYBRID DRAINED-UNDRAINED METHODOLOGY

For simplicity, standard design techniques using the TSA approach for fast loading conditions assume that loading is instantaneous (i.e., the actual loading rate is neglected) and use undrained shear strength for stability analysis. Although the undrained shear strength increase due to accelerated consolidation is accounted for in the various construction stages, the entire soil layer with PVDs installed is still considered undrained in the TSA approach.

If the ESA approach was used instead, the maximum excess pore pressures (u_{max}) generated after placement of a soil lift should be estimated assuming that the loading is applied instantaneously, and excess pore pressures are equal to the weight of the soil lift. Although it is evident that excess pore pressures at the PVD location should be zero and increase with radial distance from the PVD, in practice, it is conservatively assumed that excess pore pressures between PVDs are uniform and equal to u_{max}. However, this conservative assumption made for computation and monitoring expedience not only neglects the fact that the excess pore pressures are not uniform between PVDs, but also does not take into consideration how PVDs change the soil response to loading.

The proposed Hybrid Drained-Undrained (HDU) approach consider a zone surrounding the PVD to behave differently than the original low-permeability soil. Figure 1 shows a hypothetical variation of pore pressures between PVDs. The closer to the PVD, the smaller the generated excess pore pressure is and the faster it is dissipated. Hence, depending upon the speed of construction and PVD spacing, designers can assume the existence of two distinct zones with different shear strength characteristics during loading: a fully drained zone (i.e., with negligible excess pore pressures generated) near the PVDs, and an undrained zone further away from the PVDs. This concept constitutes a significant departure from standard design of soft cohesive soils with PVDs and it is the central element of the foundation improvement design procedure for construction of embankments over soft soils.

As the excess pore pressures of the soil located closer to the PVDs are not only smaller, but dissipate faster than pore pressures generated farther away from the PVD, depending upon the rate of loading, a portion of the soil that surrounds the PVDs will dissipate a significant portion of the pore pressures generated during loading. In essence, the soils surrounding the PVDs can be viewed as 'virtual sand piles'. The idea of the HDU method is to estimate the radius of this virtual sand pile within which the generated pore pressures have a negligible effect on the stability of the foundation. Although the zone near the PVDs is described as a drained zone, excess pore pressures, albeit smaller, will be present during loading and they will need to be included in the analysis. The separation of the soft soil layer into two different sets of shear strength parameters is the centerpiece of the conceptual model described in this paper. For soft soils that display significant difference in drained and undrained shear strength, the over-conservatism in conventional design approach could be significantly reduced by considering a portion of the area close to the PVD as 'drained'.

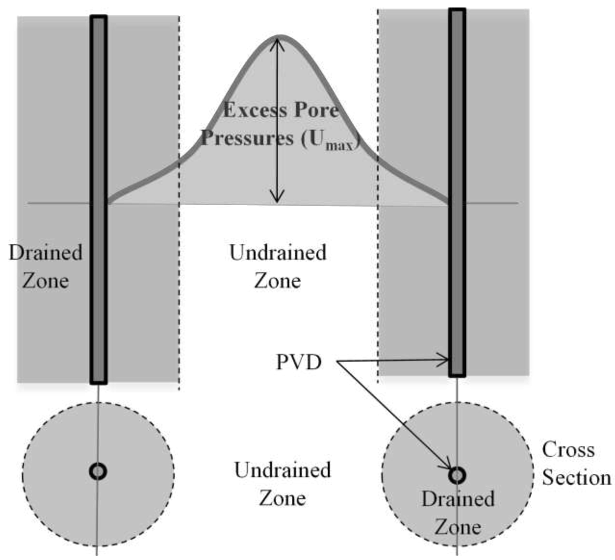


Figure 1. Conceptualization of pore pressure distribution between PVDs per the HDU methodology.

The basic steps of the HDU approach for slope stability analysis are the following:

- Estimate the excess pore water pressure distribution around the PVD.
- Select the area ratio of drained/undrained zone.
- Divide the PVD installed area into drained and undrained zone and assign effective (drained) and undrained shear strength parameters for drained and undrained zone respectively. Perform slope stability analysis.

The concepts of the HDU methodology are illustrated below.

Excess Pore Pressure Estimate

To estimate the area ratio of the drained/undrained zone, it is necessary to estimate the excess pore water pressure distribution, which is affected by the load magnitude and rate of application, consolidation characteristics of the soils, and PVD spacing. The excess pore water pressure distribution between PVDs may be estimated using conventional pore water pressure generation and consolidation theory. In what follows, a simplified method is presented.

To simplify the model development, the loading rate was assumed constant and equal to R_c (in pressure/time). For each lift of soil, it was assumed that excess pore pressures start to dissipate soon after it was placed (see Figure 2). Assuming an exponential decay function (Figure 3), the resulting excess pore pressure generated by an incremental loading can be calculated as follows, based on Barron's one-dimensional radial consolidation theory:

$$u(t)=\frac{R_c}{\alpha}\left[1-e^{-\alpha t}\right] \quad \text{for} \quad t\le t_p \tag{1}$$

where: t_p is the time needed to place the fill and α is a parameter that is related to Barron's Equations (1948) developed for sand drains:

$$\alpha=\frac{2}{F_n}\left(\frac{c_h}{r_i^2}\right) \tag{2}$$

$$F_n=\frac{n^2}{n^2-1}\ln(n)-\frac{3n^2-1}{4n^2} \tag{3}$$

$$n=\frac{r_i}{r_e} \tag{4}$$

And c_h is the coefficient of consolidation; r_i is the radius of influence of the PVDs; and r_e is the equivalent radius of the PVD (typically defined as perimeter of PVD cross section divided by 2π). The maximum pore pressure takes place at $t = t_p$. It follows that after fill placement, it is assumed that excess pore pressure dissipates according to the same decay function, then:

$$u(t)=\frac{R_c}{\alpha}\left[1-e^{-\alpha t_p}\right]e^{-\alpha(t-t_p)} \quad \text{for} \quad t>t_p \tag{5}$$

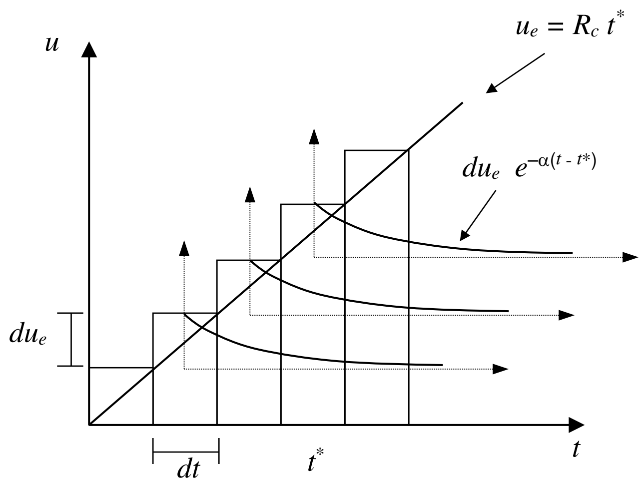


Figure 2. Pore Pressure Model for Incremental Loading.

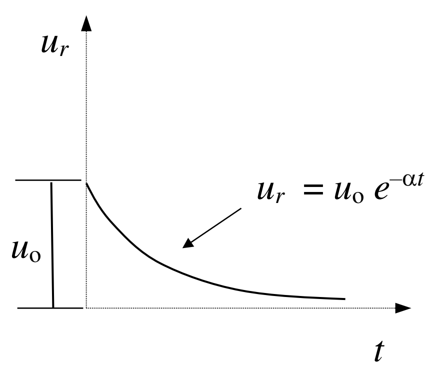


Figure 3. Pore Pressure Model for Incremental Loading.

The α parameter in Equation (2) was derived based on the assumption that the surrounding soil is not disturbed by the installation of PVD. However, in reality, PVD installation disturbs the surrounding soil creating a smear zone with reduced permeability around the periphery of the PVD. The dissipation of excess pore water pressure is also dependent upon the vertical flow capacity of the PVD. When well capacity and smear zone are needed to be considered, the α parameter in Equation (2) should be replaced with the following equations (Hansbo, 1981):

$$\alpha = \frac{2}{F}\left(\frac{c_r}{r_i^2}\right) \tag{6}$$

$$F = F_n + F_s + F_r \tag{7}$$

$$F_s = [k_r / k_s - 1]\ln s \tag{8}$$

$$F_r = \pi z(2l - z)\cdot\frac{k_r}{q_w} \tag{9}$$

where k_r is the soil hydraulic conductivity in the radial direction; k_s is the hydraulic conductivity of the smear zone in the radial direction; s is the radius of smear zone divided by r_e (typically 2 to 3, but may be as high as 6.3 according to Da Silva (2013); q_w is the PVD discharge capacity; l is the drain length; and z is the soil thickness, and F_n is defined by Equation (3). The terms F_s and F_r in Equation (4) represent the effect of smear and well resistance, respectively. F_s and/or F_r will be set to zero, if either or both of these effects are not considered.

The above equations represents the basic method for estimating the excess pore water pressure distribution within foundations with installed PVD. Although significant improvement to the PVD consolidation theory and excess pore water pressure distribution has been made over the past few years (e.g., Holtz et. al., 1991; Zhu and Yin, 1998 and 2001; Deng, et., al., 2013; Conte and Troncone, 2009; Indraratna and Redana; 2000; Wang and Jiao, 2004; Rujikiatkamjorn and Indraratna, 2009), the focus of this paper is to implement a practical solution to be used with limit equilibrium theory. For applications focused on the PVD consolidation process itself, the reader should refer to the papers listed above.

Selection of Drained Undrained Area Ratio

As stated previously, for the HDU approach, soils enhanced with PVDs could be viewed (and analyzed) as a soft soil layer enhanced with virtual sand piles. In other words, the soil columns around the PVDs (hereafter, virtual sand piles) develop a drained shear strength during loading, whereas the soil outside the virtual sand piles develops an undrained shear strength response during loading. The key concept of the HDU methodology is to select the percentage of drained and undrained area based on the estimated excess pore water pressure distribution. The modified procedure consists of the following:

1. Selecting the magnitude of excess pore pressure that would have negligible effect on factor of slope against instability. If the average pore pressure generated within a certain distance from the PVD is below a selected threshold (hereby referred to as the drained/undrained excess pore water pressure threshold), the material within this distance could be considered drained. As a rule of thumb, the excess pore water pressure threshold can be selected as (0.55~1.1)S_u *(This corresponds to 0.1 to 0.2 times the weight of critical embankment height constructed over the soft soil ($\gamma H_c = 5.52\times Su$)),* where S_u is the undrained shear strength of foundation soil before PVD installation. The validity of selected threshold could be checked by applying an excess pore water pressure equivalent to 50% of the selected threshold to the assumed 'drained zone' using the procedures outlined below and evaluate the sensitivity of the factor of safety to this assumed excess pore water pressure. If after applying the defined excess pore water to the drained zone the change of factor of safety is insignificant, then the proposed threshold is considered acceptable.
2. Estimate the excess pore water pressure generation for various distances from the PVD at the time of interest, based on the construction rate, consolidation parameters and designed PVD spacing, and then back-calculate the distance from center of PVD to where the excess pore water pressure equals the drained/undrained excess pore water pressure threshold. This distance is considered as the radius of virtual sand pile (r_s).

3. Select the undrained/drained area ratio. The percentage of drained area can be simply estimated as:

$$A_r = \left(\frac{2r_s}{D_{pvd}}\right)^2 \times 100 \tag{10}$$

where D_{pvd} is the effective PVD spacing (D_{pvd} =1.05S for triangular pattern, and 1.13S for square pattern, where S = distance center-to-center PVD spacing) and r_s is the radius of the virtual sand pile.

It is noted that the ratio A_r may change with time. For slope stability evaluation of staged construction, the most critical time would be the end of construction at each stage when the load and excess pore water pressure are both at the maximum. Thus, the typically selected time of interest for A_r calculation will be the end of construction stage. For construction with a fixed construction pace (e.g., fixed thickness of fill in a fixed time frame for each construction stage), the A_r value can be viewed as approximately constant for the stability analysis of each construction stage.

Application of HDU Methodology for Slope Stability Evaluation

The HDU methodology expedites the stability analysis during the design stage as it can be readily implemented using conventional limit equilibrium methods taking into consideration the soil strengths in the drained and undrained zones. In the limit equilibrium model, the subsurface material enhanced with PVD was idealized with vertical strips of area representing the alternating 'drained' and 'undrained' areas. The percentage of defined 'drained' area of all PVD-enhanced area should be equal to the A_r value as calculated above. The width of the soil vertical strips in the modified stratigraphy does not need to represent the actual width of the virtual sand column. Only the ratio between the estimated drained to undrained areas needs to be maintained. However, the number of vertical strips should be selected such that the failure mechanism is not influenced by the modified stratigraphy (for instance, two vertical strips would not be appropriate as the failure mechanism would be governed by the portion of the soil that is modeled with undrained shear strength parameters). The soil columns around the PVDs can be modeled with a drained shear strength response during loading, whereas the soil outside the virtual sand piles will be assumed to develop an undrained shear strength response during loading.

CASE HISTORY

The HDU methodology has been successfully applied to design a 20-m high berm constructed over soft ground improved with PVD, which is described below.

Project Description

The Cherry Island Landfill (landfill), located in Wilmington, Delaware, is owned and operated by Delaware Solid Waste Authority (DSWA). DSWA has utilized the landfill to provide safe disposal of waste since 1985. This 100-hectare facility is hemmed in by Interstate 495, the City's wastewater treatment plant, and the confluence of the Delaware and Christina Rivers, which are two of the eastern seaboard's most navigable rivers (Figure 4). The landfill was constructed over an area that was partly reclaimed from the Delaware River in the early 1900s (see Figure 4) and that had been used for many years as a dredged material disposal site by the US Army Corps of Engineers (USACE). The subsurface conditions at the site consisted of about 12 m thick dredged materials, overlying 14 m thick alluvial deposit, underlain by a medium dense to dense sand layer. The geotechnical properties of the dredge/alluvium material were obtained from an extensive field investigation program that included in-situ cone penetration tests, field vane tests, and standard penetration tests as well as laboratory tests. The obtained geotechnical parameters are summarized in Table 1. The soft and compressible characteristics of the dredge/alluvium material constrained the landfill capacity within the permitted footprint as waste disposal was limited to the relatively flat slope of 8H:1V on average, rather than the typical configuration with 3H:1V slopes. As a result, in 2000, after only 15 years of operation, the site reportedly had five years of remaining capacity left. To meet the waste disposal needs of the community for the next 20 years, DSWA estimated that an additional 16 million m^3 of waste disposal capacity would be required. Following an investigation of several candidate sites for a replacement landfill, DSWA concluded that the most cost-effective and sustainable path forward was to pursue an expansion of the existing landfill. However, due to its proximity to the rivers, the site could not be expanded laterally. Therefore, the vertical expansion of the landfill was proposed, which required a perimeter of MSE berm of 2,400 m long and 21 m high.

A preliminary feasibility study for the project indicated that in order to build a 21-m high MSE berm, the foundation strength would need to be improved, from 10 kPa to 150 kPa. The initial conceptual solution proposed for foundation improvement was the use of deep soil mixing (DSM), a technique that consists of mixing soil with cement. Using the DSM method, the volume of soil that would need to be treated to improve the foundation strength was estimated at approximately 2 million m^3. At the time the construction of the soil improvement took place (2006), cement prices were significantly higher because of global demand, and the estimated cost for the DSM option was estimated to be $150 million (2011 US dollars).

Figure 4. Cherry Island Landfill Site Plan.

Table 1. Geotechnical Properties of Dredge Alluvium Material.

Parameters	**Range**	**Typical**
Hydraulic Conductivity, cm/sec	1×10^{-6} - 3×10^{-9}	1×10^{-7}
Undrained Shear Strength	-	0.29 times effective overburden stress
Effective Friction Angle, degrees	32-38	34
Void Ratio	2.1-2.8	2.5
Compression Index	0.53-1.1	0.8
Recompression Index	0.63-1.7	1.6
Saturated Unit Weight (kN/m^3)	11.8-17.3	15.2
Horizontal Consolidation Coefficient, C_h (m^2/day)	0.0037-0.028	0.0065
Liquid Limit, %	18-124	72
Plasticity Index, %	10-73	40
Unified Soil Classification System	CL-CH	CH

Due to the high volumes of soil mixing, alternatives to DSM were evaluated and it was concluded that PVDs would take advantage of the massive weight of the MSE berm to improve the foundation strength and would be far more economical. The PVDs would help dissipate the excess pore pressures generated in the foundation soil during the construction of the MSE berm, thereby, allowing the foundation soil to consolidate and gain strength. Figure 5 shows the schematic of the MSE berm with PVDs installed in the foundation dredge/alluvium material.

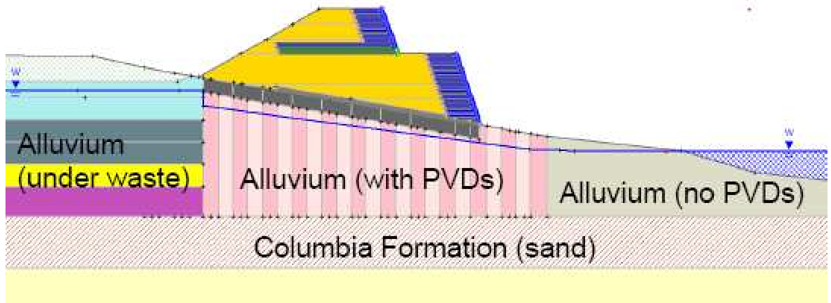


Figure 5. Schematic of MSE Berm at Cherry Island Landfill.

As discussed earlier, the maximum height of an MSE berm on soft soils is typically dictated by the undrained shear strength of the underlying soft material. Initial design analyses conducted using the approach by Ladd (1991) with a factor of safety of 1.25 indicated that the maximum berm height at the landfill would be 7.5-m (i.e., about 13.5 m shorter than required to achieve the target airspace of 16 million m^3. Using the HDU model as described in the previous sections, the design analyses indicates that the MSE berm construction can proceed to the final design elevation. The details of the analysis are presented below.

During construction, a geotechnical monitoring program was implemented to confirm that the foundation soils behave as expected. Geotechnical monitoring instruments were installed along 16 cross sections of the MSE berm prior to construction. The key monitored parameters and corresponding monitoring devices include the following:

- excess pore water pressures below the center of the berm monitored by vibrating wire piezometers installed at three depths below the center of the triangular pattern of the installed PVDs;
- lateral displacement profile at the toe of the berm monitored by inclinometers;
- vertical displacement below the center of the berm monitored by settlement plates.

Finite element models calibrated using the geotechnical monitoring data collected during the early stages of the construction were used to evaluate the stability condition for various construction stages and to judge if the dredge/alluvium has sufficiently consolidated to allow for the construction of the next stage. Figures 6 through 8 show the comparison of the monitoring results with the finite element model predictions for one of the analyzed cross sections. As shown in these figures, the finite element model provided good prediction of the soil's behavior. Factors of safety were calculated from the calibrated finite element models using the shear strength reduction technique (Matsui and San, 1996), and compared to the factors of safety calculated using HDU model (see discussion below). The good agreement between geotechnical monitoring data and finite element model prediction allowed us to build confidence with the original design conducted using the HDU approach. Correlations between the monitored displacements and the stability conditions obtained from the Cherry Island Landfill experience are discussed in detail in Li and Espinoza (2017).

Design Using HDU Approach

Based on the site investigation and laboratory testing, the dredge/alluvium material underlying the site was characterized using the geotechnical properties shown in Table 1. Undrained shear strength was estimated from results of in-situ cone penetration tests along with field vane shear tests normalized using the SHANSEP concept (Ladd and Foote, 1974). Effective shear strength was obtained from simple shear tests of undisturbed samples. The HDU method was used to estimate the radius of the drained zone (i.e., the virtual sand pile diameter where generated excess pore pressures are negligible), the appropriate PVD spacing, and the corresponding rate of construction that result in the appropriate ratio of soil that could be considered drained during MSE construction. The MSE berm was designed to be constructed at the following rate:

- approximately 3-m lifts with each lift constructed at a rate of 0.45 m per week;
- subsequent lifts constructed after 90% of excess pore pressures generated from loads of previous lifts have been dissipated (estimated to be approximately three months).

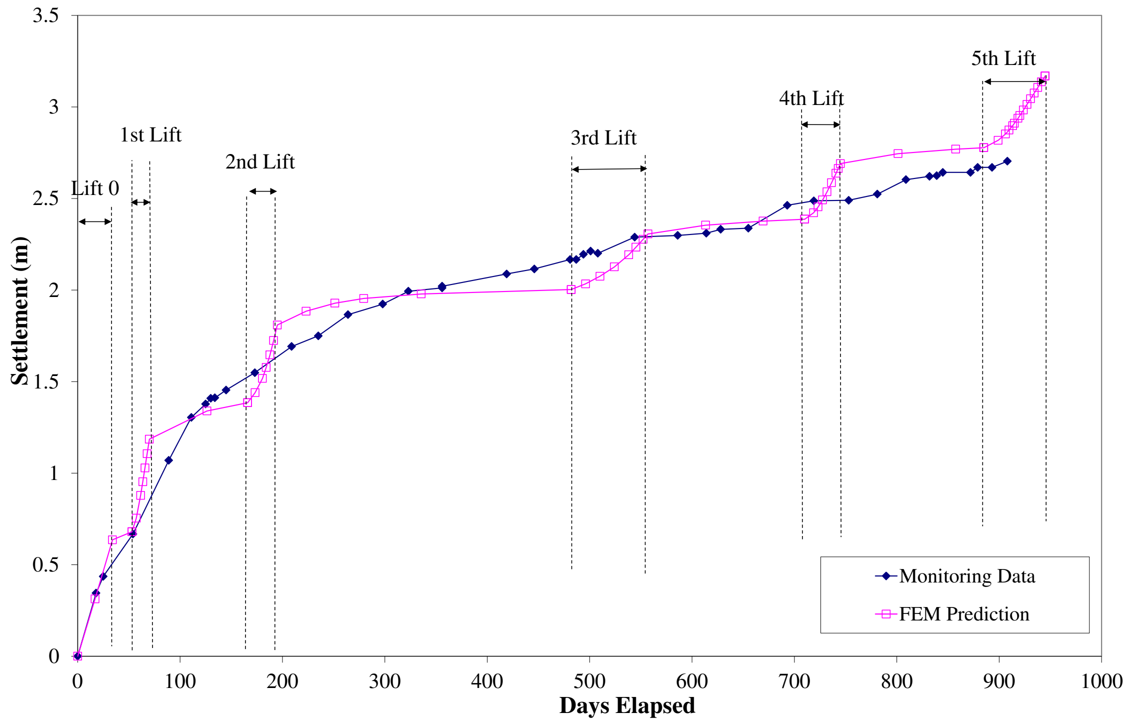


Figure 6. Comparison of Calculated Embankment Settlement with Monitoring Results.

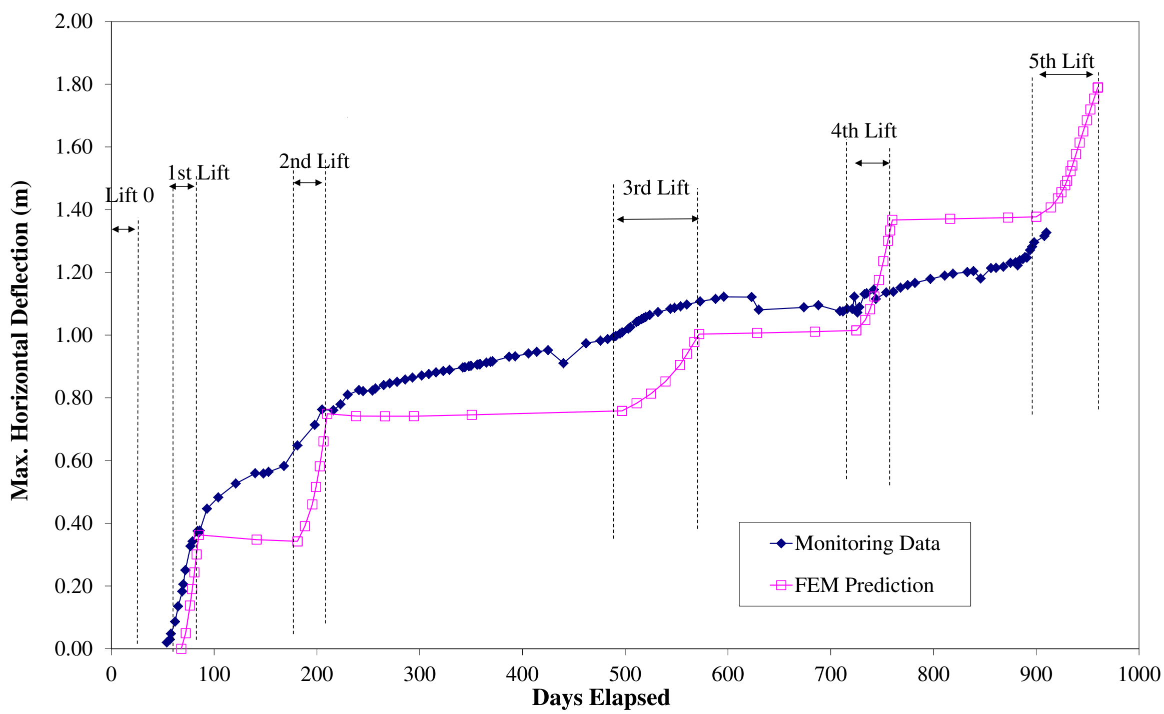


Figure 7. Comparison of Calculated Embankment Lateral Displacement with Monitoring Results.

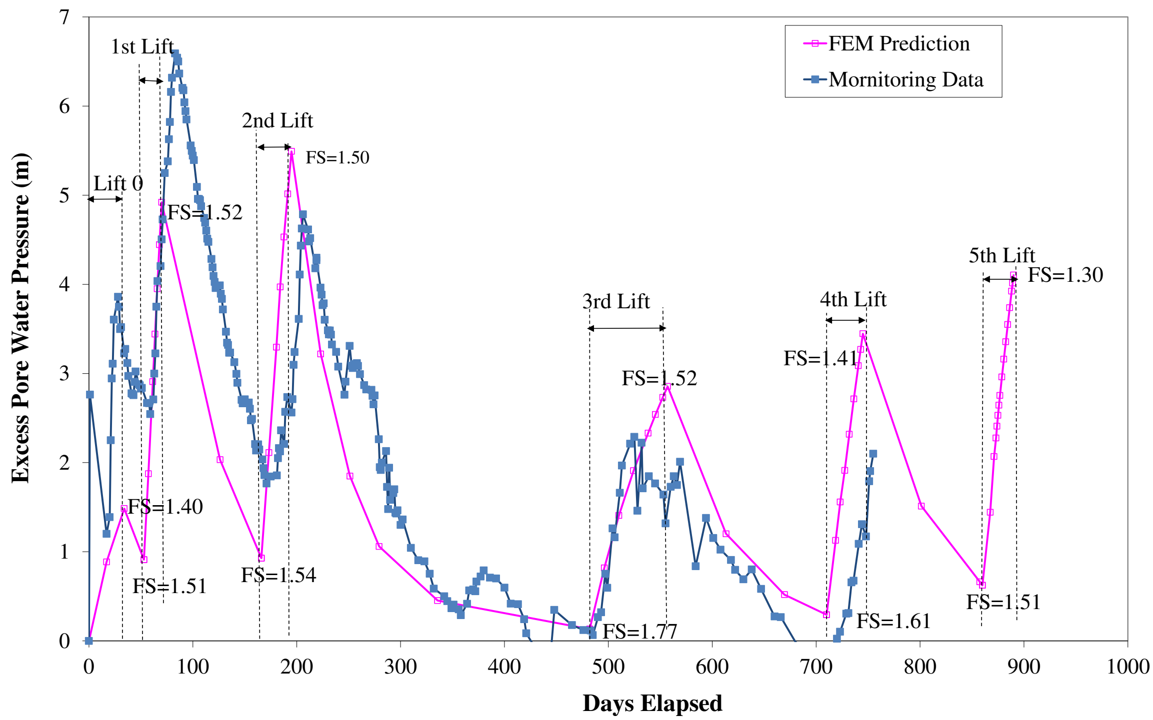


Figure 8. Comparison of Calculated Excess Pore Water Pressure with Monitoring Results.

The equivalent radius for a 100 mm wide 5 mm thick PVD is calculated as 33.4 mm. For PVD constructed at a 1.5-m center-to-center spacing in a triangular pattern, and at the construction pace as described above, the excess pore water pressure at various radius of influence (*ri*) was calculated using Equations (1) through (4). No smear effect or well capacity was considered. At the selected construction pace (3-m lift placed in 45 days), the estimated *Rc* was 1.34 kPa/day. Using 1.1*Su* as the drained/undrained excess pore water pressure threshold, this threshold is estimated as 30 kPa using foundation undrained shear strength of 27 kPa prior to PVD installation. As shown in Figure 9, it was estimated that within a distance of 0.46 m from the center of the PVD, the estimated induced excess pore water pressure at the end of the construction of a 3-m thick lift was at the selected threshold. The material within this distance from the center of PVD was considered drained. If constructed at a faster rate, say 3-m lift constructed in 1 day, the drained area will be much smaller. At the selected construction rate, the assumed 'drained' area represented approximately 36 percent of the total area for PVDs installed at a 1.5 m center-to-center spacing, as calculated below.

$$A_r = \left(\frac{2\times0.46}{1.05\times1.5}\right)^2 \times 100 = 36\% \tag{11}$$

For comparison, if the MSE berm fill is placed at an extremely fast rate, say 3-m thick layer in 24 hours (R_c = 60 kPa /day), the estimated distance from center of PVD to where the excess pore water pressure exceeds the selected threshold would be 0.12 m. The corresponding drained area will be 2 percent of the total area. This will yield an HDU model that is essentially the same as the standard approach which considers 100 percent of the PVD-installed area undrained.

In the limit equilibrium model for the Cherry Island Landfill, the subsurface dredge/alluvium material enhanced with PVD was idealized with vertical strips of area representing the alternating 'drained' and 'undrained' areas. Figure 10a shows a hypothetical homogenous soil stratigraphy prior to PVD installation and Figure 10b shows the modified stratigraphy to account for the installation of PVDs. The shear strength for the 'drained' area was assumed to have the effective friction angle of 34 degrees. The 'undrained' area was assigned an undrained shear strength equal to 0.29 times of the effective overburden stress, as shown in Table 1. The stability at various construction stages was evaluated using limit equilibrium analysis. It was assumed that the excess pore water pressure induced by the previous lift has fully dissipated before placement of the next lift. To avoid increasing the undrained shear strength due to the recently placed new lift, it was assumed that the vertical effective stress remains unchanged during loading. This was achieved by applying to the undrained material excess pore water pressure equal to the overlying weight of the new lift.

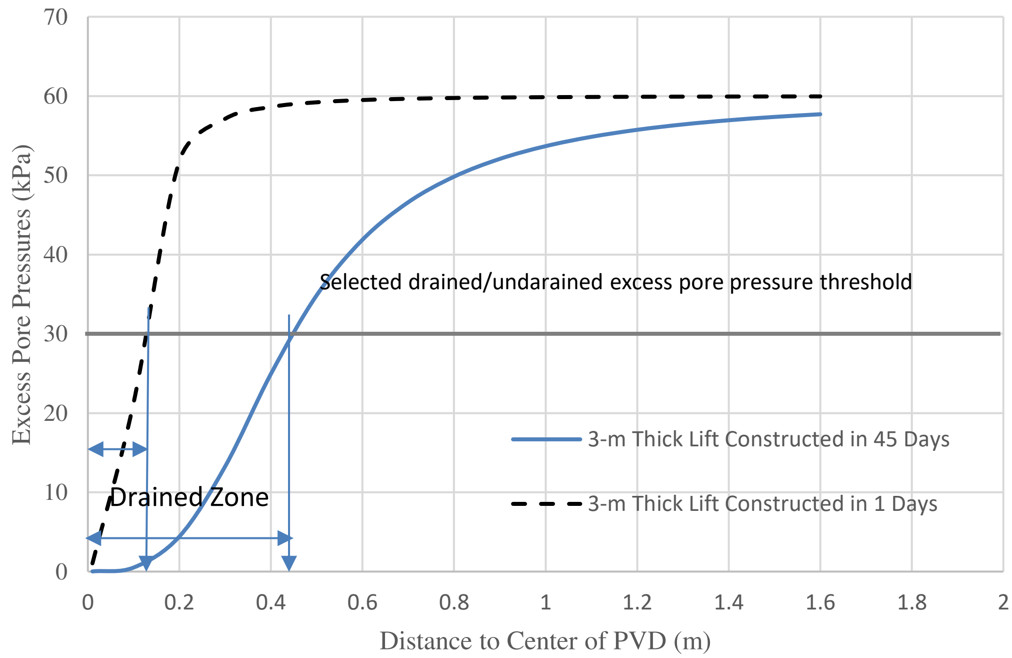


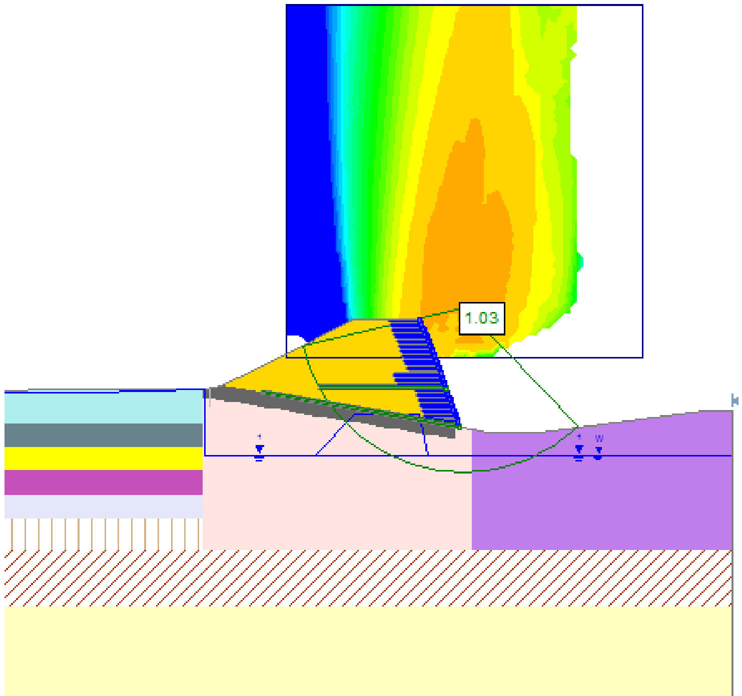
Figure 9. Estimated Excess Pore Water Pressure Distribution with Varying Distance to Center of PVD at the End of Each Lift Placement.

Comparison of Factors of Safety Calculated

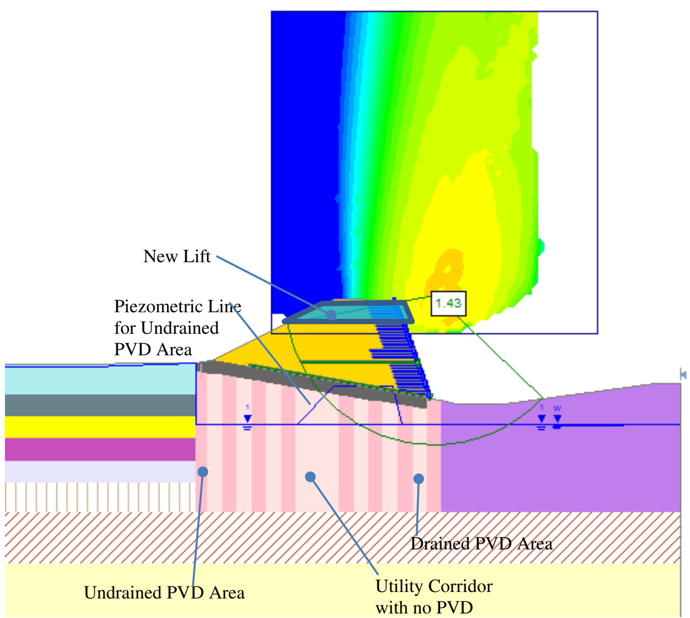
As shown above, the HDU approach considers drained parameters for zones close to the PVD considering that the generated excess pore pressures can be quickly dissipated, and assigns undrained strength to zones away from the PVD. Using the standard approach (e.g., Ladd, 1991), the entire PVD would have been assumed to behave as ‘undrained’, which is significantly more conservative than the HDU approach. In addition to the HDU and conventional approach, factors of safety were also calculated using the shear strength reduction method in the calibrated finite element model. The finite element model was calibrated to reproduce the measured excess pore water pressure generated and dissipated in the field. Effective stress shear strength parameters along with excess pore water pressure were considered in the finite element approach. Essentially, the factor of safety calculated by the finite element models is based on ESA. For the Cherry Island Landfill project, the factors of safety calculated for one critical MSE berm section immediately after the construction of the last lift using HDU and conventional approach are shown in Figure 10. The conventional approach (i.e., the entire dredge/alluvium layer with PVDs is considered to behave undrained) predicted a factor of safety of 1.03. The HDU method as described above calculated as factor of safety of 1.43. The shear strength reduction method of the finite element model predicted a factor of safety of 1.36. As it can be seen from these results, the predicted factors of safety by FEM model and HDU are comparable, while the conventional approach is significantly more conservative than the other two methods.

CONCLUSIONS

A new methodology for stage construction on soft soils using PVDs, termed the HDU methodology, was presented. The HDU methodology constitutes a departure from standard design of soft cohesive soils with PVDs as a portion of the soil that is closer to the PVDs is modeled using drained parameters. For soft soils that show significant difference between drained and undrained shear strength, the consideration of the improved drainage conditions of the soils surrounding the PVDs could lead to significant cost savings in the design. The robustness of the proposed method was demonstrated in the field through the successful construction of a 21-m high, 2,400-m long, 1.5-million-cubic-meter MSE berm over 30-m deep layer of very soft soils (undrained shear strength as low as 10 kPa) that was designed using the HDU methodology. The MSE berm was constructed from 2006 through 2010. A comprehensive geotechnical monitoring program was implemented to collect data during construction and subsequent loading.



(a) Conventional Approach



(a) HDU Approach

Figure 10. Limit Equilibrium Results Using Conventional and HDU Approach.

A step-by-step procedure was presented to facilitate its application by practicing engineers. As discussed above, the HDU concept can be readily integrated into standard slope stability analysis. Instead of assuming a fully undrained condition in the conventional approach, the proposed HDU methodology allows the construction rate to be taken into account in the standard limit equilibrium approach. Although a more sophisticated analysis such as finite elements along with an appropriate geotechnical monitoring program is recommended for stability analysis of critical structures, this method can be used for design of less critical structures with limited design and monitoring budgets or applied for a preliminary analysis of critical structures to facilitate the design process.

Because of the innovative design methods and construction techniques used for the construction of the MSE berm, this project was selected by the American Society of Civil Engineering among the five finalists for the 2012 Outstanding Civil Engineering Achievement Award.

REFERENCES

Atkinson, M.S. and Eldred, P.J.L. (1981). "Consolidation of Soil Using Vertical Drains", *Geotechnique*, 31(1), 33-43.

Barron, R.A. (1948). "Consolidation of Fined-Grained Soils by Drain Wells", *ASCE Trans*, paper 2346, 113, 718-724.

Basu D., Basu P. and Prezzi M. (2006). "Analytical solutions for consolidation aided by vertical drains", *Geomechanics and Geoengineering: An International Journal (1991)*, 1(1), 63-71.

Bergado, D.T., Asakami, H., Alfaro, M.C. and Balasubramaniam, A.S., (1991). "Smear effects on vertical drains on soft Bangkok clay", *J. Geotech. Eng., ASCE*, 117(10), 1509-1530.

Conte E, Troncone A (2009). "Radial consolidation with vertical drains and time dependent loading", *Can Geotech J,* 46, 25-36

D'Appolonia, J.D., Lambe, T.W., and Poulos, H (1971). "Evaluation of Pore Pressures Beneath an Embankment", *Proc. of the ASCE*, 97, SM6, 881-897.

Deng, Y. B., Xie, K. H., Lu, M. M., Tao, H. B., & Liu, G. B. (2013). "Consolidation by prefabricated vertical drains considering the time dependent well resistance", *Geotextiles and geomembranes*, 36, 20-26.

de Melo, L. and Espinoza, D. (2012). "Design and Construction of Mechanically Stabilized Earth Wall over Very Soft Soils (In Portuguese)", *XVI Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica*, Recife.

Espinoza, R.D., Repetto, P.C., and Muhunthan, B.(1992). "General Framework for Slope Stability Analysis", *Geotechnique, 42(* 4), 603-615.

Espinoza, R.D., Bourdeau, P.L., and Muhunthan, B.(1994). "Unified Formulation for Stability Analysis of Slopes with General Slip Failure Surfaces", *Journal of Geotechnical Engineering ASCE*, 120, (7), 1185-1204.

Espinoza, R.D**.**, Houlihan, M.F. and Ramsey, T. (2011). "Design of High Soil Berms Over Soft Soils", *Geo-Strata*, 52-54.

Espinoza R.D. and Li, C. (2013). "The Application of a Novel Design Approach for Construction over soft soils: The Hybrid Undrained-Drained model.", *Proc. of the 18th International Conference on Soil Mechanics and Geotechnical Engineering,* Paris 2013.

Hansbo, S. (1981). "Consolidation of fine-grained soils by prefabricated drains", *Proc. of the 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, 677-682.

Henkel, D.J. (1960). "The Shear Strength of Remolded Clays", *Proc. of the ASCE research conference on shear strength of cohesive soils*, Boulder, 535-554.

Holtz, R.D. (1987). "Preloading with prefabricated vertical strip drains", *Geotext Geomembranes*, 6, 109-131.

Holtz, R.D., Jamiolkowski, M.B., Lancellotta, R., and Pedroni, R., (1991). "Prefabricated Vertical Drains: Design and Performance".

Indraratna, B. and Redana, I W. (2000). "Numerical modelling of vertical drains with smear and well resistance installed in soft clay", *Canadian Geotechnical Journal*, 37, 132-145.

Indraratna B., Rujikiatkamjorn C. and Sathananthan I. (2005). "Radial consolidation of clay using compressibility indices and varying horizontal permeability", *Canadian Geotechnical Journal*, 42(5), 1330-1341.

Li, C. and Espinoza, R.D., 2017. "Assessment of reinforced embankment stability over soft soils based on monitoring results", *Geosynth. Int.*, 24(3), 264-279.

Ladd, C.C. (1991). "Stability evaluation during staged construction", *Journal of Geotechnical Engineering*, 117(4).

Matsui, T., and San, K. C. (1992). "Finite element slope stability analysis by shear strength reduction technique", *Soils and foundations*, *32*(1), 59-70.

Müller, R. and Larsson, S. (2013). "Aspects on the modelling of smear zones around vertical drains", *Proc. of the 18th International Conference on Soil Mechanics and Geotechnical Engineering*, Paris 2013, 2965-2968.

Olson, R.E. (1977). "Consolidation under Time-Dependent Loading"*, Journal of Geotechnical Engineering Division*, American Society of Civil Engineers, 103, 55-59.

Poulos and Davis (1974). *Elastic Solutions for Soil and Rock Mechanics*, Wiley, New York.

Rixner, J.J., Kraemer, S.R., and Smith, A.D. (1986). *Prefabricated vertical drains, Vol. 1*, Federal Highway Administration, Report No. FHWA-RD-86/168, Washington D.C.

Rujikiatkamjorn C. and Indraratna, B. (2009). "Design procedure for vertical drains considering a linear variation of lateral permeability within the smear zone", *Canadian Geotechnical Journal, 46*, 270-280.

Skempton (1954). “The Pore-Pressure Coefficients A and B”, *Geotechnique*, IV, 145-147.
Walker R. and Indraratna B. (2006). “Vertical drain consolidation with parabolic distribution of permeability in smear zone”, *Journal of Geotechnical and Geoenvironmental Engineering*, 132(2), 937-941.
Wang, X. S., & Jiao, J. J. (2004). “Analysis of soil consolidation by vertical drains with double porosity model”, *International Journal for Numerical and Analytical Methods in Geomechanics*, 28(14), 1385-1400.
Zhu, G., & Yin, J. H. (1998). “Consolidation of soil under depth-dependent ramp load”, *Canadian Geotechnical Journal*, 35(2), 344-350.
Zhu, G., & Yin, J. H. (2001). “Design charts for vertical drains considering construction time”, *Canadian Geotechnical Journal*, 38(5), 1142-1148.