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# Rapid Drawdown Stability Analysis of San Luis Dam

Stark, T.D., tstark@illinois.edu
Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, Urbana, IL 61801

Jafari, N.H., njafari@lsu.edu
Department of Civil and Environmental Engineering, Louisiana State University, Baton Rouge, LA 70803

**ABSTRACT: This paper uses the 1981 San Luis Dam upstream slope failure to evaluate the progression of the phreatic surface through the fine-grained core and pore-water pressures at failure for drawdown stability analyses. The hydraulic conductivity and compressibility parameters of saturated and unsaturated soils are calibrated using the reservoir hydrograph and thirteen piezometers to evaluate the pore-water pressures at failure. The analyses show unsaturated and transient seepage analyses can be used to evaluate the progression of the phreatic surface through the fine-grained core and estimate the seepage induced, not shear induced, pore-water pressures during drawdown for various stability analyses.**

## INTRODUCTION

A rapid drawdown is an important condition controlling the design of the upstream slope in embankment dams (Bishop and Bjerrum 1960; Morgenstern 1963; Sherard 1953). In particular, slides due to rapid drawdown can lead to reduced reservoir capacity and dam failure. The current state of practice for rapid drawdown analyses involves two approaches: (1) undrained shear stability analyses (USSA) and (2) effective stress stability analyses (ESSA). The USSA method uses anisotropic undrained shear tests at consolidation pressures prior to drawdown to evaluate shearing resistance (USACE 1970; Lowe and Karafiath 1959; Duncan et al. 1990; Duncan and Wright 2005). ESSA expresses drained shear strength in terms of effective stress parameters and estimates seepage and shear-induced pore-water pressures at drawdown. One advantage of the ESSA method is that drained shear strengths can be determined reliably for use in this method but estimating the pore-water pressures is challenging. In particular, the location of boundaries between materials, soil hydraulic conductivity and compressibility properties, and maximum rate of drawdown are necessary to estimate the pore-water pressures during drawdown (Terzaghi et al. 1996). Because the San Luis Dam material boundaries and rate of drawdown are well-documented (VonThun 1985; Stark 1987; Stark and Duncan 1991) and 13 piezometers were installed after the 1981 upstream slide, the hydraulic conductivity and compressibility properties of saturated and unsaturated soils could be calibrated using a transient seepage analysis. As a result, this study is focused on transient seepage (drawdown and flood loading conditions) through unsaturated embankment soil, e.g., levees and dams, for input in slope stability analyses.

## SAN LUIS DAM

This 1981 failure event case history involves the upstream slide in San Luis Dam (now known as B.F. Sisk Dam) in California, which is described in VonThun (1985) and Stark and Duncan (1991). In September 1981, after the reservoir was drawn down 55 m (180 ft) in 120 days, a major slide occurred in the upstream slope (VonThun 1985; Stark and Duncan 1991). Prior to the 1981 slide, San Luis Dam experienced several drawdown cycles, but the 1981 drawdown was the longest and fastest in San Luis Dam history. The slide was about 550 m (1,800 ft) long along the centerline of the dam crest. Failure causation analyses by VonThun (1985) and Stark (1987) found the slide was deep-seated with the majority of the failure surface located in the slopewash left in the foundation during construction. The construction specifications required the existing hill in the slide area to be stripped to a horizon that exceeded the strength of the overlying embankment material. Because the slopewash was highly desiccated at the time of construction, it was not removed (Stark and Duncan 1991). Upon reservoir filling and wetting of the desiccated slopewash, the shear strength reduced to fully softened strength. Then, the possible colluviual nature of the slopewash and cyclic loading from the reservoir water level resulted in shear deformations sufficient to mobilize a shear strength near the residual value. As a result, the significant reduction in slopewash strength resulted in the slope failure (Stark 1987; Stark and Duncan 1991).

### *Embankment Zones*

An upstream to downstream geologic and embankment cross-section of San Luis Dam at Station 135+00 is shown in Figure 1. The slopewash shown in Figure 1 blankets the bedrock in the lower portion of the upstream slope, covering an area that extends from the toe of the dam to a horizontal distance of -60 m in Figure 1. The slopewash liquid limit (*LL*), plasticity index (*PI*), and natural water content ($w_o$) are about 38-45%, 19-21%, and 7-8%, respectively. The impervious fine-grained core (Zone 1) in Figure 1 is a high plasticity clay compacted to +2% wet of optimum and a dry unit weight of 14.5 kN/m$^3$. The miscellaneous clayey gravel fill (Zone 3) overlying the slopewash is borrow material that originates from the channel excavation and is predominantly clay with *LL*, *PI*, and $w_o$ of 28-35%, 14-21%, and 14%, respectively. Zones 4 and 5 are rockfill material buttressing the fine-grained core. Zone 4 consists of minus 20 cm rockfill while Zone 5 rockfill is plus 20 cm. After the 1981 slide, a stabilizing rockfill berm was constructed at the toe of the slope (see Zone 7 in Figure 1).

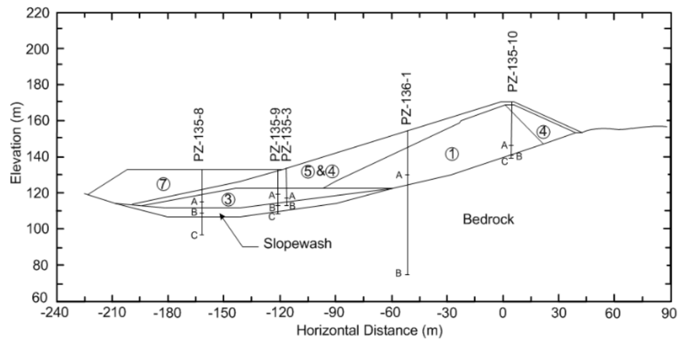


Figure 1. Geologic cross-section of San Luis Dam and location of piezometers installed after the 1981 slide (after Stark 1987).

### *Reservoir Hydrograph and Piezometer Locations*

Figure 2 shows the San Luis Reservoir hydrograph from 1968 to 1987. The first filling occurred in 1968 at an approximate rate of 0.11 m/day until the reservoir reached a capacity of Elev. +165 m. The reservoir was maintained at or near Elev. +165 m until 1974 (~6 years), which allowed Zone 3 and slopewash to saturate and approach steady-state conditions in parts of Zone 1 (Stark 1987). After 1974, the reservoir level cycled each year with the lowest level of Elev. +105 m occurring during the 1977 drought (Stark 1987). The drawdown rate of 0.45 m/day that preceded the 1981 slide was the largest and fastest the reservoir had experienced. After the 1981 slide, the reservoir was raised to Elev. +140 m at a rate of 0.10 m/day but was lowered at a rate of 0.42 m/day to Elev. +90 m in late 1982 for repair. In 1983, the reservoir level was raised to Elev. +165 m at a rate of 0.27 m/day. From 1984 until 1987, the reservoir remained at capacity (lowest level at Elev. +122 m) with two drawdown cycles. Figure 2 also identifies when the toe berm was completed and piezometers installed, which are used to calibrate the soil seepage properties. After the stabilizing toe berm was completed in 1983, a total of thirteen (13) vibrating wire piezometers were installed in five (5) borings (see Figure 1) to monitor pore-water pressures within different zones and depths. Three of the piezometers are located at mid-depth of the slopewash, three in Zone 3, and two in the Zone 1 fine-grained core. The pore-water pressures measured using the piezometers are used to establish the initial seepage boundary conditions for analysis and adjust the seepage parameters to achieve the best agreement between the measured and calculated pressure heads.

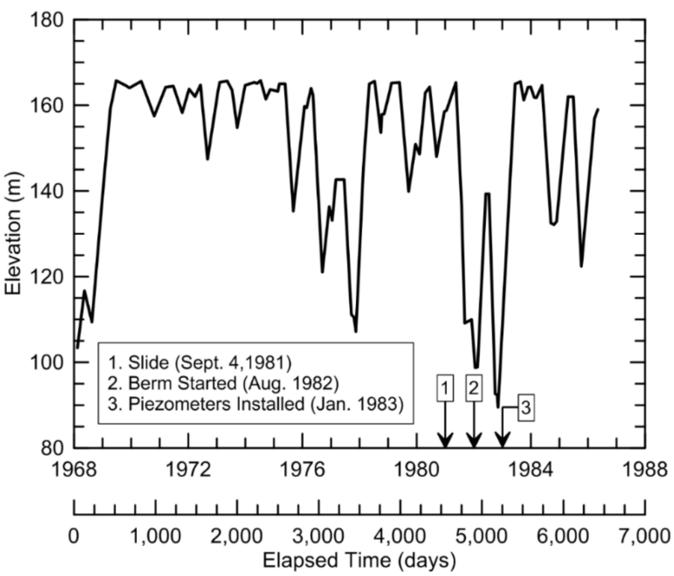


Figure 2. San Luis Reservoir hydrograph with: (1) 1981 upstream slide, (2) berm construction started and was completed in 1983, and (3) piezometers installed.

## SEEPAGE MODEL

Stark (1987) performed an unsaturated and transient seepage analysis to estimate the seepage induced pore-water pressures within the slopewash and Zone 1 materials for various stability analyses. In this study, SLIDE 6.0 (Rocscience 2010) finite element seepage software is used to calibrate soil properties and evaluate pore-water pressures in the slopewash and Zone 1 at the time of failure. SLIDE is a slope stability software package with built-in finite element groundwater seepage analysis for steady-state or transient flow conditions.

*Parent Analysis*

The initial groundwater conditions are used as the origin for the transient seepage analysis. Stark et al. (2014) use a steady-state seepage analysis to develop pore-water conditions for a floodwall case study involving foundation underseepage. In the present study, a steady-state analysis is also used to predict initial suction values (prior to reservoir filling) for the San Luis Dam cross-section in Figure 1. For the steady-state analysis, the left-hand side (LHS) and upstream slope boundary conditions are assigned a total head of Elev. +90.6 m, which reflects no reservoir. The right-hand side (RHS) total head boundary is set to a total head of 132 m to ensure the slopewash remains unsaturated before reservoir operation begins. The bottom boundary is set to a no flow condition because competent bedrock underlies the dam and slopewash.

*Transient Boundary Conditions*

For a transient seepage analysis, the initial groundwater and boundary conditions must be defined. The measured pore-water pressures from January 1983 to March 1986 provide a basis for establishing the seepage boundary conditions and refining the material properties for the transient seepage analyses. The boundary conditions applied in SLIDE are shown in Figure 3. The foundation piezometers (135-9C, 135-8C, and 136-1B) in Figure 1 show immediate response to reservoir changes, which indicates a hydraulic connection between the foundation bedrock and reservoir. The bottom model boundary is modeled as a no-flow boundary via the zero normal infiltration rate in SLIDE to reflect competent bedrock. The reservoir hydrograph in Figure 2 is applied to the upstream slope and is modeled as a total head boundary in SLIDE. The transient analysis is divided into two stages because a toe berm constructed as a remedial measure changes the model geometry. The first stage extends from 1967 to 1983 (0 to 5,665 days) and the second stage from 1983 to 1987 (5,665 to 6,615 days). Readings at piezometer 135-10A in Zone 1 (see Figure 1) remained zero indicating the soil remained unsaturated from 1983 to 1987. In addition, the landside slope of the existing hill is desiccated so a constant negative pressure head of -15 m is applied to signify the presence of a groundwater surface at a shallow depth. Stark (1987) also indicates the slopewash was desiccated and the downstream slope remained unsaturated, so the RHS boundary is modeled with a constant head of 132 m.

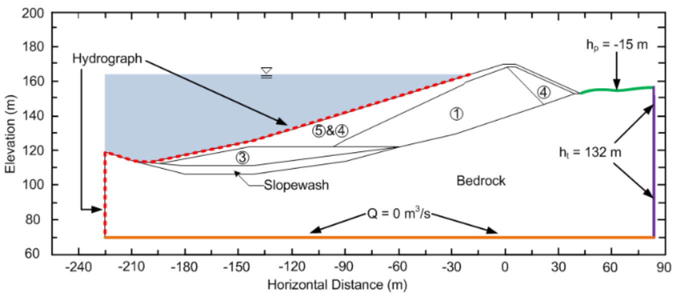


Figure 3. Boundary conditions applied for transient seepage model.

*Calibrated Soil Properties*

In unsaturated and transient seepage analyses, four soil properties are required: (1) initial matric suction pressures, (2) unsaturated hydraulic conductivity function (HCF), (3) saturated $k_h/k_v$ ratio, and (4) soil compressibility ($m_v$). The soil properties used in this analysis are based on laboratory and site data from Stark (1987), which were used as a starting point for model calibration. Unsaturated soil properties are important for Zone 1 because the fine-grained core is a compacted embankment and does not become fully saturated until the reservoir develops a phreatic surface. The van Genuchten (1980) model SWCC and HCF are applied to the slopewash, Zone 1, and Zone 3 materials. The model calibration of Zone 1 involves varying the van Genuchten (1980) curve fitting parameters "α" and "n" while the saturated $k_h$, $k_h/k_v$, and $m_v$ are varied for the saturated slopewash and Zone 3. The final soil properties are calibrated using 13 piezometer readings from 1983-1986 (elapsed time of 5,665 to 6,665 days). The SLIDE model incorporate the steady-state analysis as the initial groundwater conditions (see "parent analysis"). In addition, the transient analysis is divided into two stages to accommodate the toe berm construction and change in upstream geometry. The model time step is seven (7) days for the 1981 slide (4,850 to 5,850 days) and the period of piezometer data (5,665 to 6,615 days) to capture increasing and decreasing reservoir levels. All other periods, e.g., constant reservoir capacity, used an increased time stepping of 90 days to reduce computational time while maintaining model accuracy. The meshing utilized a four-node element with over 5,000 elements to provide adequate accuracy.

The transient model is calibrated using piezometers located in the slopewash, Zone 3, and Zone 1. Because Zone 3 and slopewash materials saturate rapidly during the first filling of San Luis Reservoir, the saturated $k_h$, $k_h/k_v$, and $m_v$ define the total head response of these materials (Stark et al. 2014) and are adjusted to reach agreement between the model and field piezometers measurements. Varying $m_v$ values produces a time lag effect, i.e., pore-water pressure response is accelerated or delayed. The calibration process is not solely focused on the individual response of the slopewash material because interaction of soils layers is present, specifically the foundation bedrock. By lowering the foundation $k_h$ to an impervious material, drainage into the foundation is limited, causing pore-water pressures to build up in the slopewash. In contrast, modeling the foundation as a pervious material allows drainage and therefore decreases the pore-water pressure response in the slopewash. The calibrated slopewash parameters produced good agreement with field measurements during drawdown, which is key for analysis of the 1981 slide and refilling (Stark et al. 2016). Because the Zone 1 fine-grained core remains unsaturated during the piezometer monitoring period, the unsaturated properties influence calibration of the Zone 1 material. By adjusting α and n parameters and saturated $k_h$ in the van Genuchten (1980) model, agreement was obtained between the calculated and measured total heads. Variations in saturated $k_h$ affect the total head magnitude, similar to the slopewash and Zone 3 calibration. Zone 3 operates under the same saturated seepage mechanism as the slopewash, i.e., saturated $k_h$, $k_h/k_v$, and $m_v$, so adjusting these parameters yields similar results. The calibration process of replicating piezometer readings and the interaction of soil layers is discussed in Stark et al. (2016). This process resulted in the engineering properties shown in Table 1.

Table 1. Summary of calibrated seepage properties.

| Material | $k_h$ (cm/s) | $k_h/k_v$ | $m_v$ (kPa$^{-1}$) | $\theta_S$ | $\theta_R$ | 1/α (m) | n |
|---|---|---|---|---|---|---|---|
| Zone 1 | $1x10^{-5}$ | 2 | $8.35x10^{-6}$ | 0.29 | 0.05 | 4.2 | 4.5 |
| Zone 3 | $1.5x10^{-6}$ | 2 | $1.0x10^{-6}$ | 0.27 | 0.02 | 28.6 | 3.2 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Slopewash | $1.0x10^{-8}$ | 2 | $3.5x10^{-6}$ | 0.24 | 0.02 | 50 | 2.6 |
| Foundation | $1.4x10^{-4}$ | 1 | $1.67x10^{-5}$ | -- | -- | -- | -- |

## SLOPEWASH PORE-WATER PRESSURES

The 1981 upstream failure surface passed through the slopewash at a depth of 3 to 15 m (Stark 1987), so evaluating the pore-water pressures at failure in the slopewash is necessary to perform effective stress inverse stability analyses. The main objective is to compare the slopewash pore-water pressures from SLIDE with Stark (1987). Pore-water pressures are reported herein at the middle and top of slopewash to provide an upper and lower bound, respectively, for the stability analyses. The pressure-heads at the top of slopewash in Figure 5a are depicted along the horizontal length of slopewash from the slope toe to Zone 1. Maximum pore-water pressures occur near the middle of the slopewash (horizontal distance= -145 m) and decrease in both directions for Stark (1987) and SLIDE. Stark (1987) and SLIDE pressure heads are about 18 m and 14 m at this location, respectively, and are in good agreement given the different models. Variations between Stark (1987) and this study are attributed to the value of foundation saturated $k_h$. Stark (1987) models the foundation as an impermeable material, whereas this study uses a saturated $k_h$ of $1.3x10^{-4}$ cm/s. As a result, pore-water pressures in Stark (1987) are higher, especially in the first half of the slopewash (horizontal distance from -210 m to -145 m).

Figure 5b shows a comparison of pressure head for the middle of slopewash for Stark (1987) and this study. The present analyses do not capture the peak pressure head of 42 m exhibited at horizontal distance of -83 m from Stark (1987). The SLIDE results indicate a maximum pressure head of ~21 m at a horizontal distance of -150 m. The peak pressure head of 42 m in Stark (1987) occurs at a horizontal distance of -80 m, where the Zone 1 material overlies the slopewash. These soil layers were modeled with much lower $k_h$ values (about 2 orders of magnitude) than this study, which is important because this portion of the slopewash lies between Zone 1 and bedrock foundation (see Figure 1). Because the pore-water pressures could not drain in the vertical direction into Zone 1 or the bedrock, the high pore-water pressures in Stark (1987) are attributed to the long drainage path in the horizontal direction along the slopewash.

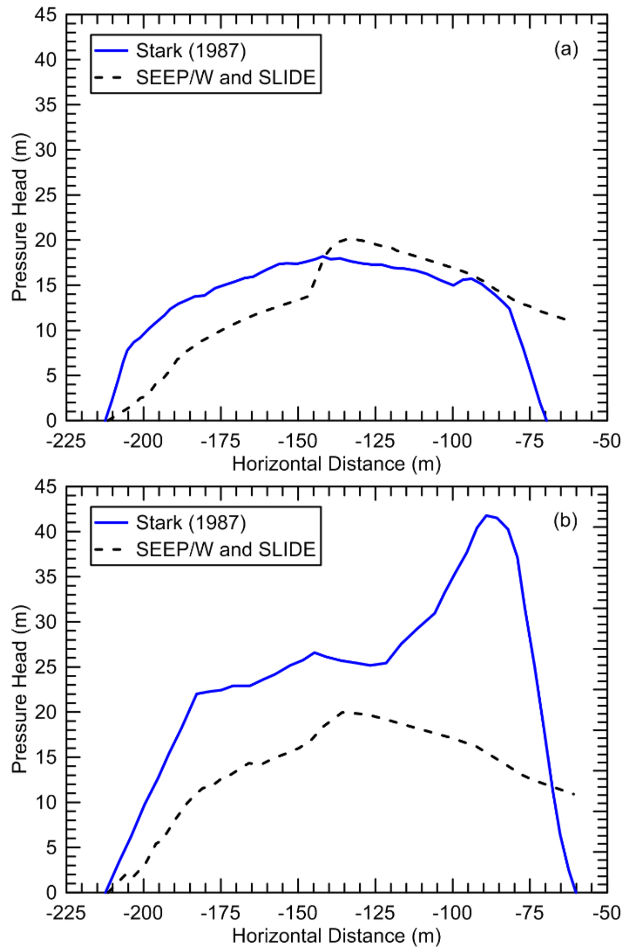


Figure 5. Calculated seepage pressure head at the time of 1981 slide along (a) top and (b) middle of slopewash.

## STABILITY ANALYSES

A major impetus for using the San Luis Dam case history is to investigate the decrease in factor of safety (FS) from transient seepage and shear-induced pore-water pressures because the FS should be about unity in 1981. SLIDE was used to compute the upstream slope FS during drawdown conditions and also analyze the prior stability analysis by Stark and Duncan (1991). The slopewash shear strength parameters obtained from direct shear and triaxial compression tests (Stark and Duncan 1991) and the resulting FS are shown in Tables 2 and 3. The peak shear strengths for Zones 1 and 3 are c'=5.3 kPa, ϕ'=25° and c'=4.8 kPa, ϕ'=25°, respectively (Stark and Duncan 1991).

To perform a stability analysis in SLIDE, the pore-water pressures are determined at the time of failure (September 4, 1981), which are provided in Figure 5. Stark and Duncan (1991) and SLIDE analyses were performed using the Spencer (1967) stability method, which satisfies all conditions of equilibrium. The Failure Surface 1 in Figure 6 corresponds to the surface used by Stark (1987) and passes through Zone 3 based on slope inclinometer measurements projected from another station to Station 135+00. The hill incorporated in the dam is undulating, thus the failure surface likely extends only through the weaker slopewash at Station 135+00 (see Figure 6 and Failure Surface 2). Because of this uncertainty, both failure surfaces are investigated herein.

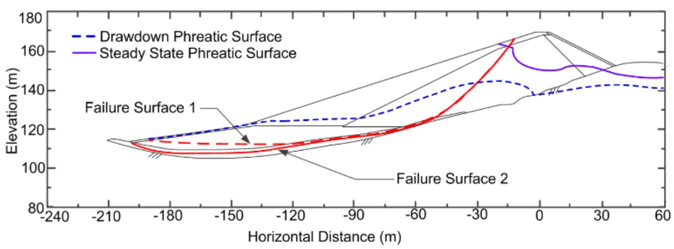


Figure 6. Estimated upstream slide planes at Station 135+00.

Table 2. Summary of shear strengths at various conditions and factor of safety computed for Failure Surface 1 by Stark and Duncan (1991).

| | Condition | Slopewash | F of S: Stark and Duncan (1991) |
|---|---|---|---|
| 1 | End of Construction, Desiccated Slopewash | c'= 263 kPa, ϕ'= 39° | 4.0 |
| 2 | Reservoir Full, Fully Softened Slopewash | c'= 0 kPa, ϕ'= 25° | 2.0 |
| 3 | Reservoir Drawdown, Fully Softened Slopewash | c'= 0 kPa, ϕ'= 25° | 1.3 |
| 4 | Reservoir Drawdown, Reduced Fully Softened Slopewash | c'= 0 kPa, ϕ'= 20° | N/A |
| 5 | Reservoir Drawdown, Residual Slopewash | c'= 0 kPa, ϕ'= 15° | 1.0 |

Table 3 summarizes the changes in FS for varying reservoir levels and slopewash shear strength parameters and provides a comparison of FS between this study and Stark and Duncan (1991). In particular, the column "Stark and Duncan (1991)" in Table 2 lists the FS for Failure Surface 1 as reported in Stark and Duncan (1991). The purpose of the Table 3 column "Stark and Duncan (1991) using SLIDE" is to reproduce the FS in Table 2 using the pore-water pressures from Stark and Duncan (1991) and Failure Surface 1. The FS values under Failure Surface 2 incorporates the Stark and Duncan (1991) pore-water pressures into SLIDE. In Table 3, the column "Present Study" presents the FS values for both failure surfaces with updated pore-water pressures developed using SLIDE in this study. At the end of construction with the slopewash still highly desiccated, both analyses report FS of approximately four (4). During reservoir full conditions, all analyses report FS of approximately two (2) with the slopewash shear strength reduced to the fully softened value (Gamez and Stark 2014). When the reservoir level is lowered and the slopewash is still assigned a fully softened shear strength, Stark and Duncan (1991) report the FS decreases to 1.3 (Table 2), while the present study computes a FS of 1.5 for Failure Surface 2 (Table 3). The higher FS in this study is attributed to a lower phreatic surface in Zone 1 and lower pore-water pressures in the slopewash due to the bedrock being impermeable in Stark and Duncan (1991) (see also Figure 5). The FS for Failure Surface 2 approaches unity (1.0) for both analyses once the slopewash is reduced to residual strength. Using the pore-water pressures reported in Stark (1987) and Failure Surface 2, SLIDE produced a FS of 0.9. A FS at or below unity is reasonable because the seepage pore-water pressures in the slopewash (see Figure 6) are greater than the shear-induced pore-water pressures along most of the failure surface. The values of shear-induced pore-water pressures are calculated below, but on average the seepage and shear-induced pore-water pressures contribute about 65% and 35%, respectively, between the toe and downstream extent of the slopewash.

Because the slopewash is a colluvial material and subjected to cyclic shear stresses imposed by reservoir drawdowns, the shear strength can range from fully softened to residual (reduced fully softened strength in Table 2) as suggested by Stark and Eid (1997) and Stark and Duncan (1991), respectively. Therefore, an additional reservoir drawdown analysis was performed with the slopewash strength at $\phi$'= 20°, which is between the fully softened and residual friction angles. The FS is 1.45 and 1.25 for Failure Surfaces 1 and 2, respectively.

Table 3. Summary of factor of safety for upstream slope of San Luis Dam.

| | Stark and Duncan (1991) using SLIDE | | Present Study | |
|---|---|---|---|---|
| Condition | Failure Surface 1 | Failure Surface 2 | Failure Surface 1 | Failure Surface 2 |
| 1 | 3.9 | 4.8 | 3.7 | 4.7 |
| 2 | 2.1 | 2.0 | 2.1 | 1.9 |
| 3 | 1.3 | 1.27 | 1.6 | 1.5 |
| 4 | 1.2 | 1.08 | 1.45 | 1.25 |
| 5 | 1.08 | 0.9 | 1.3 | 1.05 |

The FS values reported in Table 2 for Stark and Duncan (1991) and Failure Surface 1 are slightly greater than unity for the residual strength condition (slopewash $\phi$'= 15°), which is acceptable because shear-induced pore-water pressures are not calculated in the transient seepage analysis and not included in the stability analysis. The effects of seepage and shear-induced pore-water pressures was inferred from the FS values at the reservoir full and drawdown conditions using only Failure Surface 2. When the stability was evaluated for only total head seepage pressures and a slopewash strength of $\phi$'= 20º, the FS value decreased from 1.45 to 1.25 after reservoir drawdown. Therefore, the FS should approach unity when shear-induced pore-water pressures are accounted for in the stability analysis. In other words, the FS should decrease by about 0.25 if shear-induced pore-water pressures are included. To confirm this decrease in FS of about 0.25, the shear-induced pore-water pressures in the slopewash were estimated using Skempton's (1954) $A$ coefficient and the change in normal total stresses along the observed failure surface due to reservoir drawdown. The $A$ coefficient is used and not $\overline{B}$ (Bishop 1954) because only changes due to shear or deviator stresses are being considered. The value of $A$ was calculated for the failure condition in triaxial compression tests, resulting in values of $A$ at failure ($A_f$). An average $A_f$ value of 0.42 was estimated for the slopewash using results from isotropically consolidated-undrained triaxial compression tests, with pore-water pressure measurements conducted by Stark (1987) on an upstream slopewash sample wetted by the reservoir. These triaxial compression tests show $A_f$ ranges from 0.4 to 0.43 for effective confining pressures of 89.5 to 275.8 kPa. A value of $A_f$ of about 0.42 is also in agreement with lightly overconsolidated clays ($A_f$ = 0 to 0.5) and normally consolidated clays ($A_f$ = 0.5 to 1.0) according to Skempton (1954). Because the triaxial compression tests performed by Stark (1987) at low normal stresses show the upstream slopewash is slightly overconsolidated even after reservoir filling, a value of $A_f$ close to 0.5 is reasonable.

The shear-induced pore-water pressures along the slopewash failure surface were estimated using $A_f$ of 0.42 and the change in normal total stress caused by lowering of the reservoir. The transient seepage pore-water pressures from SLIDE and shear-induced pore-water pressures estimated from the $A_f$ value were combined together to estimate the total pore-water pressure acting on each slice along the failure surface. The normal effective stresses on each slice were estimated from the normal total stress obtained from SLIDE and estimated total pore-water pressure, which were then used to determine the mobilized shear strength along the failure surface. The resulting FS is 1.02, which suggests the transient seepage and shear-induced pore-water pressures can be separately evaluated and combined to provide a reasonable value of FS for this case history. Although coupled hydro-mechanical analyses are more complex than transient seepage analyses, they are recommended to directly account for the interactions of transient flow, shear-induced volume change, and consolidation due to changes in reservoir levels as suggested by Alonso and Pinyol (2011).

## SUMMARY

This paper uses a field calibrated seepage model of San Luis Dam to illustrate the influence and effect of rapid drawdown on the upstream slope. The unsaturated and transient seepage analysis utilized SLIDE software package to predict the migration of phreatic surface during various reservoir levels and evaluate the influence of unsaturated properties on pore-water pressure dissipation during drawdown. Initial suction conditions for an unsaturated and transient seepage analysis can be estimated using a steady-state analysis. The steady-state results serve as the start or origin of the transient seepage analysis. In practice, in situ measurement of volumetric moisture content and suction should be utilized to validate the initial suction profile. Unsaturated and transient seepage analyses can be used to estimate the pore-water pressures caused by changes in hydraulic conditions for input in an ESSA with shear-induced pore-water pressures that are estimated using the pore pressure coefficient $A_f$. As a result, this case history suggests slope stability analyses can be performed for a range of drawdown rates without requiring undrained shear strengths at multiple consolidation pressures and stress conditions.

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