The Application of a Novel Design Approach for Construction over soft soils: The Hybrid Undrained-Drained model

L'application d'une nouvelle méthode de conception pour des constructions sur sols mous: le modèle hybride non drainé - drainés

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ABSTRACT: The number of case histories of construction of high mechanically stabilized earth (MSE) berms over soft foundations, such as dredge disposal sites, is limited for obvious reasons: it is difficult to build without risking foundation instability. The purpose of this paper is to describe the challenges of designing and building embankments over extremely soft foundations using innovative design and construction techniques. The case history site for this paper is located in Wilmington, Delaware, where a 2400-m long, 21-m high MSE berm was completed in July 2010. The completion of this 1.5-million-cubic-meter berm represents a significant engineering achievement considering the size of the embankment and the 30-m deep layer of very soft soils (undrained shear strength as low as 10 kPa) over which the berm was constructed. The project was recently selected by the American Society of Civil Engineering among the five finalists for the 2012 Outstanding Civil Engineering Achievement Award.

RÉSUMÉ : Le nombre d'études de cas de construction de digues en terre compactée (MSE) sur des couches de fondations molles, comme le sont les sites de dépôts de dragage, est limité pour des raisons évidentes : il est difficile de construire sans risquer l'instabilité de la fondation. Le but de cet article est de décrire les défis de la conception et de la construction sur des fondations extrêmement molles en utilisant une conception innovante et des techniques de construction appropriées. Le site de cette étude de cas se trouve à Wilmington, dans le Delaware, où sur 2400 m de long une digue de 21 m de haut a été achevée en Juillet 2010. La réalisation de cette digue d'1,5 millions de mètres cubes représente une réalisation technique importante compte tenu de la taille du remblai et la couche de 30 m d'épaisseur des sols très mous (résistance au cisaillement non drainée voisine de 10 kPa) sur laquelle l'ouvrage a été construit. Le projet a été récemment choisi par la Société américaine de génie civil parmi les cinq finalistes pour le prix d'excellence 2012 des réalisations de génie civil.).

1 INTRODUCTION

This paper presents a case history of a reinforced earth structure constructed over extremely soft soils. The design and construction techniques developed for this project are applicable to many of the dredge management, levees, and other waterfront earthen structures.

Figure 1. Site Location Plan

The site where this case study was developed is the Cherry Island Landfill (CIL), located in Wilmington, Delaware, which was constructed over an area that was partly reclaimed from the Delaware River in the early 1900s (see Figure 1) and that had been used for many years as a dredged material disposal site for the U.S Army Corps of Engineers (USACE). Under the dredge layer lies an alluvium deposit with similar geotechnical characteristics as the overlying dredge. As a result, the subsurface at the site consist of unconsolidated, very soft, low permeability and extremely compressible materials with undrained shear strengths as low as 10 kPa and thickness ranging from 18 to 30m. Under the alluvial deposit lies the Columbia Formation (a 12m to 15m thick deposit of medium-to-coarse, medium-dense-to-dense sand).Because of the dredge layer's thicknessand permeability, it was initially estimated that it would take over 30 years for any excess pore pressures to dissipate.

To meet the growing demands for waste disposal in the Wilmington area, an additional 17 million cubic meters (mcm) of waste disposal capacity (i.e., 20 plus years capacity at the disposal rates at the time of design) was estimated. Because of the subsurface conditions, the additional airspace required to satisfy the needs of the state could not be obtained by increasing either the landfill sideslopes (8Horizontal:1Vertical in the original landfill layout) or the landfill height without compromising the overall foundation stability.Because the site is located at the confluence of the Delaware and Christina Rivers (See Figure 1), the potential for a horizontal expansion was limited;hence, the main alternative to obtain additional capacity at this facility was to expand it vertically.In order to obtain the required capacity, the only option available that would provide the additional disposal capacity was to build a 2400-m long and 21-m high mechanically stabilized earth (MSE) berm around the facility and place waste behind this structure. The preliminary feasibility study indicated that, in order to build the 21-m high MSE berm, the foundation shear strength needed to be improved from 10 kPa to 160 kPa as a minimum. The preliminary conceptual solution for achieving this strength gain was to use deep soil mixing (DSM), a technique that consists of mixing soil with cement. Because of the depth, length, and width of the soft soils that needed to be improved, the volume of soil that needed to be treated was approximately 2.5 mcy. 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).

2 PREFABRICATED VERTICAL DRAINS FEASIBILITY STUDY

Installation of prefabricated vertical drains (PVDs) is a cost-effective foundation improvement technique at sites where a surcharge load will be applied (e.g., an MSE berm). In general, PVDs are installed in soft soils to improve the drainage characteristics hence accelerating the dissipation of excess pore pressures generated during stage construction of embankments. The time it takes for pore pressures to dissipate depends upon the permeability of the dredge and the spacing between PVDs and it can be estimated using well known radial flow equations (e.g., Barron, 1948).

Initially, the use of PVDs to improve the foundation strength appeared unfeasible due to the massive weight of the proposed 21-m high MSE berm which was required to gain the needed airspace. Typically, the maximum height of an MSE berm on soft soils is dictated by the undrained shear strength of the underlying soft material. At the CIL site, the maximum height that could have been built using standard design techniques would have been on the order of 7.5-m (i.e., about 13.5 m shorter than required to achieve the target airspace of 17 million cubic meters).

Standard design techniques assume that when PVDs are installed in soft soils: (i) the excess pore pressures generated between PVDs during loading is uniform; and (ii) only undrained shear strength is mobilized during loading. The maximum excess pore pressures (U_{max}) generated after placement of a soil lift (i.e., 3 m for the CIL project) is estimated assuming that the soil lift is placed at once and it generates excess pore pressures (i.e., the pressure of the water stored within the dredge) approximately equal to the weight of the soil lift. Although it is recognized that excess pore pressures at the PVD location is nil and increases with radial distance from the PVD (Figure 2), it is typically assumed that excess pore pressures between PVDs are uniform and equal to U_{max}.

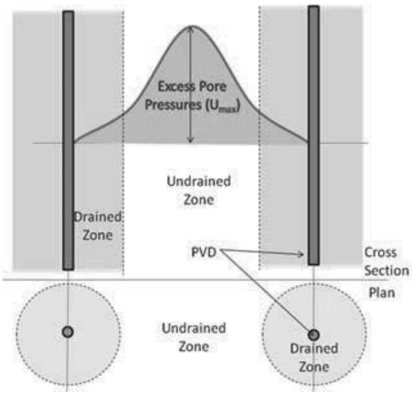

Figure 2. Pore Pressure Model

Because piezometers are located to monitor the maximum pore pressure, the radial variation is usually neglected. However, this conservative assumption made for computation and monitoring expedience not only neglects the fact that the excess pore pressures is not uniform but also does not take into consideration how PVDs change the dredge response to loading. In theory, drained parameters could be used to represent the shear strength of soft soils with PVDs if the applied loads (i.e., construction of the MSE berm) are imposed slowly enough to allow all excess pore pressures to dissipate as loading takes place. In practice, this could not be implemented because the rate of loading would need to be too slow to be feasible.

3 VIRTUAL SAND PILES: HYBRID DRAINED-UNDRAINED MODEL

The centerpiece of innovation for the design and construction of this massive MSE berm was the improvement of the weak dredge/alluvium foundation material using the concept of 'virtual sand piles', also described as the Hybrid Drained-Undrained (HDU) model (Espinoza et al., 2011).

The virtual sand pile concept is illustrated Figure 2. As shown in this figure, the closer the dredge is to the PVD the smaller the generated excess pore pressure and the faster that are dissipated. Hence, depending upon the speed of construction, it can be assumed that there are two distinct zones with different shear strength characteristics during loading: a drained zone, near the PVDs, and an undrained zone further 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 design. The development of the novel HDU design methodology for PVD design, to analyze the strength characteristics of the soft foundation soils during construction made the use of PVDs feasible for the CIL Project.

Subsequently, a more realistic model was developed to consider that: (i) the soils located closer to PVDs dissipate excess pore pressures generated during construction to more quickly than the soils located farther away from PVDs (Figure 2); and (ii) the rate of construction influences the maximum excess pore pressure that could be generated (i.e., pore pressures dissipate as the soil lift is placed). To simplify the model development, the rate of berm placement construction was assumed constant and equal to R_c. For each lift of soil, it was assumed that excess pore pressures starts to dissipate soon after it was placed (see Figure 3). Assuming an exponential decay function, the resulting excess pore pressure equation as a function of time is:

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

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

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

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

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

and$c_v$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. 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:

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

4 SELECTING THE DIAMETER OF THE VIRTUAL SAND PILE

Equations (1) through (5) were used to select the appropriate PVD spacing along with the corresponding rate of construction such that the soils near the PVDs would generate significantly smaller pore pressures that would allow to model the dredge around the PVD as a virtual sand pile. This meant that these soils could be considered to have a drained response during loading. The modified procedure consists of selecting the magnitude of excess pore pressure that would have negligible effect on MSE berm stability and then back-calculate the distance from the PVD that corresponds to this value. As a result, the dredge/alluvium materials 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.

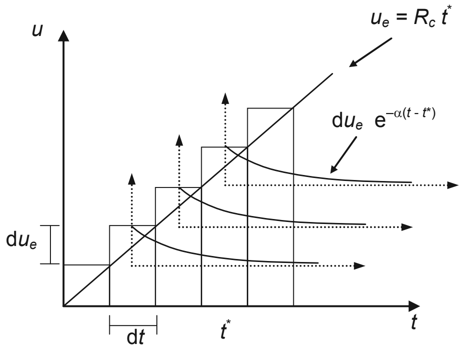

Figure 3. Pore Pressure Model

For the rate of construction (approximately 1 m of fill placed per week) and site-specific soils at this site (c_v = 0.0022 cm^2/s), approximately 1.8 million meters of prefabricated vertical drains (PVDs) were installed at a 1.5-m spacing to allow 90% dissipation of the excess pore-water pressures that were generated during construction of the overlying MSE berm within approximately 90 days. The berm was specified to be constructed 3-m high at a time at a rate of 1 m per week every 3 months (90 days). For a fill unit weight of 19.7 kN, the initial maximum pore pressure was 60 kPa (i.e., 6.1 m of water). For these conditions, it was estimated that if the average pore pressure generated within a certain distance from the PVD was about 15% of the maximum estimated excess pore pressure, the material could be considered drained. Based on this, it was estimated that the dredge/alluvium located within a 46-cm radius of the PVDs would be drained during each stage of MSE berm construction.

5 STABILITY ANALYSIS

The main purpose of the proposed methodology was to allow design engineers to use typical tools for analysis and design (i.e., limit equilibrium based methods). The HDU methodology expedited the stability analysis during the design stage as it was readily implemented using conventional limit equilibrium methods taking into consideration the soil strengths in the drained and undrained zones. In that way, hundred of different cross-sections were evaluated to optimize the design (i.e., minimize the MSE berm volume while still providing the same airspace for the same factor of safety). Accordingly, for slope stability analysis using limit equilibrium methods, the dredge/alluvium near the PVDs was considered to be drained with effective stress parameters given by ϕ' = 34º (obtained from triaxial tests), whereas the area further away from the PVDs was considered undrained with undrained parameters normalized with effective overburden given by S_u/σ' = 0.29 (parameters obtained from an extensive cone penetration tests and field vane shear tests). Figure 4 shows the soil stratigraphy during construction used inlimit equilibrium analysis. As shown in this figure, the soft dredge under the MSE berm is modeled as vertical strips of interchanging parameters (drained and undrained) to represent the HDU model. As shown in the model, the width of the soil columns does not need to represent the actual width of the virtual sand column (i.e., 0.92 m in diameter); only the ratio between drained to undrained areas needs to be taken into account. This can be simply estimated as:

$$A_r = \left(\frac{2r_s}{D_{pvd}}\right)^2 \times 100 \qquad (6)$$

where: D_{pvd} is the distance between PVDs and r_s is the radius of the virtual sand (1.5 m and 0.46 m for this project, respectively). Hence, the percentage of drained area respect to the total area for this project is:

$$A_r = \left(\frac{0.92}{1.5}\right)^2 \times 100 = 38\%$$

Hence, when modeling using limit equilibrium methods, as long as the vertical strips represent approximately 38% of total area with PVDs, the actual width of the vertical strips is immaterial. However, the number of vertical strips should be selected in a wayit does not have an influence on the failure mechanism. For instance, two vertical strips would not be appropriate. Another powerful application of the HDU model is that PVDs outside the loaded area also have a positive effect on stability as 38% dredge can be modeled using drained parameters, hence increasing the overall shear strength along the potential failure surface. As shown in Figure 4, the zone with PVDs extended beyond the toe of the MSE berm to increase the factor of safety against sliding during construction. Typical design procedures would only account for the shear strength increase due to the overburden pressure located above the PVDs, hence PVDs outside the MSE berm footprint would not be installed as it would not be considered in the analysis.

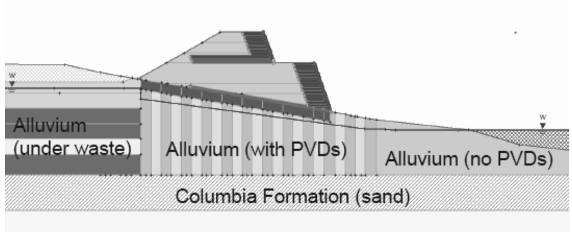

Figure 4. Limit Equilibrium Model of Enhanced Dredge with PVDs

In addition, to improve the stability of the MSE berm during construction, over 200.000 m^2 of high strength geotextile was installed at the base of the berm. The strength specified (1.170 kN/m) was one of the strongest materials ever manufactured by Tencate at the time of construction. The proposed solution for foundation improvement was significantly cheaper than DSM. The total cost of installing the PVDs including the high-strength geotextile was approximately \$11 million, thus resulting in significant savings from the initial design. Although more engineering was required for design and construction, the total cost was significantly less than the DSM alternative.

6 CONSTRUCTION MONITORING AND MODELING

In order to prevent unacceptably high pore pressures from developing, construction was conducted in stages and each stage of berm construction was limited to a 3-m thick lift followed by a 3-month pore pressure dissipation period, estimated initially. To monitor the performance of the foundation during the stages of construction, data was collected from a total of 85 geotechnical monitoring instruments along 17 lines spaced approximately 150 meters apart along the length of the MSE berm including 51 piezometers to measure pore pressures generated within the dredge/alluvium during loading at three different depths, 17 settlement sensors to measure the compressibility (i.e., vertical displacement) of the dredge/alluvium during berm construction, and 17 slope inclinometers at the toe of the berm to obtain a profile of horizontal displacement with depth during loading.

Although the use of limit equilibrium analysis expedites the analysis during design when dozens of cross sections are analyzed during the design stage, during construction, the recorded displacements (horizontal and vertical) could not be used in conjunction with limit equilibrium methods. Moreover, the monitoring data do not indicate the stability condition of the MSE berm directly.

Several finite element models (FEM) were developed for evaluating the stability of the MSE berm during construction using PLAXIS® software. The soil consolidation parameters obtained from laboratory and pilot tests were used as an initial model calibration. These parameters were adjusted during the initial 3-m lift placement and then used to predict pore pressures, lateral and vertical displacements during construction for subsequent lifts. The calibrated FEM models were used to closely monitor the construction of the MSE berm. After construction of each stage, the predicted horizontal and vertical displacements and excess pore water pressure were compared to the measured values at selected cross sections to verify whetherthe MSE berm was performing as expected. In addition, using ashear strength reduction method, factors of safety (FS) at each stage of construction was estimated by the FEM model. The procedure consisted of reducing the soil shear strength parameters by a factor in an iterative procedure until large displacements of the FEM model were observed. The ultimate factor achieved represented the factor of safety against instability using PLAXIS.Figure 5 shows an example of the comparison between the measured and predicted pore pressures.

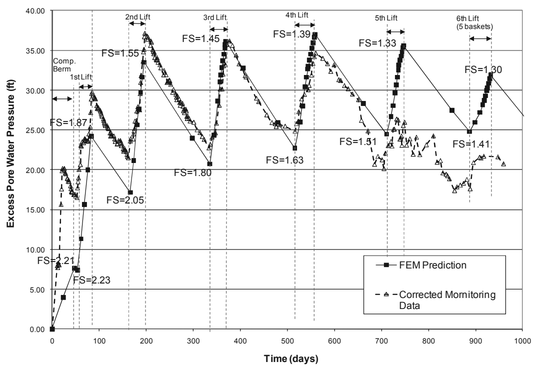

Figure 5. Example of monitoring results vs. predicted ones

The calculated FS at each stage of the construction are also shown in Figure 5.Although the construction schedule was initially established based upon estimated rates of pore pressure dissipation using the simplified drainage model described above, the schedule was constantly adjusted during construction based on the interpretation of the stability condition.

The MSE berm has undergone significant deformation.A settlement of approximately 4m was initially estimated.The recorded maximum vertical and horizontal displacements were approximately 4.2 m and 1.7 m, respectively.

7 BERM CONSTRUCTION

Construction of embankments designed using the HDU model requires close interaction between the designer, the contractor, and the owner, to allow timely geotechnical review and interpretation of monitoring data and communication of findings. During the initial stage of the project, it was found that the rate of pore pressure dissipation varied by sections of the MSE berm due to the localized subsurface geotechnical conditions. Because of the difference in consolidation rates, the contractor was required to alter its original construction sequence for the berm, moving back and forth between the different sections.By providing clarity to all parties on when subsequent berm lifts were likely to be feasible in any particular location, flexibility in construction task management and minimized disruption to the overall construction schedule could be achieved. With daily review of geotechnical data and frequent review of finite-element modeling output prepared for numerous berm cross sections, the designer was able to identify areas of construction on a "just-in-time" basis for the contractor to continue uninterrupted work. Eventually, the original plan of building a 3m-thick lifts over 600m to 900m length of the berm every 90 days evolved into construction of lifts in thicknesses as thin as 1m and/or berm lengths as short as 300m, which were patched together as review of geotechnical data would allow. The contractor's ability to reorganize its efforts to construct the various sections of the berm based on week-to-week feedback from the designer became a critical piece of the success of the project. In this way, by August 2010, 36 months after starting, MSE berm construction was completed. A detailed description of the berm construction is presented by Espinoza et al (2008) and (2011).

8 CONCLUSIONS

The completion of this 1.8 million cubic meters MSE berm (see picture below) represents a significant engineering achievement considering the size of the embankment, the deep layer of very soft soils over which the berm was constructed, and the amount of settlements during construction. The successful design and construction of a 2,400m long, 21m high MSE berm over extremely soft dredge using innovative design and construction techniques opens opportunities not only for extending the capacity of existing disposal facilities over dredge disposal sites but also for very cost effectively raising levees and dykes at critical locations prone to flooding.

Figure 6. View of Completed MSE berm

The use of PVDs at this site, which was shown to be feasible using the HDU methodology, resulted in savings of over $150 million when compared to conventional ground improvement techniques such as DSM.

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