

Unsaturated and Transient Seepage Analysis of San Luis Dam

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ABSTRACT: This analysis uses the San Luis Dam upstream slide to evaluate pore-water pressures at failure and the progression of the phreatic surface through the fine-grained core for drawdown stability analyses. The soil hydraulic and compressibility parameters are calibrated using the reservoir hydrograph and the response of thirteen piezometers and then used to evaluate the pore-water pressures at failure. The analyses show unsaturated and transient seepage analyses can be used to estimate pore-water pressures during drawdown for various stability analyses and locate the progression of the phreatic through the fine-grained core. The transient results also indicate the van Genuchten parameter “ α ” significantly influences unsaturated soil response during drawdown. Different meshing techniques produce inconsistent moisture content profiles in seepage software packages, so in situ measurement of moisture content and suction pressure is recommended to develop an unsaturated and transient seepage model.

KEYWORDS: rapid drawdown, transient seepage analysis, hydraulic conductivity function, soil water characteristic curve

INTRODUCTION

Sudden or rapid drawdown is typically 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 undrained shear tests at consolidation pressures prior to drawdown to evaluate shearing resistance (USACE 1970; Lowe and Karafiath 1959; Duncan et al. 1990). 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).

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Lambe and Whitman (1969) define transient flow as the condition during water flow where pore-water pressure, and thus total head, change with time. Saturated soil seepage depends on the saturated horizontal hydraulic conductivity (k_h), hydraulic conductivity anisotropy ratio (ratio of horizontal k to vertical k or k_h/k_v), and coefficient of volume compressibility (herein referred to as soil compressibility or m_v) of embankment and foundation strata through which seepage will occur (Stark et al. 2014). Unsaturated soil delays seepage and propagation of pore-water pressures in embankments and is controlled by the soil-water characteristic curve (SWCC) and hydraulic conductivity function (HCF) (Fredlund and Rahardjo 1993). Stark et al. (2014) report the effects of saturated k_h and m_v on saturated foundation strata but they do not address partially saturated embankment materials.

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 (described below), the unsaturated and saturated soil hydraulic conductivity and compressibility properties could be calibrated using a transient seepage analysis in SLIDE. As a result, this case 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. In addition, the study investigates the progression of the phreatic surface through the embankment during reservoir operation and the usefulness of unsaturated soil models in SLIDE.

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). Sanctioned in 1960 under the Central Valley Project and the State of California Water Plan, San Luis Reservoir is the largest off stream man-made lake in the United States with a capacity of about 252 million m^3 (USBR 2013). San Luis Reservoir is located approximately 170 km southeast of San Francisco and began filling in 1968. 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 slopewash was highly desiccated at the time of construction and thus was not removed during construction because the existing hill only had to be stripped to a horizon that exceeded the strength of the overlying embankment material (Stark and Duncan 1991). The failure is attributed to the desiccated slopewash losing strength to the fully softened shear strength upon wetting. Then, the possible colluvium nature of the slopewash and cyclic loading from the reservoir water level resulting in shear deformations sufficient to mobilize shear strengths between fully softened and residual values (Stark 1987; Stark and Duncan 1991).

Geology

A geologic cross-section of San Luis Dam at Station 135+00 is shown in Fig. 1. The bedrock consists of interbedded and faulted sandstone, shale, and conglomerate. The slopewash shown in Fig. 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 Fig. 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 Fig. 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 (Zone 7 in Fig. 1).

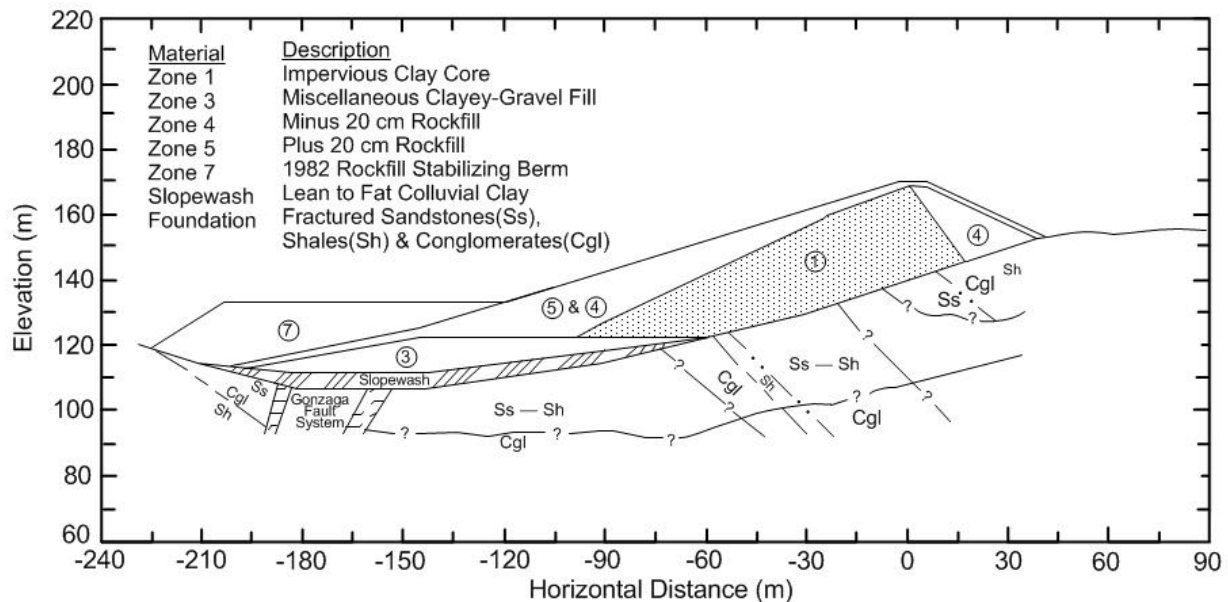


Figure 1. Geologic cross-section of San Luis Dam (after Stark 1987)

Reservoir Hydrograph and Piezometer Locations

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. Fig. 2 also identifies when the toe berm was completed to stabilize the upstream slope and multi-level 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) multi-level piezometers were installed in five (5) borings (see Fig. 3) to monitor pore-water pressures. Three

of the piezometers are located 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.

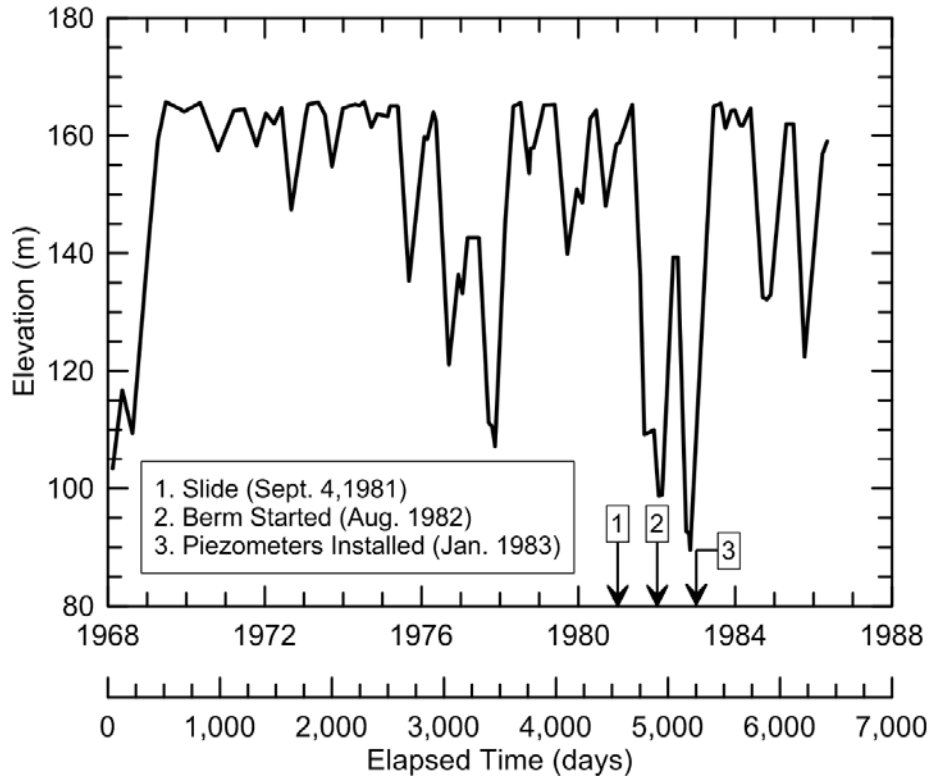


Figure 2. San Luis Reservoir hydrograph with (1) 1981 upstream slide, (2) berm construction started and was completed in 1983, and (3) piezometers installed

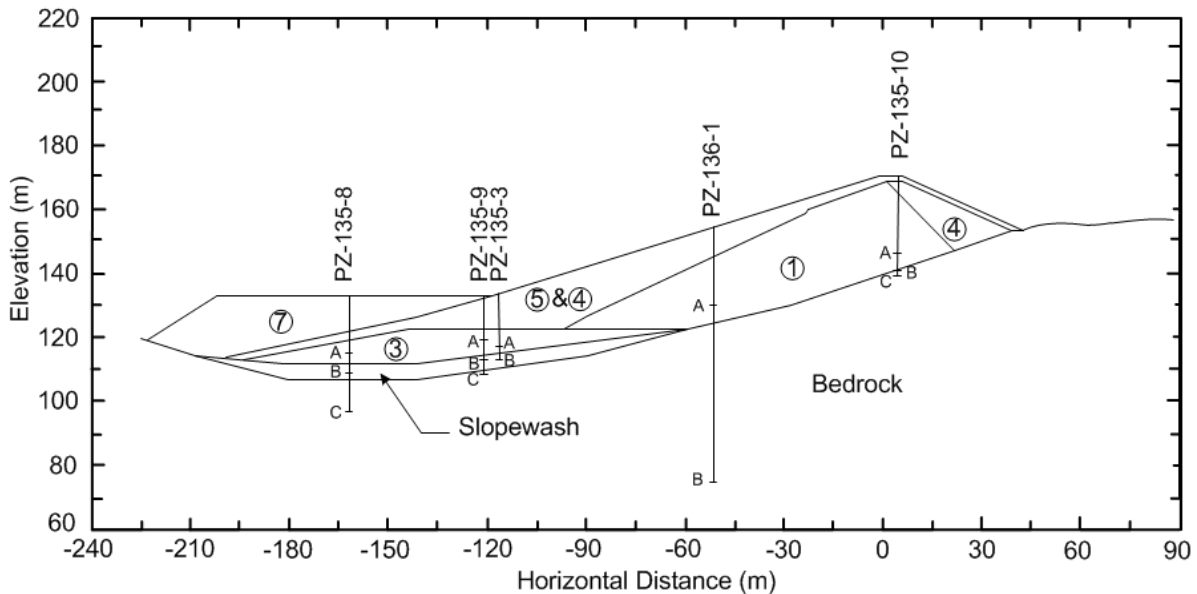


Figure 3. Location of piezometers installed after the 1981 slide

SLIDE SEEPAGE MODEL

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. SLIDE can model unsaturated hydraulic conductivities, water contents, and changes in water content as a function of pore-water pressure and are used to compare predicted pore-water pressures to the piezometers. The input method when incorporating initial conditions or “parent analyses” into staged analyses in SLIDE requires a separate file to input the pore-water pressure, total head, or pressure head grid from prior analyses as the initial groundwater 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 Figs. 1 and 3. 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, it is essential to define the initial groundwater and boundary conditions. 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 Fig. 4. The foundation piezometers (135-9C, 135-8C, and 136-1B) in Fig. 3 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 zero normal infiltration rate in SLIDE 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 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 Fig. 3) 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 downstream slope remained unsaturated, so the RHS boundary is modeled with a constant head of 132 m.

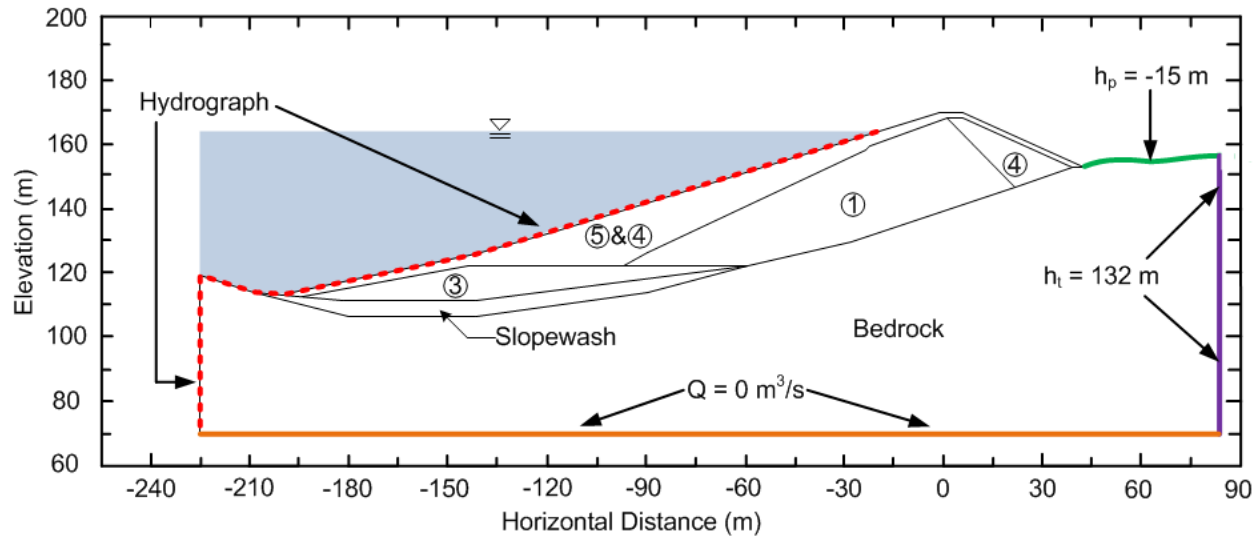


Figure 4. Boundary conditions applied for transient seepage model

Calibrated Soil Properties

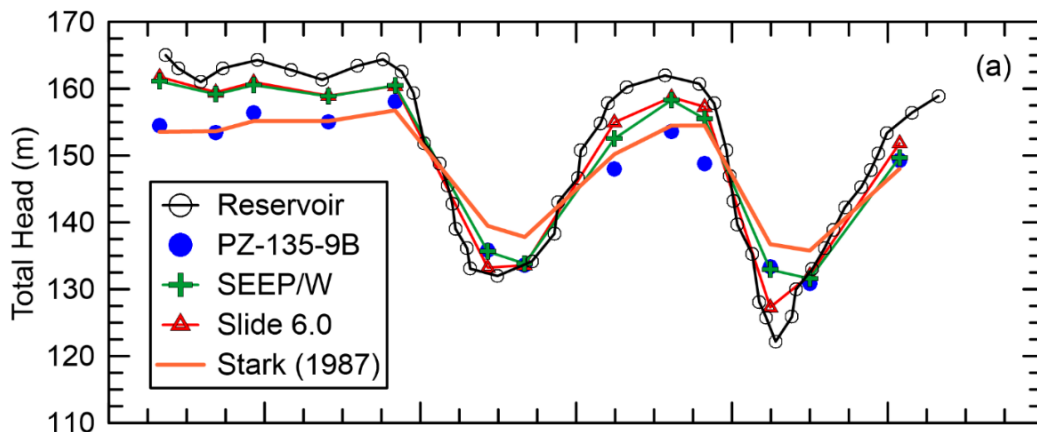
In an unsaturated and transient seepage analyses, four soil properties are required: (1) initial matric suction profile, (2) unsaturated hydraulic conductivity function (HCF), (3) saturated k_h/k_v ratio, and (4) soil compressibility (m_v). The soil engineering properties are based on laboratory and site data from Stark (1987) were used as a starting point for model calibration. Unsaturated soil properties are important for Zone 1 because the fine-grained core does not become fully saturated until after the piezometers are installed. 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 ensure increasing and decreasing reservoir levels are accurately captured. All other periods, e.g., constant reservoir capacity, used an increased time stepping of 90 days to reduce computing 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. Fig. 1 shows that the foundation consists of non-homogenous materials, including fractured sandstones, shales, conglomerates, and the Gonzaga Fault System. Due to uncertainty in spacing and orientation of these fractures, it is difficult to apply these materials in SLIDE. 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.

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. For example, Fig. 5 shows plots of total head versus elapsed time. Changes in m_v manifest in Fig. 5 by shifting or translating the total head data, and values of saturated k_h and k_h/k_v affect the total head magnitude, e.g., an increase in either parameter increases the pore-water pressure in the material. 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.

Fig. 5(a) shows the final calibration of the slopewash using PZ-135-9B. At time of reservoir capacity (5,650 to 5,950 days), SLIDE are in agreement but slightly under-predict the measured total heads. In subsequent refilling (6,150 to 6,350 days), SLIDE again slightly under-predict the measured total heads. At the end of the analysis, the calculated total head from SLIDE is lower than observed in the field. In summary, the calibrated slopewash parameters produce good agreement with field measurements during drawdown, which is key for analysis of the 1981 slide and refilling.

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 is 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. Fig. 5(b) shows the final calibrated results compared to the field observations for PZ-136-1A in Zone 1. The SLIDE model, with the exception of a small period between 6,300 and 6,350 days, slightly under-predicts the total head in Zone 1. The largest variance between field and calculated total heads occurs during periods of drawdown, which may be attributed to using one HCF for both the wetting and drying SWCCs.



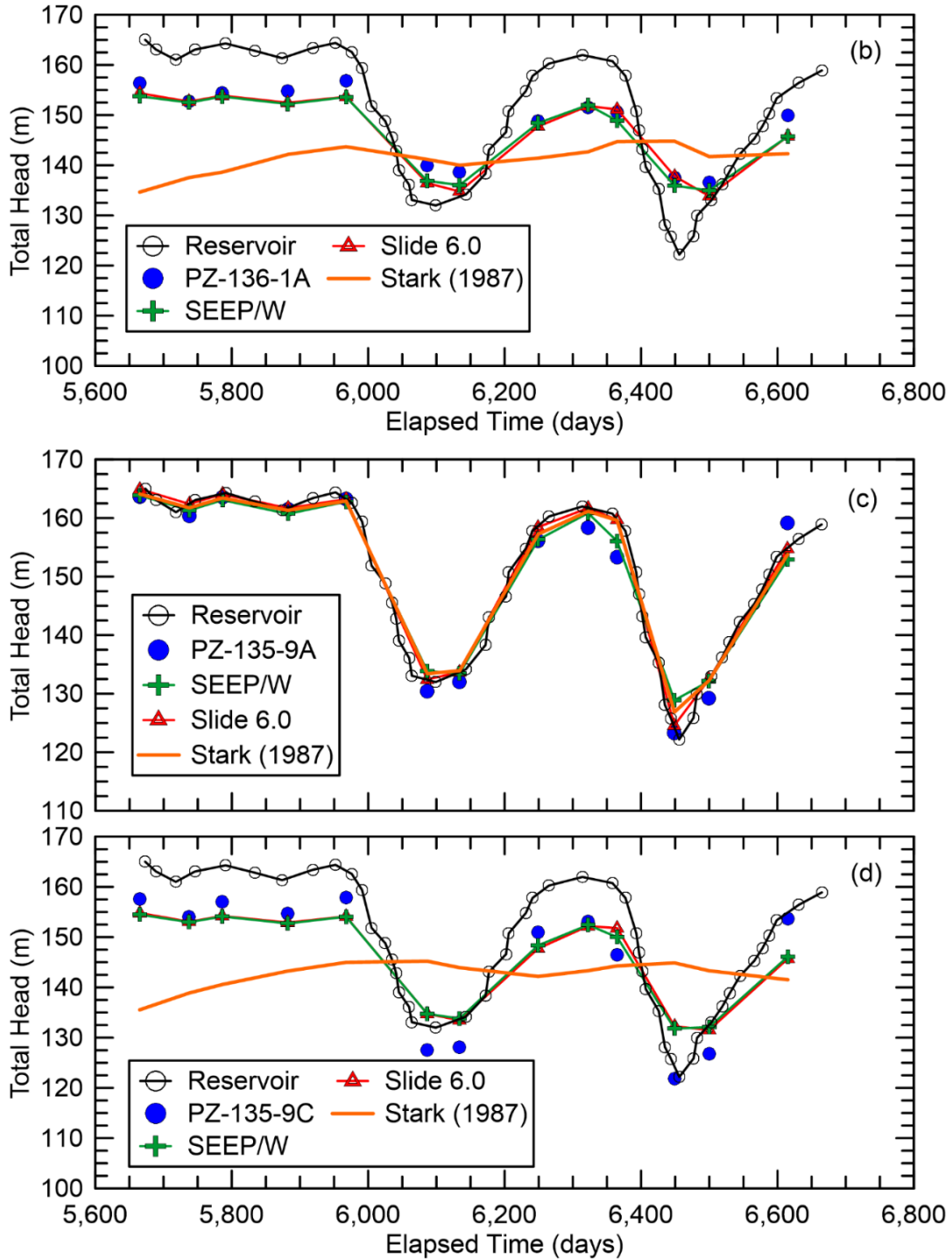


Figure 5. Total head response in: (a) Slopewash, (b) Zone 1, (c) Zone 3, and (d) Bedrock Foundation during piezometer monitoring period

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. Unlike the slopewash response, adjusting the foundation k_h resulted in a smaller influence on Zone 3 because the slopewash

underlies the Zone 3 and acts as a buffer for changes in the foundation. Fig. 5(c) shows the final calibration results for PZ-135-9A in Zone 3. PZ-135-9A in Fig. 5(c) indicates a total head similar to the reservoir level. SLIDE produces matching total heads except from elapsed time of 5,650 to 5,950 days. During this period, SLIDE slightly under-predicts the total head. After 5,950 days, the reservoir level decreases and SLIDE slightly over-predicts the total head in Zone 3. Difficulty in calibrating Zone 3 was attributed to less certainty in material parameters as opposed to the slopewash and Zone 1, which were tested (Von Thun 1985; Stark 1987). However, the calibrated Zone 3 parameters duplicate the field measurements during drawdown and refilling.

Fig. 5(d) compares the field (PZ-135-9C) and calculated total heads for the foundation. As indicated in Fig. 5(d), SLIDE over-predicts the total head during drawdown, but under-predict during filling and constant reservoir capacity. These inconsistencies could be attributed to the inability to model a non-homogenous material like the sheared and fractured foundation bedrock.

The calibration process of replicating piezometer readings and the interaction of soil layers resulted in the engineering properties shown in Table 1. Zone 1, Zone 3, and slopewash are assumed to have equal k_h/k_v ratios of two (2) while a value of unity (1.0) is assumed for the foundation. In addition, the compatibility of m_v and saturated k_h for the slopewash and Zone 1 materials is within limits recommended by Stark et al. (2014).

Table 1. Summary of calibrated seepage properties

Material	k_h (cm/s)	k_h/k_v	m_v (kPa ⁻¹)	θ_s	θ_R	$1/\alpha$ (m)	n
Zone 1	1×10^{-5}	2	8.35×10^{-6}	0.29	0.05	4.2	4.5
Zone 3	1.5×10^{-6}	2	1.0×10^{-6}	0.27	0.02	28.6	3.2
Slopewash	1.0×10^{-8}	2	3.5×10^{-6}	0.24	0.02	50	2.6
Foundation	1.4×10^{-4}	1	1.67×10^{-5}	--	--	--	--

EFFECT OF Van GENUCHTEN PARAMETERS

The unsaturated parameters α and n in the van Genuchten (1980) model played a significant role in the calibration process and should be emphasized in other unsaturated and transient seepage analyses. Zone 1 calibration relied on PZ-136-1A because no response was measured at PZ-135-10A, indicating that this portion of the fined-grained core remained unsaturated during the monitoring period. Because the van Genuchten (1980) parameters play a significant role in the calibration process, a parametric analysis of Zone 1 is provided to demonstrate the influence of α and n (see Fig. 6). Fig. 6(a) presents the reservoir hydrograph and piezometer response while Fig. 6(b) and 6(c) show the computed results using SLIDE when α ranges from 0.05 to 2 at $n=4.5$ and when n ranges from 1.5 to 4.5 with $\alpha=0.24$, respectively. Fig. 6(b) and 6(c) show that varying α has a greater influence than the parameter n on the difference between SLIDE predicted values and measured total heads, i.e., Difference (P-M).

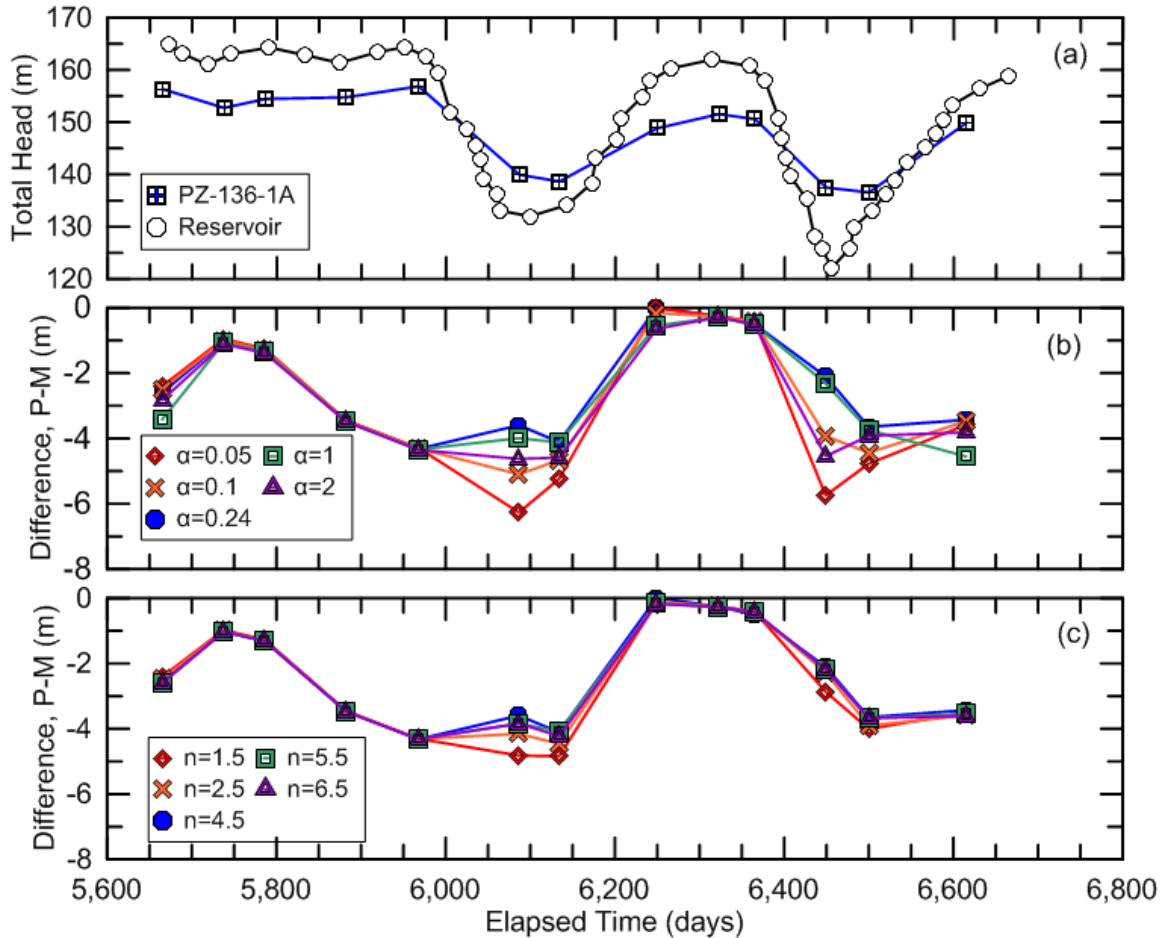


Figure 6. Sensitivity analyses of van Genuchten (1980) parameters α and n : (a) reservoir levels and piezometer data, (b) effect of α , and (c) effect of n

The first effect of parameters α and n is during periods of changing reservoir level, i.e., drawdown and filling. The largest discrepancy between predicted total head and observed piezometer measurements are found at elapsed time periods of 6,000 to 6,150 days and from approximately 6,350 to 6,650 days. Both time periods correspond to periods of drawdown and subsequent filling (see reservoir level in Fig. 6(a)). During constant reservoir levels, the SWCC and HCF corresponding to different α and n parameters provide nearly identical results.

The parameter α is related to the volumetric water content at which air first begins to enter the largest soil pores. As α decreases, the air entry condition also lowers so desaturation begins at lower matric suction. In the present study, α value of 0.24 m^{-1} results in the best fit to the field measurements. For values of α greater than 0.24 m^{-1} (1 or 2 m^{-1} in the parametric study), the difference in predicted and measured total heads, i.e., Difference (P-M), in Fig. 6(b) is more pronounced because a value of $\alpha = 0.24 \text{ m}^{-1}$ results in an equal or larger hydraulic conductivity at a specific suction compared to $\alpha = 1$ or 2 m^{-1} . This decrease in hydraulic conductivity for $\alpha = 1$ or 2 m^{-1} prevents pore-water pressures from draining at times of reservoir drawdown and contributes to the lag experienced when the reservoir begins to fill following drawdown.

The parameter n corresponds to the pore size distribution of the material and hence is related to the “S” shape curvature in the SWCC and HCF. The n values lower than the calibrated value of 4.5 affect the SWCC and HCF more than n values of 5.5 or 6.5 by decreasing the slope between saturated and residual volumetric contents. As a result, Fig. 6(c) shows the values of n at 1.5 and 2.5 create the largest difference between predicted and observed, and n values of 5.5 and 6.5 produce nearly identical total head values as the calibrated van Genuchten (1980) parameters in Table 1.

PHREATIC SURFACE

Another objective of the unsaturated and transient seepage analyses is to determine the progression of the phreatic surface through the Zone 1 fine-grained core, establish when steady-state conditions are achieved in the embankment, and evaluate the phreatic surface at slope failure. To achieve this objective, seepage through the core is analyzed using SLIDE with the calibrated soil properties shown in Table 1 and the boundary conditions in Fig. 4. The phreatic surface migration through the embankment at certain time periods of San Luis Reservoir operation is shown in Fig. 7 after 7.5 years. The phreatic surfaces after 7.5 years from Stark (1987) and SLIDE are comparable in Fig. 7(d). The SLIDE analyses show minor changes in the phreatic surface between 4.5 to 7.5 years while Stark (1987) analysis shows a continued progression through the center of the Zone 1 core. The inconsistent phreatic surfaces between this study and Stark (1987) are likely attributed to different impermeable foundation and HCF relationships. Still, the phreatic surface progresses in a similar manner in all three analyses, and the models indicate Zone 1 approaches steady-state conditions after eight years of reservoir operation.

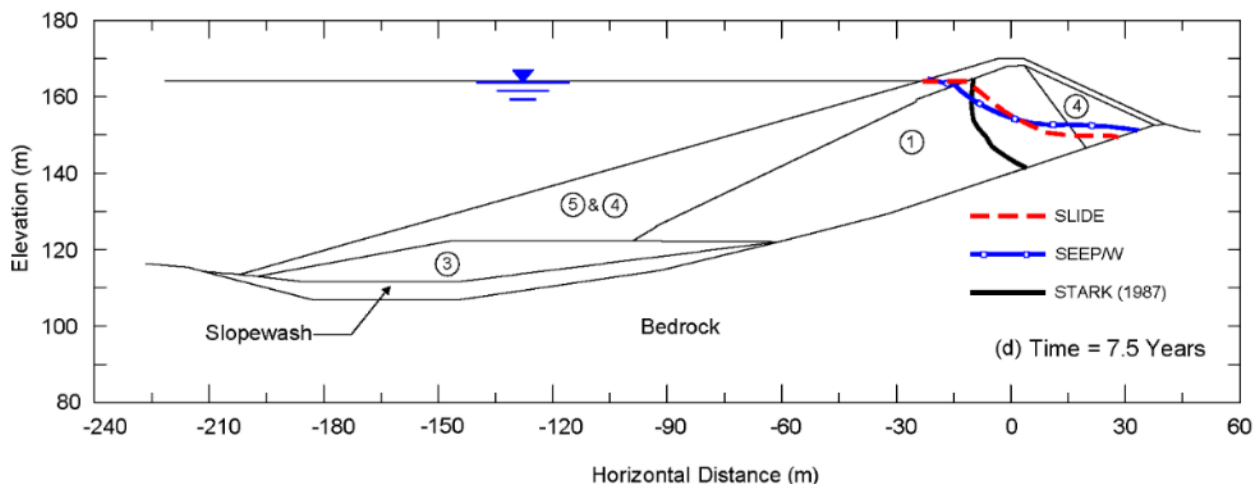


Figure 7. Phreatic surface after 7.5 years of reservoir operation

SUMMARY

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 SLIDE 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 , n , and m parameters to model the SWCC and HCF. Increasing α shifts the SWCC and HCF to lower matric suction without changing the overall shape of the curve while the parameter n steepens the SWCC and HCF slope between θ_s and θ_r . 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 than n .
- 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.

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The contents and views in this paper are those of the authors and do not necessarily reflect those of any dam or reservoir owner/operator, consultant, regulatory agency or personnel, or anyone else knowledgeable about the case study referenced. In particular, the contents of this paper/publication are the personal opinions of the author(s) and may not reflect the opinions, conclusions, policies or procedures of the U.S. Bureau of Reclamation or U.S. Army Corps of Engineers.

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