San Luis Dam Case History: Seepage and Slope Stability Analyses and Lessons Learned

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ABSTRACT

Rapid drawdown is an important condition controlling the design of the upstream slope in embankment dams and levees. This paper presents the upstream slope failure at San Luis Dam as a case study for performing effective stress drawdown stability analyses. In effective stress analyses, stress-dependent drained shear strengths were used for the fine-grained core and slopewash materials. The seepage-induced pore-water pressures were estimated in the slopewash and fine-grained core by first calibrating the saturated and unsaturated hydraulic parameters with piezometers installed after the slide. The shear-induced pore-water pressures due to the changes in total stress were estimated from Skempton's A coefficient. The stability analysis methodology incorporates stress-dependent drained shear strength envelopes and shear-induced pore-water pressures to determine the factor of safety (FS). The FS determined at various reservoir levels and slopewash shear strengths are used to provide recommendations for calibrating unsaturated soil properties, integrating seepage-induced and shear-induced pore-water pressures in stability analyses, and use of commercial software for drawdown stability analyses.

INTRODUCTION

Sudden or rapid drawdown is an important condition controlling the design of the upstream slope in embankment dams and levees (Bishop 1952; Lowe and Karafiath 1959; Bishop and Bjerrum 1960; Morgenstern 1963; Sherard 1953; Sherard et al. 1963; Duncan et al. 1990; Terzaghi et al. 1996). 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 undrained shear tests at consolidation pressures prior to drawdown to evaluate shearing resistance (USACE 1970; Lowe and Karafiath 1959; Duncan et al. 1990). In contrast, ESSA expresses drained shear strength in terms of effective stress parameters and estimates seepage and shear-induced pore-water pressures at drawdown. Existing procedures to estimate pore-water pressures after drawdown include (1) assuming $\bar{B}$, i.e., change in pore-water pressure (Δu) divided by change in the major principal stress ($\Delta\sigma_1$) or ($\Delta u/\Delta\sigma_1$) based on Skempton (1954), is unity for saturated soils (Bishop 1952, 1954; Skempton 1954); (2) finite-element unsaturated and transient seepage analyses, which provide pore-water pressure from boundary hydraulic conditions (Stark et al. 2014); and (3) coupled hydro-mechanical finite element analyses (Alonso and Pinyol 2011). The first procedure ($\bar{B}$) is the sum of transient flow induced pressure heads and shear induced pore-water pressure change caused by an instantaneous drawdown (Skempton 1954). The second procedure predicts transient pore-water pressures using finite-element seepage analyses with transient hydraulic boundaries. This procedure does not include shear-induced pore-water pressures because transient flow is solved using Laplace's equation, which assumes void ratio is constant. The third procedure is a hydro-mechanical analysis that models total head induced flow and volume change induced flow caused by swelling and compression.

Because the San Luis Dam material boundaries and rate of drawdown are well-documented (VonThun 1985; Stark 1987; Stark and Duncan 1991) and thirteen piezometers were installed after the 1981 upstream slide, the saturated and unsaturated soils were calibrated using a partially saturated transient seepage analysis. This study presents the transient seepage analyses to determine the pore-water pressures at failure for input in slope stability analyses. The factor of safety at San Luis Dam is evaluated for a range of shear strengths of the slopewash and the effect of including shear-induced pore-water pressures using Skempton's A coefficient. The results are summarized in recommendations for performing rapid drawdown stability analyses.

SAN LUIS DAM

The drawdown case history involves the 1981 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. The slide was about 550 m (1,800 ft) 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 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 (see location in Fig. 1), the shear strength reduced to fully softened strength. Then, the possible colluvium nature of the slopewash and cyclic loading from the reservoir water level resulted in shear deformations sufficient to mobilize shear strengths between fully softened and residual values. As a result, the significant reduction in slopewash strength was the driving force behind the slope failure (Stark 1987; Stark and Duncan 1991).

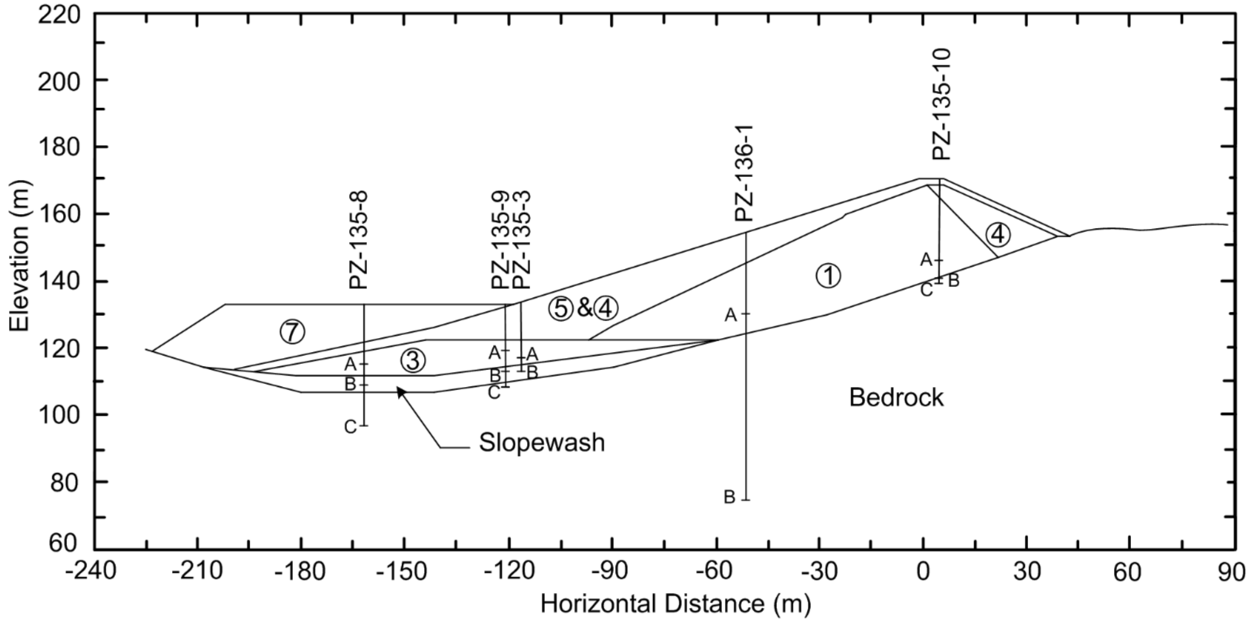


Figure 1. Geological cross-section and location of piezometers used calibration saturate and unsaturated flow properties

A geologic cross-section of San Luis Dam at Station 135+00 is shown in Fig. 1. The slopewash 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 Fig. 1. The slopewash liquid limit (LL), plasticity index (PI), and natural water content (w_o) are on average 43%, 20%, and 7%, respectively. The impervious fine-grained core (Zone 1) is a high plasticity clay compacted to +2% wet of optimum and a dry unit weight of 14.5 kN/m^3. Table 1 summarizes the soil index properties for the fine-grained soils in Fig. 1. The initial degree of saturation and volumetric water content reported in Table 1 were used in developing the unsaturated soil properties.

Table 1. Summary of index and hydraulic engineering properties from Stark (1987)

Soil Property	Zone 1	Slopewash
Liquid Limit, LL (%)	40-47	38-45
Plasticity Index, PI (%)	23-30	19-21
Natural Water Content, w_o (%)	15	7-8
Initial Degree of Saturation, S_r (%)	90	55-60
Porosity, n	0.29	0.29
Volumetric Water Content, θ	0.26	0.16-0.17

Fig. 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 again 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. The timeline in Fig. 2 indicates the toe berm was started in August of 1982. The piezometers were installed in January 2013, which are used to calibrate the soil seepage properties.

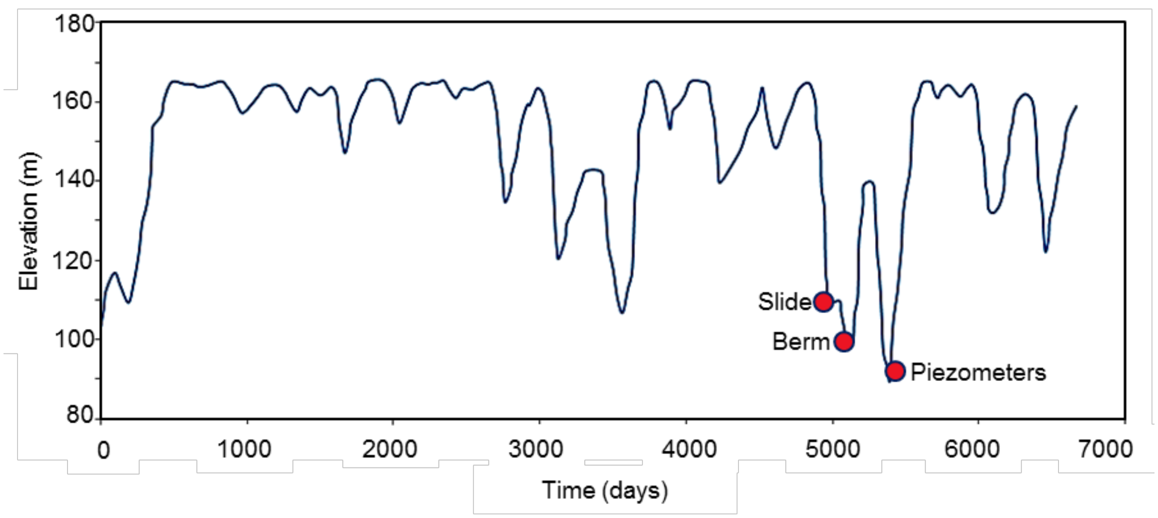


Figure 2. San Luis Dam hydrograph depicting times of 1981 upstream slide (September 4); berm construction started (August 1982); and piezometers installed (January 1983).

TRANSIENT SEEPAGE MODEL

In this study, 2-D SEEP/W (Geo-Slope 2007) finite element software was used to calibrate soil properties and evaluate pore-water pressures in the slopewash and Zone 1 at the time of failure. SEEP/W is a general seepage analysis program formulated to model saturated and unsaturated soils for transient flow and excess pore-water pressure dissipation estimated from a stress-deformation analysis within porous materials.

For calibrating the unsaturated and saturated seepage properties, the initial groundwater and boundary conditions are necessary at the time when the piezometers were installed. The measured pore-water pressures from January 1983 to March 1986 provide a basis for establishing the seepage boundary conditions before refining the material properties for the transient seepage analyses. The boundary conditions applied in SEEP/W are shown in Fig. 3.

The foundation piezometers (135-9C, 135-8C, and 136-1B) in Fig. 1 show immediate response to reservoir changes, which indicates a hydraulic connection between the foundation and reservoir. The bottom boundary condition is modeled as a no-flow boundary via the no unit flux condition in SEEP/W to reflect competent bedrock. The reservoir hydrograph in Fig. 2 is applied to the upstream slope and is modeled as a total head boundary in SEEP/W. 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 Fig. 1) remained zero indicating the soil remained unsaturated from 1983 to 1987. In addition, the assumed water surface in the downstream side of the dam corresponds to a pressure head of -15 m, and the RHS boundary is modeled with a constant head of 132 m.

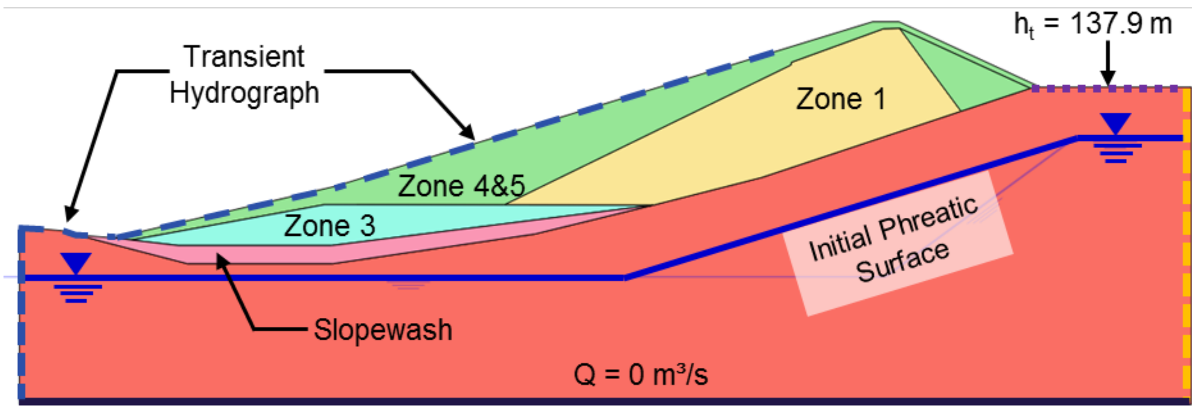
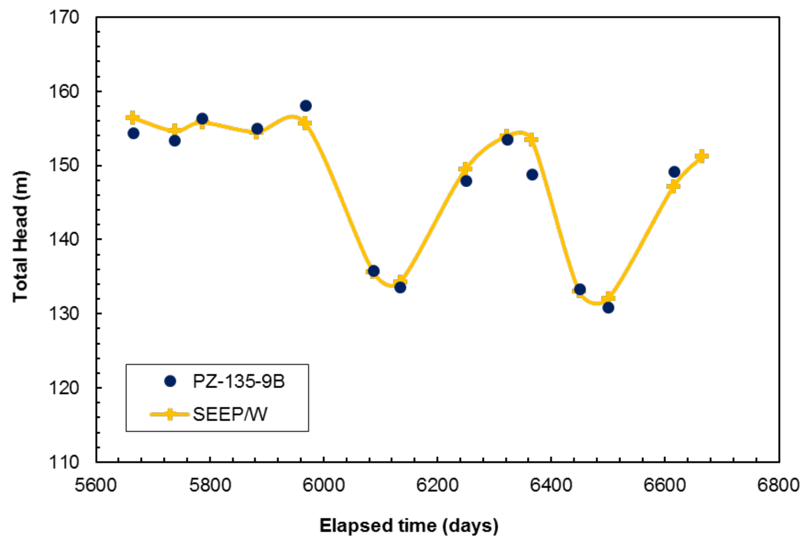


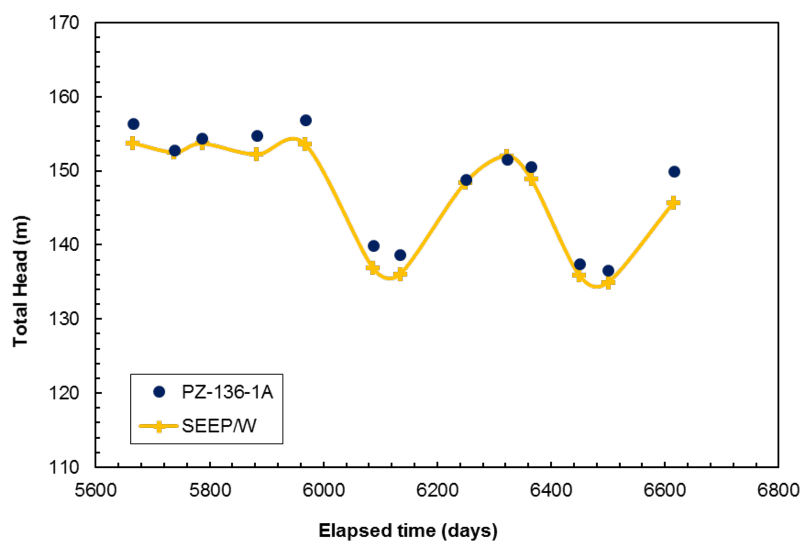
Figure 3. Boundary conditions applied for transient seepage model

In an unsaturated and transient seepage analysis, four soil properties are required: (1) initial matric suction profile, (2) unsaturated soil properties (soil water retention curve (SWRC) and hydraulic conductivity function (HCF)), (3) saturated k_h/k_v ratio, and (4) soil compressibility (m_v). The van Genuchten (1980) model SWRC 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 to 1986 (elapsed time of 5,665 to 6,665 days). The SEEP/W models incorporates the steady-state analysis as the initial groundwater conditions. This is followed by a transient analysis that is divided into two stages to accommodate the toe berm construction and change in upstream geometry. For SEEP/W, the model time step is seven (7) days during the 1981 drawdown (4,850 to 5,850 days) and the period of piezometer data (5,665 to 6,615 days) to ensure fluctuations in reservoir levels are accurately captured. All other reservoir operation periods, e.g., constant reservoir levels, used an increased time stepping of 90 days to reduce computation time while also maintaining model accuracy.

The transient model is calibrated using piezometers located in the slopewash (PZ-135-8B, PZ-135-9B, PZ-135-3B), Zone 3 (PZ-135-8A, PZ-135-9A, PZ-135-3A), and Zone 1 (PZ-136-1A and PZ-135-10A). Because the bedrock did not play a significant role in the 1981 slide and some uncertainty persists in the bedrock properties, less emphasis is placed on calibrating the response of the foundation piezometers. Therefore, the calibrated bedrock properties are selected so the response of the slopewash and Zone 1 match the piezometer measurements. The initial seepage analysis showed Zone 3 and slopewash materials saturate rapidly during the first filling of San Luis Reservoir. Therefore, 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 (see Fig. 4 for slopewash calibration).



(a)



(b)

Figure 4. Calibration of (a) Slopewash and (b) Zone 1 fine-grained core to piezometers

Preliminary seepage analyses demonstrated that the Zone 1 results were influenced by movement of the phreatic surface, i.e., cycles of wetting and drying. Because the Zone 1 fine-grained core remains unsaturated during the piezometer monitoring period, the unsaturated soil 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 is obtained between the calculated and measured total heads (see Fig. 4(b)). The process of replicating piezometer readings and the interaction of soil layers resulted in the engineering properties shown in Table 2. The calibrated parameters were used to determine the pore-water pressures at failure for the stability analyses.

Table 2. Summary of calibrated seepage properties

Material	k_h (cm/s)	k_h/k_v	m_V (kPa^{-1})	θ_S	θ_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	$1.67x10^{-5}$	--	--	--	--

STABILITY ANALYSES

A major impetus for using San Luis Dam 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 Table 4. The peak shear strengths for Zones 1 and 3 were c'=5.3 kPa, ϕ'=25° and c'=4.8 kPa, ϕ'=25°, respectively (Stark and Duncan 1991).
To perform a stability analysis in SLOPE/W, the pore-water pressures determined at the time of failure (September 4, 1981) are exported at nodes in the transient seepage model and then imported in the slope stability analysis as a grid (see Fig. 5 phreatic surface). In particular, the slopewash pore-water pressures applied in Stark and Duncan (1991) and this study are provided in Fig. 5. Stark and Duncan (1991) and SLOPE/W analyses were performed using the Spencer (1967) stability method, which satisfies all conditions of equilibrium. The Stark and Duncan (1991) failure surface (see Failure Surface 1 in Fig. 5) 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 Fig. 5 and Failure Surface 2). Because of this uncertainty, both failure surfaces are investigated herein.

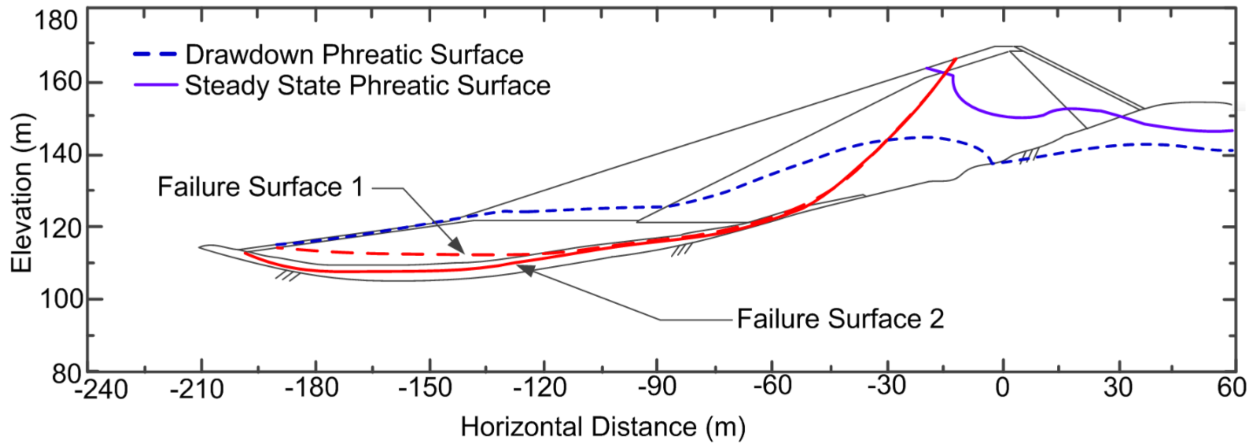


Figure 5. Estimated upstream slide planes at Station 135+00

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 first column “Stark and Duncan (1991)” in Table 3 lists the FS for Failure Surface 1 as reported in Stark and Duncan (1991). The purpose of the second column “Stark and Duncan (1991) using SLIDE” is to reproduce the FS in the first column in SLIDE by using the pore-water pressures from Stark and Duncan (1991) and Failure Surface 1. The FS values under Failure Surface 2 again incorporate Stark and Duncan (1991) pore-water pressures into SLIDE. The third column “Present Study” presents the FS values for both failure surfaces but with updated pore-water pressures developed from 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 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 while the present study computes a FS of 1.5 for Failure Surface 2. 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). 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, SLOPE/W produced a FS of 0.9. A FS at or below unity is reasonable because the seepage pore-water pressures in the slopewash 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 4) 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 ϕ'= 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. Calculated Factors of Safety for upstream slope of San Luis Dam

Condition	Stark and Duncan (1991)	Stark and Duncan (1991) using SLIDE		Present Study	
	Failure Surface 1	Failure Surface 1	Failure Surface 2	Failure Surface 1	Failure Surface 2
End of Construction Desiccated Slopewash: c'=263 kPa, ϕ'=39˚	4.0	3.9	4.8	3.7	4.7
Reservoir Full Fully Softened Slopewash: c'=0 kPa, ϕ'=25˚	2.0	2.1	2.0	2.1	1.9
Reservoir Drawdown Fully Softened Slopewash: c'=0 kPa, ϕ'=25˚	1.3	1.3	1.27	1.6	1.5
Reservoir Drawdown Reduced Fully Softened Slopewash: c'=0 kPa, ϕ'=20˚	N/A	1.2	1.08	1.45	1.25
Reservoir Drawdown Residual Slopewash: c'=0 kPa, ϕ'=15˚	1.0	1.08	0.9	1.3	1.05

The FS values reported in Table 3 for Stark and Duncan (1991) and Failure Surface 1 are slightly greater than unity for the residual strength condition (slopewash ϕ'= 15˚), which is acceptable because shear-induced pore-water pressures are not calculated in the transient seepage analysis and subsequently 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 ϕ'= 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, 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. In particular, $\overline{B}$ is the sum of transient flow induced pressure heads and shear induced pore-water pressure change caused by an instantaneous drawdown (Skempton 1954). In this method, Skempton (1954) defines $\overline{B}$ as:

$$\overline{B} = B\left[1-(1-A)\left(1-\frac{\Delta\sigma_3}{\Delta\sigma_1}\right)\right] \quad (1)$$

where the *B* coefficient represents pore-pressures developed from all-around pressures and is about unity (1.0) for saturated soils, *A* coefficient corresponds to shear-induced pore-pressure and is estimated during the application of a deviator stress, and $\Delta\sigma_1$ and $\Delta\sigma_3$ are changes in major and minor principal stresses, respectively. In a triaxial compression test subjected to an isotropic confining stress, increasing the deviator stress generates shear-induced pore-water pressures. Because only shear-induced pore-water pressures are generated in this triaxial compression test, the $\overline{B}$ procedure simplifies to Skempton's *A* coefficient (Skempton 1954), which can be used to estimate shear-induced pore-water pressures due to reservoir drawdown.

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 Af 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 was 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 SEEP/W 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 (Alonso and Pinyol 2011).

SUMMARY AND LESSONS LEARNED

This paper uses a piezometer 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 the software package SEEP/W 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. The following information and recommendations were derived from the analyses:

- The van Genuchten (1980) unsaturated soil model uses α and n parameters to model the SWRC and HCF. Increasing α shifts the SWRC and HCF to lower matric suction without changing the overall shape of the curve while the parameter n steepens the SWRC and HCF slope between θs and θr. Adjusting these parameters in the parametric study shows that α causes a greater impact on pore-water pressure response than n. In particular, increasing α lowers the HCF and inhibits pore-water pressure from draining at time of drawdown, which contributes to the lag time experienced in subsequent refilling. Therefore, practitioners should place significant emphasis on estimating α, which corresponds to the air entry value in the SWRC.
- Initial suction conditions for an unsaturated and transient seepage analysis can be estimated using a steady-state analysis. For seepage software to yield comparable steady-state results and initial suctions, the following input and model parameters must be similar: unsaturated properties and functions, dam geometry, boundary conditions, material properties, and meshing technique. 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 calculated 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 A_f. For example, SEEP/W predicted the seepage-induced pore-water pressures at the time of the 1981 slide. 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 conditions.

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