Staged construction on soft clay: A design case history

Construction par étapes sur une argile molle: Un cas de conception antécédent

Makram El Sabbagh

AMEC Earth & Environmental, Burnaby, BC, Canada

Demetrious Koutsoftas

Demetrious Koutsoftas Geotechnical Consultants Inc., San Francisco, CA, USA

ABSTRACT

The Hamilton Army Airfield Wetlands Restoration project involved the construction of more than 8 kilometers of levees over deep deposits of soft high plastic clay, along the margins of San Francisco Bay. The levees will contain extensive hydraulic filling that is part of the wetlands restoration program. This paper addresses the staged construction design at the North-1 section of the levee system. Staged construction was used to improve the stability of the new proposed containment dike and wetlands fill, and reduce post-construction settlements. The Stress History And Normalized Soil Engineering Properties (SHANSEP) design procedure was used to develop the shear strength parameters that were necessary for the evaluation of stability at different stages during construction. The design process was based on extensive in-situ field vane testing and a comprehensive laboratory testing program. The use of vertical wick drains was evaluated for different areas along the levee. Various consolidation periods between construction stages were considered, and their impacts on shear strength increase assessed.

RÉSUMÉ

Le projet “Hamilton Army Airfield Wetlands Restoration” pour la restauration des terres marécageuses comprend la construction de digues de plus de 8 kilomètres de long sur de l’argile molle et plastique. Les digues seront construites à proximité de la baie de San Francisco et contiendront de vastes quantités de remblais placés par voie hydraulique qui formeront les terres marécageuses. Cet article aborde le côté conceptuel de la construction par étapes pour la section “North-1” du système de digues. La méthode de construction par étapes a été utilisée pour améliorer la stabilité de la nouvelle digue de confinement et des remblais, ainsi que pour réduire les tassements futurs suivant la construction. La méthode de conception “Stress History And Normalized Soil Engineering Properties” (SHANSEP) a été employée pour développer les paramètres de résistance au cisaillement nécessaires pour l’évaluation de la stabilité des digues à différentes étapes de la construction. La conception a été fondée sur un nombre élevé d’essais scissométriques en place et un programme considérable d’essais en laboratoire. Le recours aux drains verticaux préfabriqués a été évalué pour différentes zones le long de la digue. Diverses périodes de consolidation entre les étapes de construction ont été examinées, et leurs impacts sur l’accroît de la résistance au cisaillement évalués.

Keywords : Soft clay, staged construction, SHANSEP, stability, lateral deformations.

1 INTRODUCTION

The Hamilton Wetlands Project (HWP) includes construction of new levees, a separator berm, and modifications to existing levees to reinstate a system of seasonal and tidal wetlands. The levees will contain extensive hydraulic filling that is part of the restoration program. The HWP will be developed within the limits of the former Hamilton Army Airfield Base, located in the city of Novato, California (USA). This paper addresses the staged construction design at the North-1 (N-1) levee, one of seven levee segments of the HWP.

The subsurface conditions at the HWP consist of deep deposits of soft and compressible clay, locally known as Bay Mud, underlain by a sequence of stiff to very stiff alluvium over very dense/stiff colluvium, overlying sandstone bedrock. The N-1 levee is approximately 1,200 m long and generally follows the alignment of an existing 1.5-m high levee. As specified by the US Army Corps of Engineers (USACE), the new containment levee shall have a minimum crest width of 4.9 m, and the crest shall not be lower than elevation +3.4 m (NAVD 1988) 50 years following levee construction and placement of the hydraulic fill. The centerline of the existing levee is approximately 12 m away from the centerline of an existing 1.4-m-diameter sanitary sewer outfall pipe that was constructed in 1971.

The placement of new compacted fills directly on the soft Bay Mud would result in significant settlements and lateral deformations that could potentially damage the existing sewer. Therefore, staged construction was required to ensure stability of the new levee and to protect the integrity of the sanitary sewer.

The levee has been constructed following the design procedure presented in this paper and has been performing as expected. The staged construction program included extensive instrumentation consisting of piezometers, surface settlement markers, pipe displacement markers, and inclinometers, to verify the construction staging phases and monitor the performance of the levee. The objective of this paper is to present the methodology used in the stability evaluation and construction staging of the proposed N-1 levee.

2 SUBSURFACE CONDITIONS

The subsurface conditions at the section of interest are illustrated on Figure 1. The fill that forms the initial levee consists of soft Bay Mud, which was placed with minimal compactive effort over medium dense to dense sandy fill. The embankment fill is underlain by a desiccated medium stiff to stiff Bay Mud crust. Below the Bay Mud crust is a layer of recent Bay Mud, a highly plastic clay, slightly organic, soft, with in situ undrained shear strengths typically below 25 kPa. Below the Bay Mud is a layer of alluvial deposits underlain by colluvium.

The alluvium and colluvium generally consist of stiff to very stiff silty and sandy clays and dense to very dense silty sands. Sandstone bedrock underlies the colluvium.

Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering
M. Hamza et al. (Eds.)

doi:10.3233/978-1-60750-031-5-3491

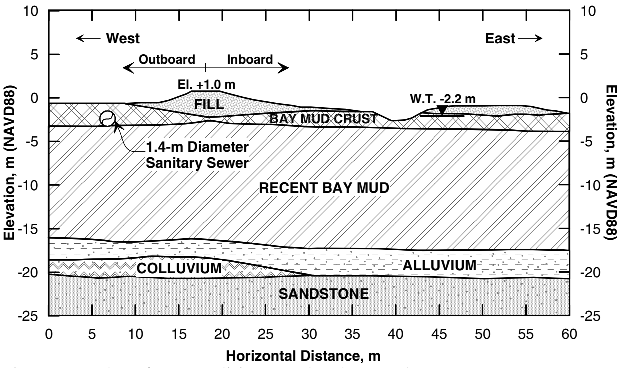

Figure 1. Subsurface conditions under the N-1 levee.

3 GEOTECHNICAL CHARACTERIZATION

The Bay Mud layer (i.e, crust and recent) is the most critical stratum to the settlement, lateral deformations (undrained and drained), and stability of the proposed levee. Therefore, it is essential to properly characterize the engineering properties of the Bay Mud layer.

In addition to exploratory boreholes, extensive in-situ field vane shear testing (using the Geonor H-10 apparatus) and cone penetration testing were performed to determine the undrained shear strength profiles of the Bay Mud for stability evaluations. The cone data were found to correlate well with the vane shear strength data, but for reason of brevity only the vane shear data is discussed in this paper. The Atterberg Limits in the Bay Mud and initial strength profiles in the free field and underneath the crest of the original levee are presented in Figure 2. The strength values shown in Figure 2 have been corrected to account for the effects of strain rate as recommended by Chandler (1988), using a correction factor of 0.8, for a corresponding average plasticity index in the Bay Mud of 50.

The undrained shear strength of the Bay Mud is a function of the in situ vertical effective stress and overconsolidation ratio. Fill placement causes excess pore pressures to build up. As excess pore pressures dissipate, there is a corresponding increase in vertical effective stresses. This translates into shear strength increase in the Bay Mud, and as a result the stability of the levee improves. Therefore the critical condition generally occurs during placement of new fill, before the mud consolidates and gains strength.

The Stress History and Normalized Soil Engineering Parameters (SHANSEP) design procedure developed by Ladd and Foott (1974) was used to evaluate the strength increase of the Bay Mud following consolidation, using the equation:

$$S_u = \sigma'_v \text{ x S x } (OCR)^m \tag{1}$$

where S_u is the undrained shear strength, σ'_v is the in-situ vertical effective stress, S is the normalized undrained shear strength ratio for normally consolidated soil, and m in an empirically determined exponent. Based on extensive laboratory testing conducted on the HWP and experience from other projects (Koutsoftas et al. 2000), the Bay Mud, like many other clays, is anisotropic with regards to its undrained strength characteristics. Moreover, the amount of strain that is required to mobilize the undrained shear strength also varies with the mode of shear. Based on strain compatibility (which accounts for the different rates of mobilization of shear strength) for embankment stability, Koutsoftas and Ladd (1985) have shown that the average undrained shear strength that can generally be mobilized along a failure surface is very close to the strength mobilized in direct simple shear. Furthermore, and for most practical purposes, it is generally recognized that the undrained shear strengths from direct simple shear are similar to those determined from field vane tests (following correction for strain rate effect), which are therefore equal to the average undrained shear strength along typical failure surfaces for embankments on soft clays. As a result, a comprehensive program of Ko-consolidated undrained direct simple shear tests (CKoU-DSS) was conducted at different overconsolidation ratios, and parameters S and m were determined as 0.26 and 0.85, respectively. Intrinsic to the SHANSEP method is an allowance for a certain amount of secondary compression to take place at the maximum applied stress during DSS testing. However, during staged construction, there is not enough time between stages to allow for secondary compression to take place. Accordingly, based on Ladd and De Groot (2003), a reduction factor of 0.90 was used to adjust the value of S to 0.235. Figure 2 presents undrained shear strengths estimated from SHANSEP for normally consolidated soils and shows good agreement with the field vane strengths within the zone where the clay is normally consolidated.

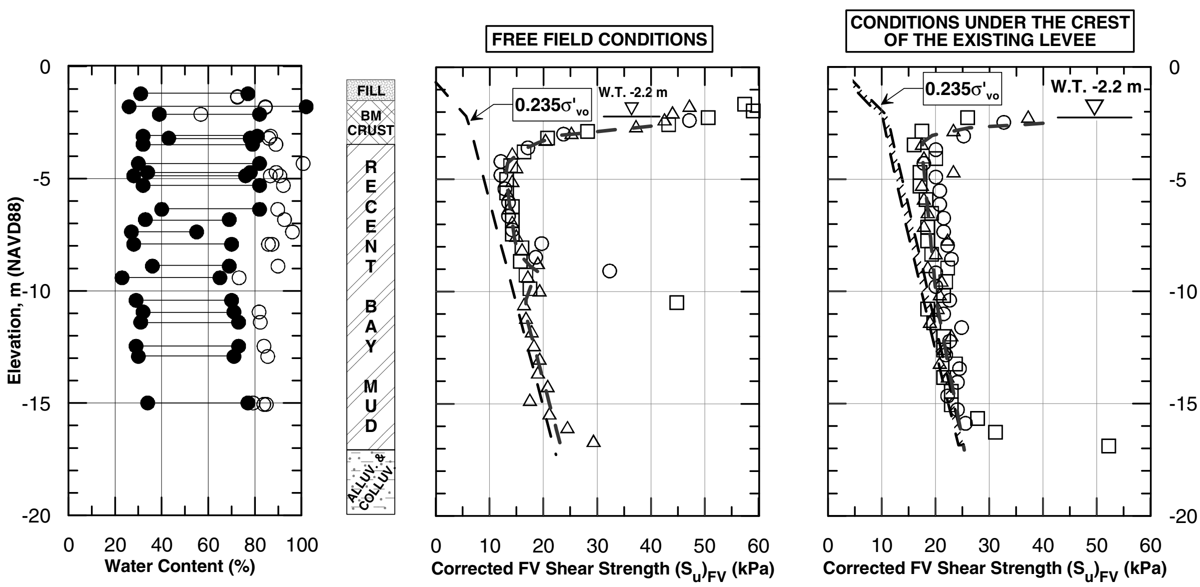

Figure 2. Atterberg Limits, and undrained shear strength profiles for the Bay Mud at the free field and under the crest of the existing levee.

At each stage of proposed fill placement, the overconsolidation ratio was re-evaluated from the in-situ vertical effective stress (determined from the total vertical stress and the excess pore pressure profile) and the maximum past pressure determined during the subsurface exploration phase. If the clay is overconsolidated, the initial corrected strengths determined from field vane tests (see Figure 2) were used. When the clay consolidates to an effective stress that exceeds the initial maximum past pressure, the clay becomes normally consolidated, and equation 1 simplifies to:

$$S_u = \sigma'_v \text{ x } S = 0.235 \text{ x } \sigma'_v \qquad (2)$$

4 STABILITY ANALYSIS

Due to the presence of the sanitary sewer pipe in close vicinity outboard of the existing and new levees, a minimum factor of safety of 1.5 was deemed necessary to keep lateral deformations around the sewer within acceptable limits. The factor of safety of 1.5 was selected based on undrained analyses following the method proposed by D'Appolonia et al. (1971), to minimize the short-term undrained settlements. Ladd (1991) had shown that the lateral deformations are directly related to embankment settlements, and therefore by minimizing the undrained settlements, it is an appropriate method to control also undrained lateral deformations without having to make detailed estimates of lateral deformations which would otherwise require sophisticated numerical analyses. In this case, undrained settlements of 0.15 m to 0.20 m were calculated under the proposed containment dike. As proposed by Tavenas and Leroueil (1980), maximum undrained lateral deformations under the toe of the dike were anticipated to be of the same order of magnitude, around 0.15 m to 0.20 m. Given the distance of 15 m to 20 m between the toe of the proposed dike and the sewer pipe, it was estimated that the lateral displacements of the pipe would be in the order of 0.05 m to 0.10 m. If staged construction were not used, considerably larger undrained lateral displacements of the sewer pipe would have been expected.

Undrained strength analyses were conducted to evaluate the stability of the proposed embankment. The results of the initial stability analyses are presented on Figure 3 as factors of safety versus embankment crest elevations.

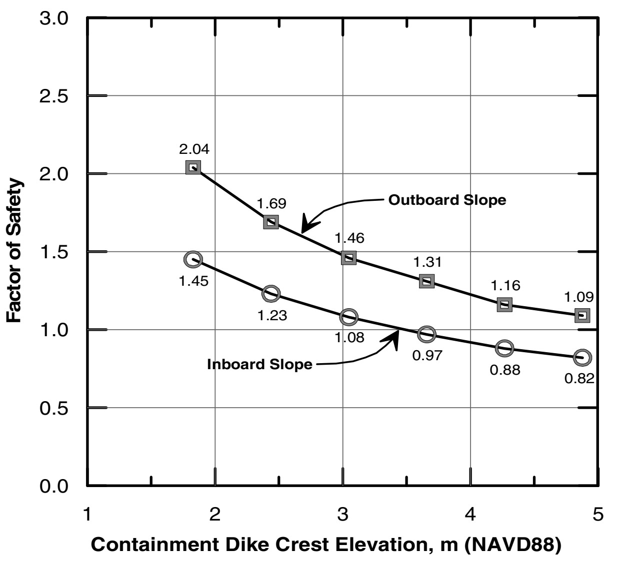

Figure 3. Summary of the results of short-term stability for the containment dyke.

In order to meet the required factor of safety at the outboard side, the maximum crest elevation of the new levee needed to be limited to +3.05 m during the first stage of filling. Stability of the inboard slope was not considered critical given that hydraulic filling during levee construction can be placed at the inboard side, which will buttress the levee and increase the factor of safety. Since consolidation of soft clays is a very slow process, and given the Bay Mud thickness of approximately 14 m, it was concluded that vertical drains extending to the bottom of the Bay Mud were essential to allow construction of the new levee over a reasonable period of time, maintain stability, and limit undrained deformations and their potential impacts on the adjacent sewer.

The vertical drains would be connected to collection drainage mats placed under the levee. Prefabricated vertical drains that are 102 mm wide and 3.8 mm thick, and which are arranged in a triangular grid spaced at 1.2 meters on-centers, were considered.

Two different alternatives were considered for the analyses that were performed to evaluate staged construction: the first one involving a delay period of 6 months between construction stages, and the second one involving 12 months delay between stages.

In order to meet the minimum required crest elevation criterion of +3.4 m 50 years after completion of construction while achieving minimum acceptable factors of safety at the outboard slope, the following stages of construction were identified: (i) construct the levee to elevation +3.05 m while placing hydraulic fill at the inboard side of the levee (stage I); (ii) allow consolidation for six months to one year; (iii) raise the containment dike by 2.0 m (stage II); (iv) allow consolidation for six months to one year; (v) raise the containment dike by 0.6 m (stage III).

There are generally two important factors that influence the stability of the levee during staged construction: first is the strength gain of the clay resulting from consolidation, and second is the change in geometry and submergence of the levee as a result of consolidation settlements. In order to account for strength gain during construction, the incremental vertical stresses in the clay resulting from fill placement were evaluated using finite element program PLAXIS, and the excess pore water pressures estimated from the results of consolidation analyses. The vertical effective stresses in the clay were determined by subtracting the excess pore pressures from the final effective stresses, which are equal to the initial effective stresses plus the incremental vertical stresses resulting from the placed fill. Based on the estimated vertical effective stresses, the SHANSEP method (equations 1 and 2) was applied to evaluate the new undrained shear strength profiles in the clay.

Results of the staged construction stability analyses are presented in Figures 4 and 5, which correspond to consolidation periods between stages of 6 months and 1 year, respectively.

If the effects of consolidation settlements and strength gain are not accounted for, the factors of safety at the end of stage I filling are 1.46 and 1.08 for the outboard and the inboard slopes, respectively. If the effects of consolidation during construction are accounted for, and assuming a stage I duration of 3 months (which corresponds to an average degree of consolidation U of 27%), the factors of safety at the end of stage I will be 1.62 and 1.21 for the outboard and inboard slopes, respectively.

At the end of the 6-month consolidation period the average degree of consolidation U under the levee is 74%, which results in a factor of safety for the outboard slope of 2.18 (see Figure 4). With the placement of the stage II fill, the factor of safety decreases to 1.56. After a 6-month waiting period, the average degrees of consolidation under the stage I and II fills are 93% and 74%, respectively, with a resulting factor of safety for the outboard slope of 1.89. The addition of the final 0.6 m of fill reduces the factor of safety to 1.61. Figure 5 presents similar stability analysis results for the 1-year delay option between stages.

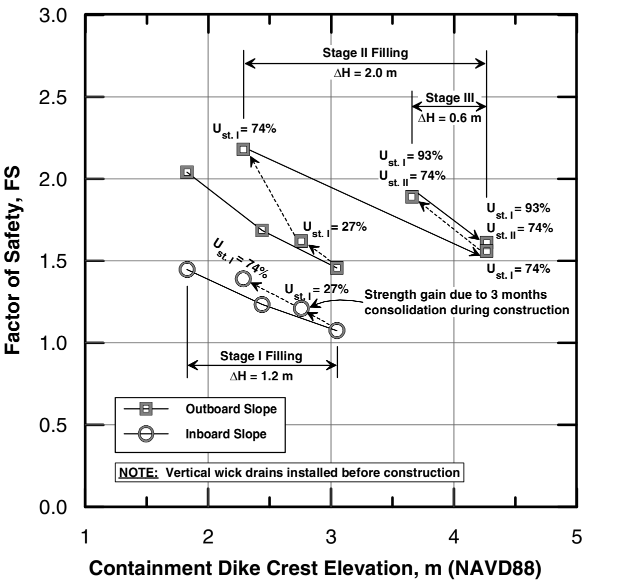

Figure 4. Results of stability analysis with staged construction with 6 months delay between stages.

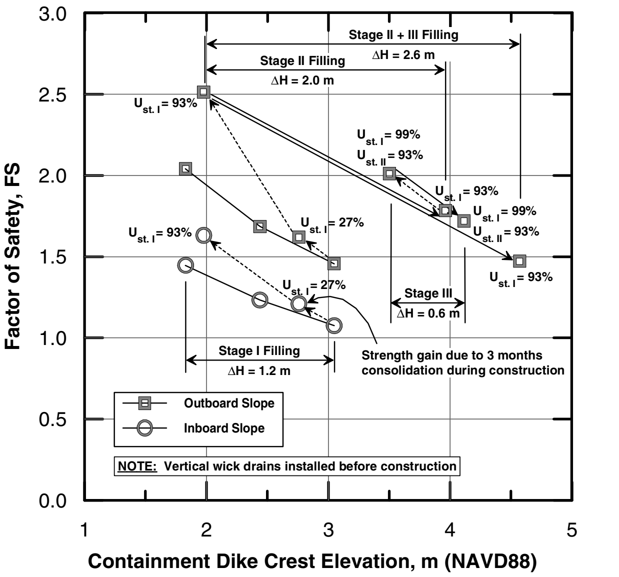

Figure 5. Results of stability analysis with staged construction with 1 year delay between stages.

5 SUMMARY AND CONCLUSIONS

Staged construction is a valuable technique that enables the gradual construction of embankments on soft clays, which could otherwise not be possible. It helps control undrained deformations within tolerable limits, by maintaining high factors of safety, which is the key to minimizing undrained deformations (D'Appolonia et al. 1971) during construction. The controlled rate of loading enables the clay to gain strength via consolidation, which is essential to maintain high factors of safety at each construction stage. The SHANSEP procedure can be used to evaluate the undrained shear strength profiles in the clay based on the initial stress history and the vertical effective stresses in the soil during construction. Some of the key recommendations for the design of a staged construction program are:

1. Accurately evaluate the in-situ shear strength profile of the clay through extensive in-situ field vane testing.
2. Determine the stress history of the clay deposit from a comprehensive program of laboratory consolidation tests.
3. Perform a suite of CKoU-DSS tests at different overconsolidation ratios on high-quality samples to check that the soil follows normalized behavior, which is the key assumption of SHANSEP.
4. Determine the rate of consolidation, and changes to that rate with the progress of consolidation.
5. Accurately compute the magnitude of consolidation settlements for embankments for each construction stage, given the impact settlements have on the geometry and stability of such structures.
6. Install appropriate instrumentation in the clay and monitor the performance during construction.

ACKNOWLEDGEMENTS

Work presented in this paper was performed for the USACE under subcontract with URS Corporation while the authors were employed in Arup's San Francisco office.

REFERENCES

Chandler, R.J. 1988. The In-Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane: State of the Art Paper, ASTM, STP 1014, Vane Shear Strength Testing in Soils, Field, and Laboratory Studies, Conshohoken, PA, pp. 13 44.

D'Appolonia, D.J., Poulos, H.G. and Ladd, C.C. 1971. Initial Settlement of Structures on Clay, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, SM10, pp. 1359-1377.

Koutsoftas, D.C. and Ladd, C.C. 1985. Design Strengths for an Offshore Clay, Journal of Geotechnical Engineering, ASCE, Vol. 111, No. 3, pp. 337 355.

Koutsoftas, D.C., Frobenius, P., Wu, C.L., Meyersohn, W.D. and Kulesza, R. 2000. Deformations During Construction of MUNI Metro Turnback Project, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 4, pp. 344 359.

Ladd, C.C. and Foott, R. 1974. New Design Procedure for Stability of Soft Clays, Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. 7, pp. 763 786.

Ladd, C.C. 1991. Stability Evaluation During Staged Construction, Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 4, pp. 540 615.

Ladd, C.C. and De Groot, J.D. 2003. Recommended Practice for Soft Ground Characterization. Arthur Casagrande Lecture, 12th Pan American Conference on Soil Mechanics and Geotechnical Engineering, MIT, June 2003, 55 pp.

Tavenas, F. and Leroueil, S. 1980. The Behavior of Embankments on Clay Foundations, Canadian Geotechnical Journal, Vol. 17, No. 2, pp. 236-260.