An Often-Overlooked Approach for Slope Stability of Tailings Dams Built Using Centerline and Downstream Construction

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Abstract

Many tailings dams are founded on overconsolidated clays of stiff to very stiff consistency, which in general tend to provide a relatively suitable foundation for these earthen structures. The authors have found in multiple dam safety reviews that there is a tendency in practice to overlook the nature and impact that the stress changes have on the characteristics of these clays. On one hand, there are cases in which the stiff clays are solely characterized with drained shear strength because they are seen as "stiff clays" and thus the drained shear strength should govern stability. On the other hand, there are cases in which the possibility of mobilizing the undrained shear strength is acknowledged but it is assumed that the undrained shear strength ratio initially determined remains constant over time and does not change with increasing effective stresses as the dams are raised.

The paper presents lessons learned on characterization of foundations on clays of stiff to very stiff consistency through several dam safety reviews as well as practical examples and steps associated with the SHANSEP technique to be utilized in design of tailings dams using the centerline and downstream construction methods.

Introduction

When tailings dams are founded on overconsolidated clays of stiff to very stiff consistency, these clays tend to provide a relatively suitable foundation for these earthen structures. However, as tailings dams are raised to provide additional storage capacity, the stress history of these foundation clays changes and so does their behavior and undrained shear strength. The authors have found that in practice there is a tendency to overlook the nature and impact that the stress changes have on the characteristics of these clays. On one hand, there are cases in which the stiff clays are solely characterized with drained shear strength because they are seen as "stiff clays" and thus the drained shear strength should govern stability. This is initially true but upon loading and subsequent consolidation, this behavior changes. On the other hand, there are cases in which the possibility of mobilizing the undrained shear strength is acknowledged but it is assumed that the undrained shear strength ratio (s_u/σ'_{vc}, or USSR) initially determined on the overconsolidated specimens remains constant over time and does not change with increasing effective stresses as the dams are raised. In reality, the USSR of the clay foundation will change as a function of the stresses imposed by the dam (i.e., overconsolidation ratio, or OCR), which are variable under the dam in terms of stress history, pore pressures, applied loads, and consolidation. This is particularly relevant for centerline and downstream construction, which are becoming more typical in the industry for slurried tailings storage. There is a relatively simple but powerful technique to incorporate these stress history and strength effects in slope stability analyses, which is often overlooked in practice.

The authors have found through multiple dam safety reviews that in practice there is a tendency to overlook the nature and impact that the stress changes have on the characteristics of these clays. This complex impact of stress history on undrained shear strength is best captured by the SHANSEP approach. The SHANSEP approach is briefly described subsequently and followed with an applicable example to illustrate its benefits and implications.

SHANSEP Concept

SHANSEP, which stands for Stress History and Normalized Soil Engineering Properties, is a concept introduced by Ladd and Foott (1974) with follow-up research by Ladd (1991). This approach is based on the experimental observation that the shear strength of many soils can be normalized with respect to the vertical consolidation stress. In other words, the method is based on the concept that, for a given failure mode such as triaxial compression (TC) or direct simple shear (DSS), the undrained stress-strain-strength behavior of most "ordinary" clays is governed by the stress history of the clay. The method assumes that these soils exhibit normalized behavior. Accounting for stress history in the laboratory is done by using mechanical overconsolidation to represent all preconsolidation mechanisms, thus the procedure explicitly requires the stress history profile for the clay to be evaluated.

The increase in undrained shear strength ratio s_u/σ'_{vc} of clays with increasing OCR can be modeled by the following equation:

$$s_u/\sigma'_{vc} = S(OCR)^m$$

where:

s_u = undrained shear strength

σ'_{vc} = effective vertical consolidation stress

$S = s_u/\sigma'_{vc}$ at OCR = 1

OCR = overconsolidation ratio

m = empirical factor

Obtaining this relationship in the laboratory first involves determining the stress history of the soil in the form of the preconsolidation stress σ'_p , which can be obtained from laboratory consolidation tests or through field methods such as dilatometer or CPT. Specimens are then often isotropically consolidated to 1.5, 2.5, and 4.0 times σ'_p so they can be sheared in the normally consolidated range at OCR = 1 while other specimens are often isotropically consolidated to 2.5 times σ'_p and then unloaded to nominal OCR values of 2, 4, and 6 so they can be sheared in the overconsolidated range (Ladd & DeGroot, 2003). The resulting USSR values can then be plotted against OCR on a log scale to obtain the empirical factor m.

Numerical Modelling

A generic, two-dimensional numerical modelling exercise was carried out in FLAC2D to demonstrate the effect of a change in stress history for a "very stiff" foundation clay under staged construction of a downstream tailings dam. The modelling is intended to show that the foundation clay gains undrained shear strength due to material loading and subsequent consolidation during and after each dam raise, but also that the OCR and the corresponding USSR decrease as the dam is raised. The OCR can decrease to the point where the material transitions from a very stiff, dilative clay to a lightly overconsolidated or even normally consolidated, brittle clay.

Model Geometry

The model geometry comprises a downstream tailings dam constructed in six phases on a simple clay foundation as shown in Figure 1.

The foundation clay is 15 meters thick with a horizontal native ground surface. The clay is underlain by 15-meter-thick bedrock, which is considered to be impenetrable, with a horizontal contact between them.

The embankment comprises a 30-meter-high started dam called Phase 1, which includes a 3-meter-thick drainage blanket extending 60 meters under the starter dam from the dam toe. The starter dam includes upstream and downstream slopes of 2.5H:1V, with 5-meter-wide benches every 10 meters vertically on the downstream face. Note that the bottom 20 meters on the downstream face does not include a bench. The crest width is 10 meters.

The five additional phases, referred to as Phases 2 through 6, are each 10-meter raises with the same slope and crest geometry as Phase 1. The ultimate dam height is 80 meters with an overall downstream slope of 2.875H:1V.

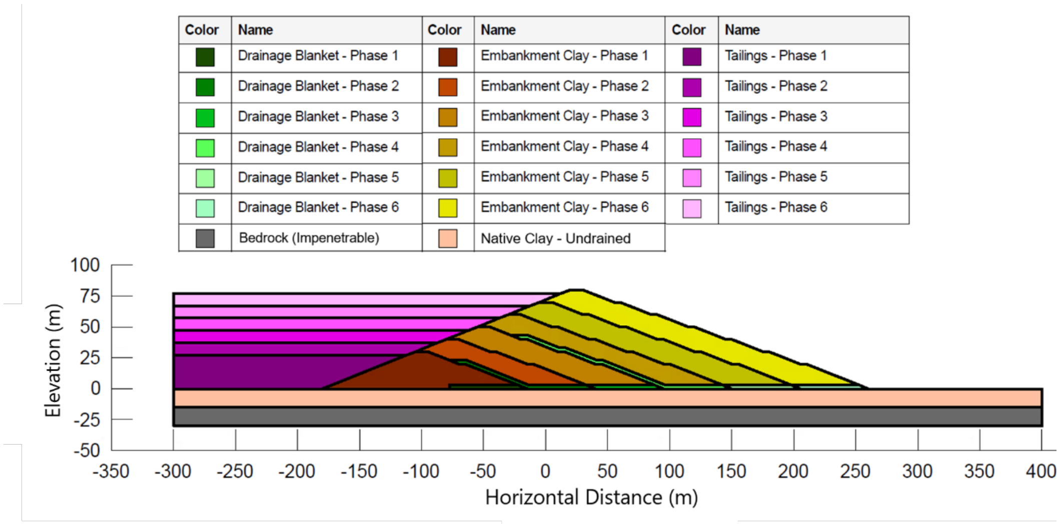

Figure 1: Model Geometry of Generic Downstream Tailings Dam

The drainage blanket is 3 meters thick for Phases 2 through 6 to reduce contrast in zone size and thus reduce model run time. The drainage blanket continuously underlies the embankment, with chimney drains covering the downstream faces of Phases 1 and 3 (extended to the first bench below the crest) to intercept seepage.

The tailings on the upstream side were modelled with a freeboard of 3 meters below the dam crest of each respective phase. The upstream pond is assumed to be at the tailings surface.

Methodology

After the Phase 1 starter dam was placed, the model was run to end-of-primary consolidation such that all excess pore pressures were dissipated. For each phase subsequent raise, the model was run coupled, meaning that mechanical and flow calculations were performed simultaneously to simulate stress increase during embankment filling and consolidation during and after embankment filling. Each phase was gradually raised over a period of 1 year by ramping up the embankment density in 10 percent increments. After construction was completed for the individual phase, the model was run for an additional 2 years to allow for consolidation and dissipation of excess porewater pressures. Tailings deposition and the upstream pond were then placed against the upstream face of the dam, with the model brought to end-of-primary conditions. These steps were then repeated for subsequent raises.

Material Parameters

The material parameters are shown in Table 1. The undrained shear strength s_u was estimated at the bottom of the clay foundation layer. The pre-consolidation stress was assumed to be about 890 kPa, which corresponds to an OCR of approximately 6 at the bottom of the clay layer, assuming a water table at the native ground surface with hydrostatic pore pressures.

Table 1: Material Properties

Property	Bedrock	Native Clay	Drainage Blanket	Embankment Clay	Tailings
Dry Unit Weight, γ_{dry} (kN/m^3)	25.0	17.5	18.0	17.5	14.5
Sat. Unit Weight, γ_{sat} (kN/m^3)	26.0	21.0	21.0	21.0	19.5
Young's Modulus, E (MPa)	100	20	30	20	1
Poisson's Ratio, ν	0.20	0.30	0.30	0.30	0.25
Horizontal Perm., k_H (m/s)	$1.0\text{x}10^{-6}$	$1.0\text{x}10^{-8}$	$2.5\text{x}10^{-5}$	$1.0\text{x}10^{-8}$	$1.0\text{x}10^{-6}$
Anisotropy, k_H/k_V	1.0	1.0	1.0	1.0	1.0
Cohesion, c' (MPa)	1.5	0	0	0	0
Friction Angle, ϕ' (deg.)	38.0	28.0	35.0	28.0	25.0
USSR[1]	--	$0.22\times OCR^{0.8}$	--	--	0.18

[1] USSR is yield undrained shear strength ratio; USSR = s_u/σ'_{vc}. Note that some researchers (e.g., Brown & Gillani, 2016) advocate capping the undrained envelope with the drained envelope at low confining stresses because negative porewater pressures can dissipate quickly in the field. This capping was not done in this exercise because the rate at which negative porewater pressures might dissipate, relative to positive porewater pressures, is unknown.

Software

The phased construction of the downstream tailings dam presented herein was performed using the FLAC2D (Fast Lagrangian Analysis of Continua) version 8.1 software program developed by Itasca Consulting Group, Inc., which uses the finite difference method to solve nonlinear stress-strain systems. In this method, materials are represented by zones that form a grid to fit the geometry of the cross-section being analyzed. Each element behaves according to the user-prescribed linear or nonlinear stress-strain-strength model (i.e., constitutive model) in response to the applied forces and boundary conditions.

FLAC simulates the consolidation process (i.e., deformation and dissipation of the excess porewater pressures being generated from embankment fill placement) in a realistic time frame by coupling the mechanical response and the porewater flow regime. As such, the model allows dam stability during construction, when excess porewater pressures are present in the native soils and consolidation is underway, to be analyzed.

Results And Discussion

During modelling, six monitoring points were established as shown in Figure 2. Three were set near the bottom of the foundation clay layer while three were in the middle of the clay layer. The x-coordinates were as follows: -105 m (under Phase 1 dam crest); -25 m (near Phase 1 dam toe); and 25 m (near Phase 2 dam toe, or under the ultimate Phase 6 dam crest).

Figure 3 shows the porewater pressure development during and after construction of Phase 2. The increase in porewater pressure during each 10 percent increment can be seen, with subsequent consolidation before the next increment is placed. This discretization is a modelling simplification, but the overall general increase in pore pressures throughout the 1 year of Phase 2 construction can be seen.

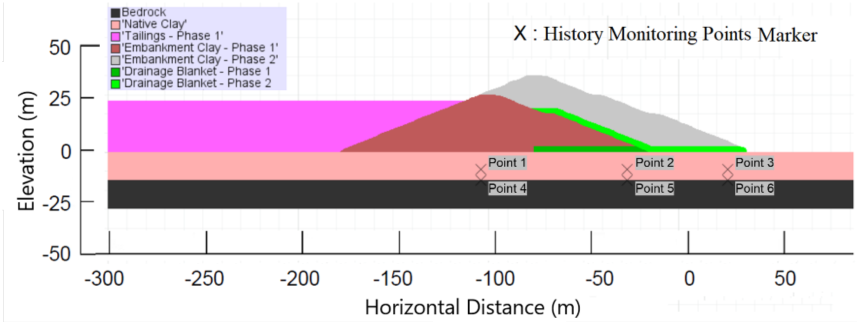

Figure 2: Monitoring point locations (with Phase 2 dam geometry)

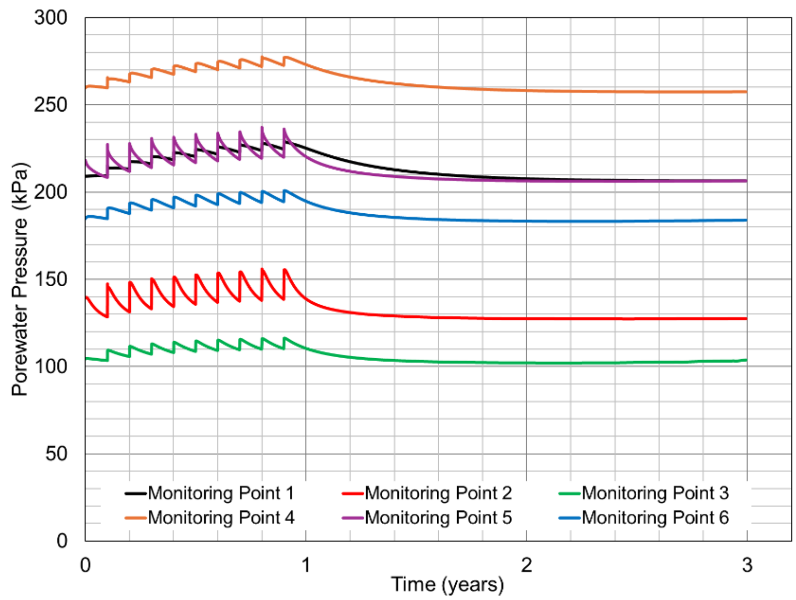

Figure 3: Porewater pressure histories in foundation clay during and after Phase 2 construction

Figure 4 is very similar to Figure 3 but it shows porewater pressure development for Phases 2 through 6. The same discretization can be seen during embankment filling for each phase, along with the 2 years of consolidation between each raise. It should be noted that an overall increase in porewater pressure can be seen even though end-of-primary conditions are generally reached after each raise. This is because the hydraulic boundary condition is increasing on the upstream face of the dam as the tailings and pond increase with the dam crest. In other words, excess pore pressure dissipation and accompanying effective stress increases are occurring during and after construction of the various raises even though steady-state pore pressures are increasing throughout the embankment construction as the pond rises.

Figure 5 shows the OCR history at the same six monitoring points for Phases 2 through 6. It can be seen that initial OCR values after Phase 1 range from about 2 to 15. The OCR values of 2 are at Monitoring

Points 1 and 4, which are located directly under the crest of the starter dam (Phase 1) such that they have already undergone some loading due to the end-of-primary conditions prior to Phase 2 loading. The OCR values of 15 and 8.5 at Monitoring Points 3 and 6, respectively, are consistent with a "very stiff" clay and represent the foundation clay in its native, pre-construction state. The OCR values then decrease with additional raises until all six fall below 2.0 during Phase 3 and reach 1.0 during Phase 5. During Phase 3, the material likely became contractive at all six monitoring locations and during Phase 5, the material became completely normally consolidated.

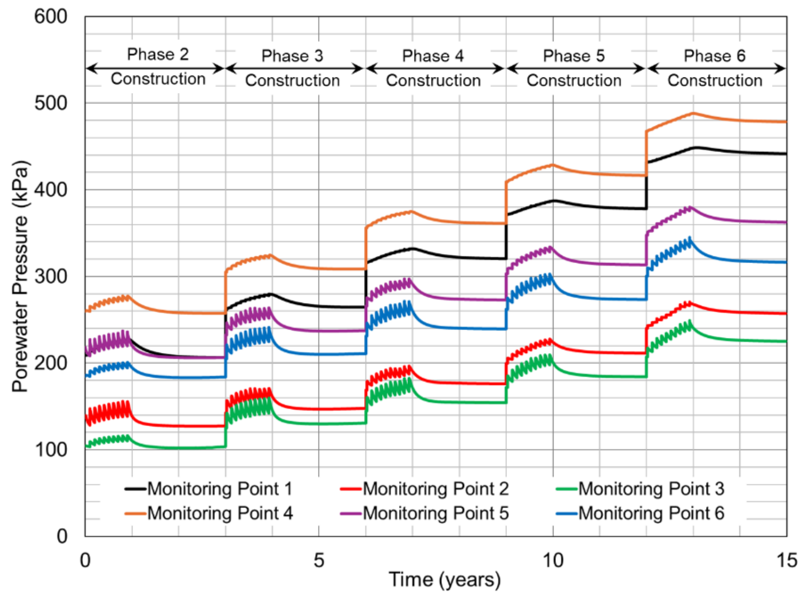

Figure 4: Porewater pressure histories in foundation clay throughout construction (Phases 2-6)

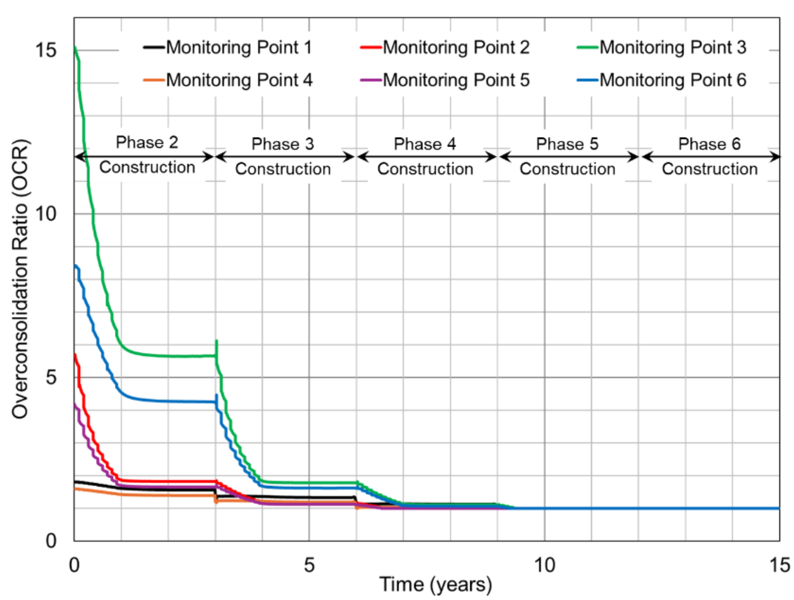

Figure 5: History of OCR at monitoring points throughout the phased construction

Given that the OCR is known throughout the construction process (Phases 2 through 6), the USSR can also be plotted as a function of the SHANSEP relationship, as shown in Figure 6. The initial USSR values at the end of Phase 1 range from about 0.3 to 1.9 with the low USSR values being under the starter dam (Phase 1) crest and the high USSR values being well downstream of the Phase 1 toe. During continued construction, the OCR values then decrease with additional raises until all six monitoring points fall below 0.40 during Phase 3 and reach 0.22 during Phase 5, which is consistent with the USSR of a completely normally consolidated clay.

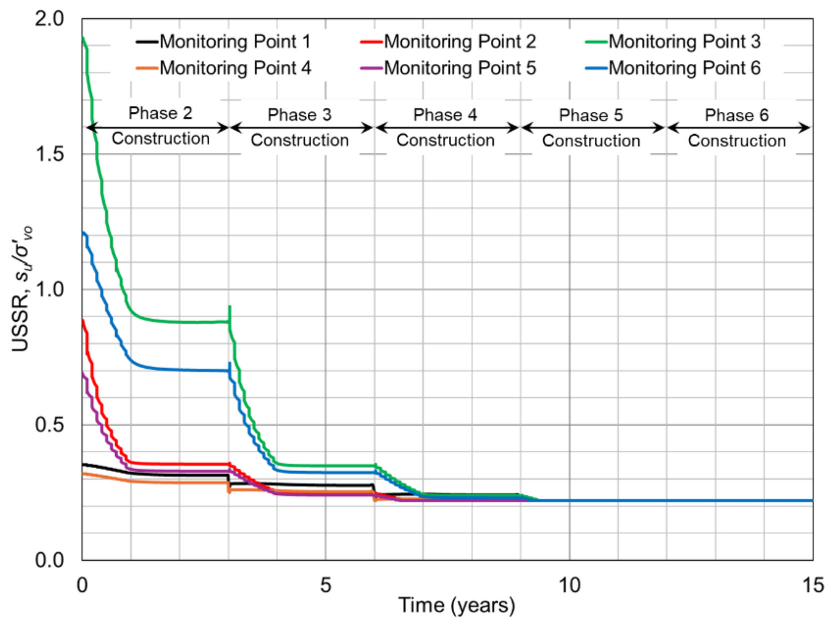

Figure 6: History of USSR at monitoring points throughout the phased construction

Figure 7 shows the evolution of s_u for all six phases of construction. It can be seen that s_u starts at 137 kPa at the bottom of the foundation clay downstream of the starter dam (Phase 1), which classifies the material as "very stiff" according to Terzaghi & Peck (1967). With subsequent raises, the s_u only increases with additional embankment filling and associated consolidation until it reaches about 325 kPa under the upstream portion of the ultimate embankment, with the load from the tailings contributing to this maximum s_u. However, as noted in Figures 5 and 6, the OCR and the normalized undrained shear strength ratio, or USSR, are decreasing, which leads to contractive, brittle behavior in which the undrained shear strength of the clay will likely be mobilized rather than the drained shear strength.

For each phase of construction, the factor of safety was computed for three cases: ESSA, USSA (EOP), and USSA (EOC) as shown in Table 2. ESSA stands for Effective Stress Stability Analysis; USSA stands for Undrained Strength Stability Analysis; EOP stands for end-of-primary in which excess load-induced porewater pressures from the current raise are dissipated; EOC stands for end-of-construction in which excess load-induced porewater pressures from the current raise are fully engaged. It should be noted that the ESSA factors of safety remain relatively constant throughout all phases of construction, as expected. The USSA cases, on the other hand, show a consistent drop in the computed factor of safety as the USSR in the foundation clay continues to decrease with increasing load from successive embankment filling.

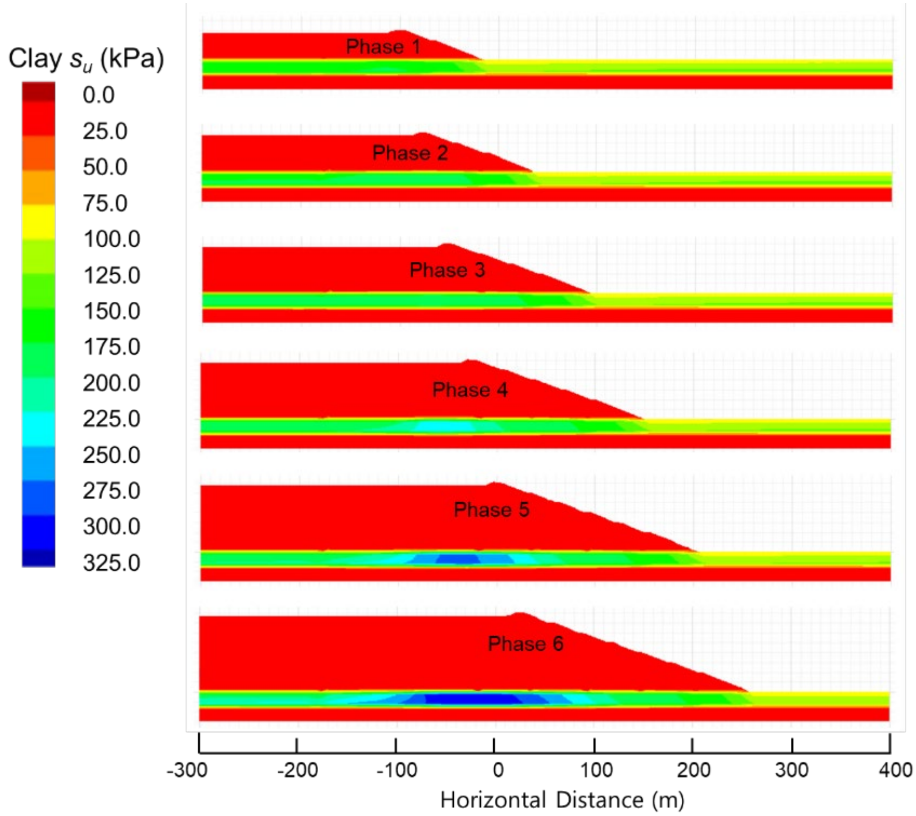

Figure 7: Evolution of s_u in foundation clay throughout the phased construction

Table 2: Factor of Safety Summary

Phase	ESSA	USSA-EOP	USSA-EOC
Phase 1 (Initial)	1.37	1.58	--
Phase 2	1.36	1.44	1.35
Phase 3	1.37	1.37	1.24
Phase 4	1.37	1.32	1.15
Phase 5	1.37	1.23	1.11
Phase 6	1.36	1.20	1.07

Figure 8 shows the critical slip surface for USSA-EOP conditions at Phase 5, with a computed factor of safety of 1.23. A very large composite, or block, potential failure surface can be seen extending to the bottom of the foundation clay, indicating that the clay that is becoming normally consolidated at the base of the foundation clay is governing stability.

Figure 9 shows the critical slip surface for USSA-EOC conditions at Phase 6. The computed factor of safety is 1.07, and this represents the most critical undrained case at the ultimate dam height. Obviously, a factor of safety of 1.07 would not be considered acceptable for a tailings dam, especially when a brittle failure mode would be predicted.

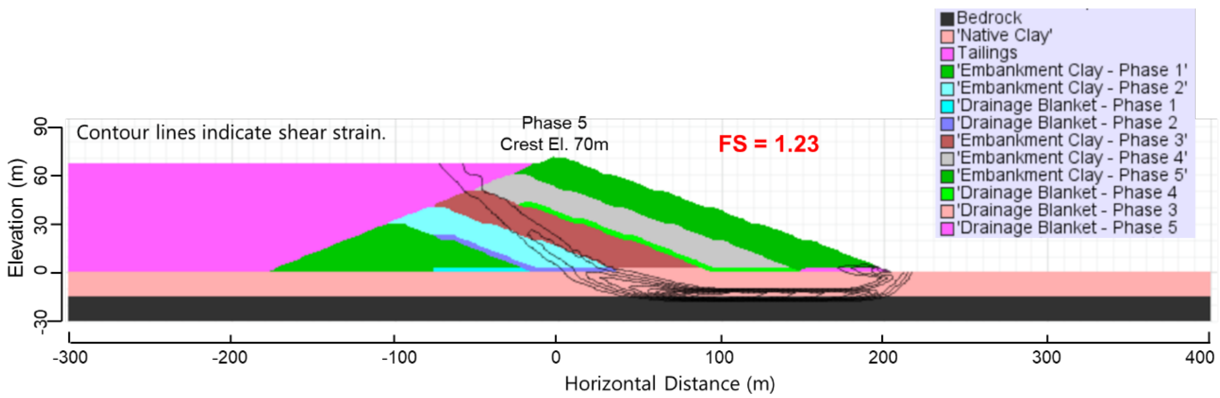

Figure 8: Typical critical slip surface for USSA-EOP conditions (Phase 5 shown)

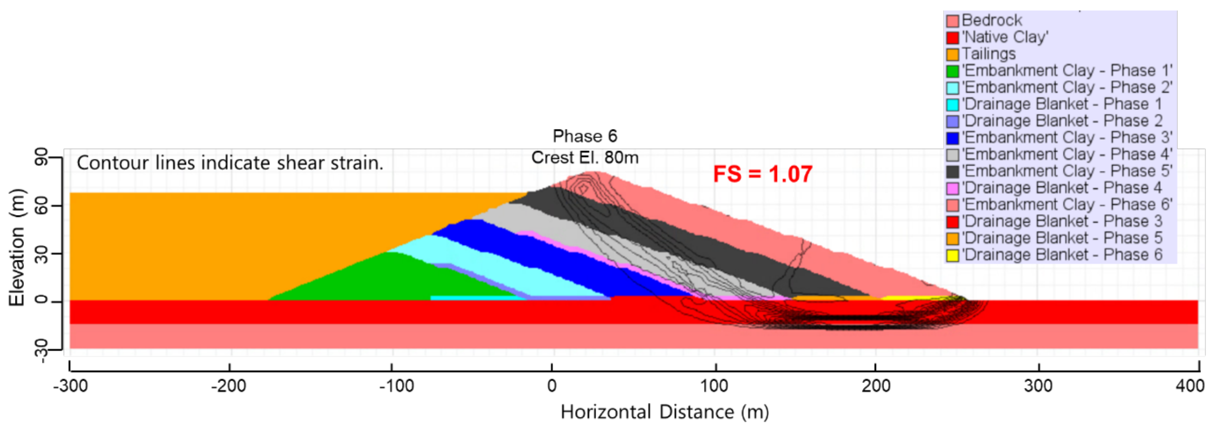

Figure 9: Critical slip surface for USSA-EOC conditions of Phase 6 dam raise

As a point of contrast to the USSA figures related to the SHANSEP phenomenon included above, Figure 10 shows the critical slip surface for the ESSA case at Phase 6, with a computed factor of safety of 1.36. As stated previously, this factor of safety remains relatively constant throughout dam construction, likely due to the shallow potential failure surface at the toe that remains constant throughout construction, in addition to low pore pressures in this area as a result of the drainage blanket. Also, the drained shear strength of the embankment and foundation materials, in the form of the friction angle, remains relatively constant throughout construction in contrast to the USSR, which decreases as a function of increasing load according to the SHANSEP formulation.

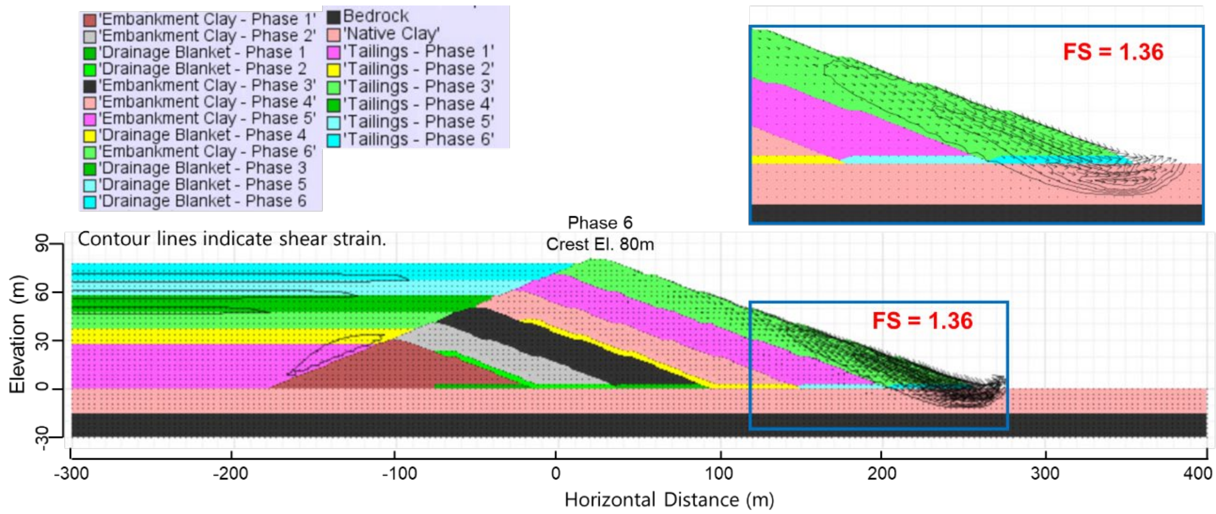

Figure 10: General critical slip surface for ESSA conditions (Phase 6 shown)

Conclusions

The discussion in this paper has led to the following conclusions:

- Overconsolidated clays of stiff to very stiff consistency can provide, in general, a relatively suitable foundation for tailings storage facilities when properly designed considering the impact of stress history on strength.
- It is found that in practice these foundations are often mischaracterized during design, and this can lead to and has resulted in catastrophic failures.
- In some cases, the stiff clay foundations are solely characterized using the drained shear strength because they are considered "stiff clays" and thus they will mobilize the drained shear strength. While this is initially true, the stress history may change as the effective stress increases and the OCR decreases, resulting in slightly overconsolidated or normally consolidated clays, which tend to mobilize the undrained shear strength.
- In other cases, the possibility of mobilizing the undrained shear strength is acknowledged but it is assumed that the undrained shear strength ratio initially determined on overconsolidated specimens remains constant over time and does not change with increasing effective stresses as the dams are raised. In reality, the initially high undrained shear strength ratio will tend to decrease with increasing effective stress.
- The SHANSEP procedure represents a practical technique that readily captures the change in undrained shear strength ratio and shear strength based on stress history. An example simulating construction of a downstream tailings dam founded on an overconsolidated clay using SHANSEP has been presented to illustrate its application.

- Figure 5 clearly shows how the OCR at the monitoring points within the foundation clay decreases as the dam crest increases with different construction phases. At Phase 5, the foundation clay becomes normally consolidated with an OCR=1.0.
- Similarly, Figure 6 clearly illustrates how the USSR at the monitoring points within the foundation clay decreases as the dam crest increases with different construction phases. At Phase 5, the foundation clay exhibits an undrained shear strength ratio of 0.22, which is associated with a normally consolidated undrained shear strength.
- In its native, pre-construction state, the bottom of the foundation clay was classified as "very stiff" and exhibited an undrained shear strength of 137 kPa with an OCR of about 8.5. With additional embankment loading, the undrained shear strength increased from 137 to 325 kPa while the OCR decreased from about 8.5 to 1, resulting in the USSR falling from about 1.2 to 0.22.
- Even though the undrained shear strength increased substantially due to embankment filling, the behavior would likely change from dilative in which the drained shear strength would govern to contractive in which the undrained shear strength would govern. This change in behavior is significant in that undrained failures of normally consolidated clay foundations can be brittle and sudden.
- The impact of these stress history effect on slope stability is indicated in Table 2, which summarizes the computed factor of safety. Table 2 shows that at Phase 6 the computed factors of safety are lower than the minimum recommended values.

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