Mechanisms of Strength Loss in Stiff Clays

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Abstract: A slide in the upstream slope of California's San Luis Dam on September 4, 1981, was caused by a loss of strength in the stiff clay in the foundation beneath the slope. This loss in strength, which occurred over 14 years of the dam's operation, is not explainable in terms of commonly held views of the behavior of stiff clays. An investigation showed that the shear strength of the desiccated clay beneath the dam decreases very rapidly to the fully softened strength when the clay is soaked. When the clay is subjected to cyclic loading, as it was by cycles of reservoir filling and emptying, the strength of the clay decreases gradually from the fully softened strength to its residual value. These mechanisms of strength loss appear to have been responsible for the San Luis Dam slide, and should be considered in stability evaluations whenever stiff clays are subjected to wetting and cyclic variations in load.

Introduction

San Luis Dam is located approximately 100 mi (170 km) southeast of San Francisco, in the arid central valley of California. On September 4, 1981, after the reservoir had been drawn down 180 ft (55 m) in 120 days, a major slide occurred in the upstream slope. A photograph of the slide in early September is shown in Fig. 1. At the section where the slide occurred, the embankment is about 200 ft (60 m) high. Although the drawdown that preceded the slide was the longest and fastest in the life of the dam, there had been four previous large, rapid drawdown events, and two nearly as severe as the one in 1981.

Unlike most drawdown slides, the slide at San Luis Dam was deep-seated, and extended into the foundation beneath the upstream slope. As shown in Fig. 2, slope indicators installed through the slide mass revealed that the slide plane passed through the core of the dam, down into the clayey slopewash material in the foundation, then out toward the toe of the dam.

The slide was repaired by building a 60-ft- (18.3-m-)high buttress against the upstream slope ("Fast Fix" 1982), but questions remained concerning the fundamental causes of the slide. Stability analyses showed that the shear strength of the slope wash in the foundation would have had to be reduced to its residual value for the slide to occur. At the time, this finding was puzzling, because it conflicted with the established understanding of the shear-strength characteristics of overconsolidated clays. Previous experience (Skempton 1964, 1970, 1977) indicated that the "fully softened" shear strength (rather than the lower residual strength) would be appropriate for evaluating stability in this case, because there was no preexisting slip surface through the foundation. However, stability analyses performed using the fully softened strength resulted in factors of safety larger than unity, and were thus

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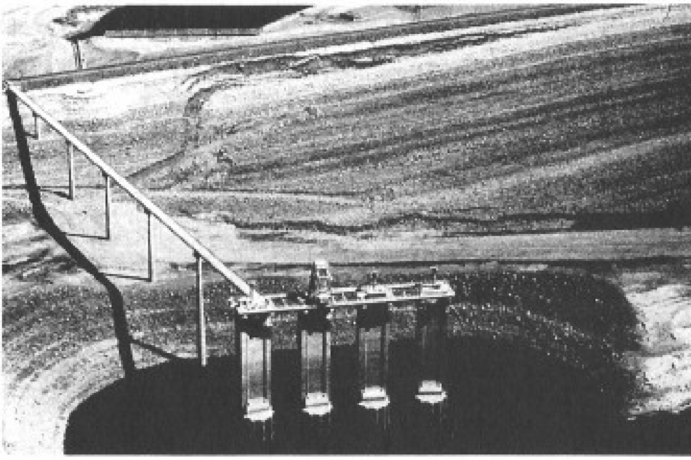

FIG. 1. San Luis Dam Slide in Early September 1981 (Photo Courtesy of California Department of Water Resources)

not consistent with the fact that the slope had failed.

To investigate this apparent conflict, a study was undertaken to examine the mechanisms through which the strength of the slope wash could have been reduced to the residual value over the 14 years of the reservoir's operation. The investigation involved extensive laboratory tests on undisturbed samples of the slope wash, finite element analyses to determine the seepage and stress conditions in the embankment and its foundation, and slope stability analyses to study the correspondence between foundation strength and the stability of the upstream slope.

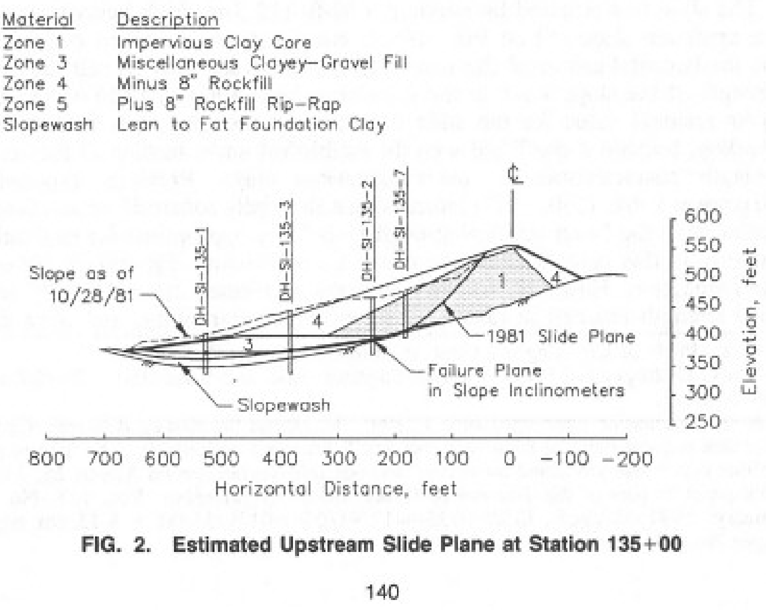

FIG. 2. Estimated Upstream Slide Plane at Station 135+00

Characteristics of the Slope Wash

The slope wash extended beneath the full length of the embankment affected by the slide. The slope wash is a medium- to high-plasticity clay derived from weathering and erosion of the surrounding hills. According to studies performed by the Bureau of Reclamation (*San Luis* 1974), the dominant clay mineral in the slope wash is beidellite, which exhibits swelling characteristics similar to those of montmorillonite. The liquid and plastic limits ranged from 35 to 45 and 18 to 24, respectively.

Downstream, where the slope wash had not been wetted, the natural water content was about 8%. Upstream, where it had been wetted by the reservoir, the water content had increased to about 19%. Both upstream and downstream, the slope wash is fissured, and contains rock fragments that range in size from fine sand to as much as 3 in. (1.2 cm). The rock fragments appear to float in the clay matrix without touching each other. The engineering properties of the slope wash are therefore governed by the clay, and are not much influenced by the rock fragments.

The slope wash blankets most of the hillsides near the dam. Its thickness varies from 5 ft (1.5 m) at the top of the hill to 20 ft (6 m) at the base. At the time of construction, the slope wash was highly desiccated and extremely strong in its dried state. The specification for stripping the foundation required that the slope wash was "to be removed to firm rock or to a horizon whose strength exceeds that of the overlying embankment." There is no doubt that this specification was met, because at the time of construction the slope wash was dry and hard, and undoubtedly stronger than the embankment fill to be placed on it. However, when the slope wash was wetted by the reservoir and subjected to cyclic loading as the reservoir rose and fell, it suffered considerable strength loss. This strength loss eventually led to the slide in the upstream slope.

Preparation of Test Specimens

Two undisturbed block samples of the slope wash were taken from the area downstream from the dam where the slope wash was still dry and hard, and two more were taken upstream, where it had been wetted by the reservoir and was considerably softer.

Consolidation and direct shear test specimens were trimmed from the upstream block samples in a moist room using a bow saw and single-edged razor blades. The bow saw was used to cut pieces from the block, and razor blades were used to trim test specimens from these pieces. The specimens were trimmed into specimen holders, which provided confinement and prevented the fissured soil from falling apart during trimming. The direct shear test specimens were 4 in. (10 cm) square by 0.8 in. (2.0 cm) thick. The consolidation specimens were 2.5 in. (6 cm) in diameter by 1.0 in. (2.5 cm) high. Each test specimen was extruded from the holder directly into the test apparatus. About three hours were required to trim each direct shear test specimen, and about one hour was required to prepare a consolidation test specimen.

The extremely hard downstream slope wash could not be cut with a bow saw. All the test specimens were trimmed directly from the sample blocks into the specimen holders using an electric drill fitted with a carbide-tipped

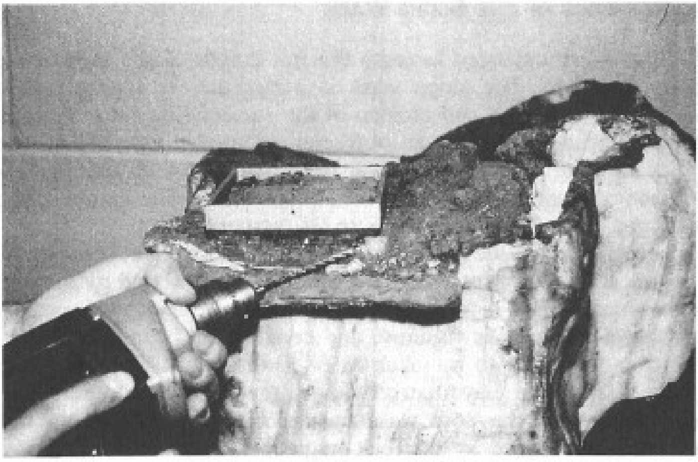

FIG. 3. Trimming Desiccated Slope Wash into Specimen Holder

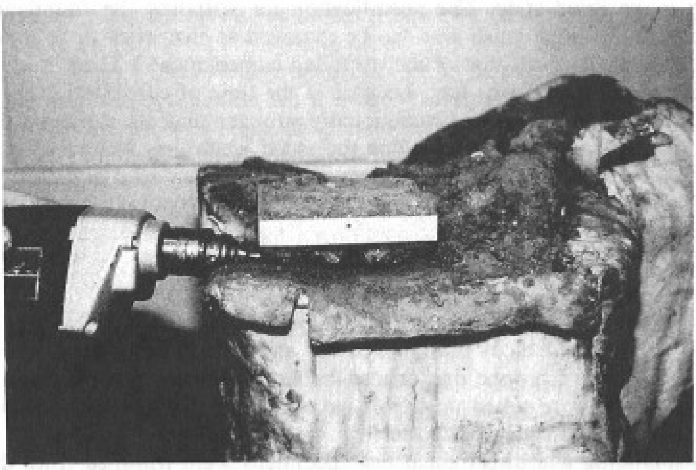

FIG. 4. Removing Desiccated Test Specimen from Slope-Wash Block Sample

drill bit as a sculpting tool. As shown in Fig. 3, the drill was used to excavate a sample slightly larger than the specimen holder. Razor blades were then used to carefully trim approximately 1/8 in. (0.3 cm) from around the specimen so the holder could slide over the specimen. The sculpting and trimming process was continued until about 1 in. (2.5 cm) of slope-wash material had passed into the holder. The test specimen was then cut from the block by drilling underneath the holder, as shown in Fig. 4. The top and bottom of the specimen were planned flush with the sample holder using the drill bit and razor blades. The downstream slope-wash specimens were trimmed in a constant-temperature, low-humidity room to minimize the change in moisture content. Seven to eight hours were required to trim a direct shear test specimen of the downstream material, while four to five hours were required to trim a consolidation test specimen.

Compressibility Characteristics

Upstream Slope Wash

The results of consolidation tests on upstream slope-wash specimens are shown in Fig. 5. One of the specimens was loaded before it was soaked (submerged in water). The specimen was kept moist by wrapping slightly damp cotton batting around the porous stone to prevent evaporation. The second specimen was soaked before testing. The compressibility characteristics of the two specimens are very similar, and that soaking the upstream specimens had little or no effect on their behavior. This is true because the upstream material had already been soaked in the field, by submergence beneath the reservoir.

The preconsolidation pressure of the specimen is about 1.0 ton per square foot (96 kPa), corresponding to a value of overconsolidation ratio (OCR) equal to 1.3. Values of OCR for other upstream specimens ranged from 1.0 to 1.8. The virgin curve slope shown in Fig. 5 was measured in tests on remolded specimens formed by mixing water with the portions of previously tested specimens that passed the number 100 sieve.

Downstream Slope Wash

The results of a one-dimensional compression test on a desiccated specimen from downstream, where the material had not been wetted by the res-

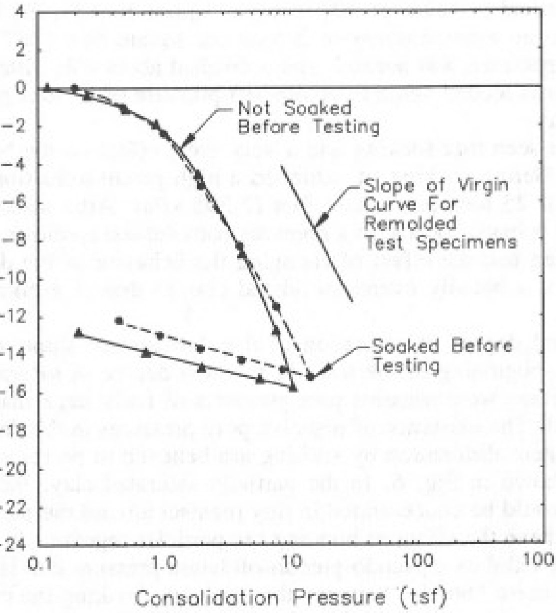

FIG. 5. Consolidation Tests Soaked and Unsoaked Upstream Slope-Wash Specimens

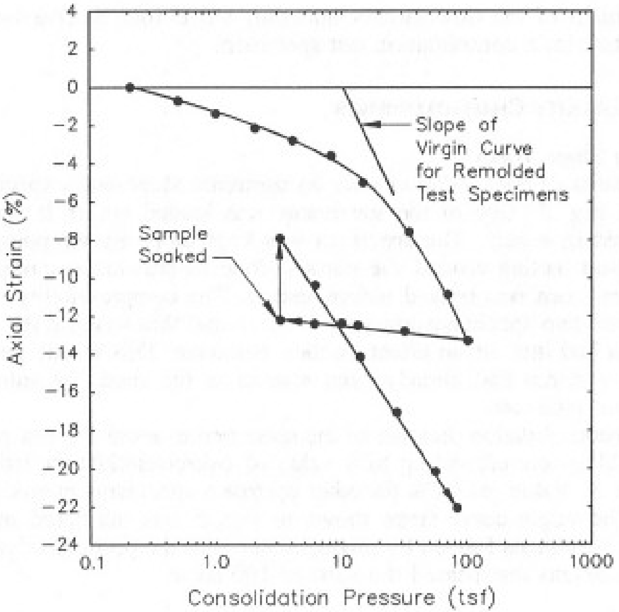

FIG. 6. Consolidation Test on Unsoaked Downstream Slope-Wash Specimen

ervoir, are shown in Fig. 6. The specimen was loaded in increments to an applied vertical pressure of 100 tons per square foot (9,580 kPa), then rebounded to an applied stress of 3 tons per square foot (287 kPa). At this stage the specimen was soaked, and it swelled about 4%. After soaking, the specimen was loaded again to an applied pressure of 85 tons per square foot (8,143 kPa).

It can be seen that soaking had a very great effect on the behavior of the specimen. Before soaking, it exhibited a high preconsolidation pressure, on the order of 25 tons per square foot (2,395 kPa). After soaking, however, it exhibited behavior typical of a normally consolidated specimen. Thus soaking the specimen had the effect of changing the behavior of the desiccated clay from that of a heavily overconsolidated clay to that of a normally consolidated clay.

The initial degree of saturation of the downstream slope wash was 50–60% at the beginning of the test. At this low degree of saturation, it seems likely that there were negative pore pressures of fairly large magnitude within the material. The existence of negative pore pressures in the desiccated slope wash and their elimination by soaking are believed to be responsible for the behavior shown in Fig. 6. In the partially saturated clay, the water within the voids would be concentrated in tiny menisci around the particle contacts, and would have the effect of bonding the particles together. In this condition the dry clay exhibits a pseudo-preconsolidation pressure that is related to the strength of these "bonds" between the particles. Soaking the clay eliminates the negative pore pressure, destroying the bonds. The clay appears to retain little memory for its stress history once it has been soaked. Even though the clay had a psuedopreconsolidation pressure of about 25 tons per square foot

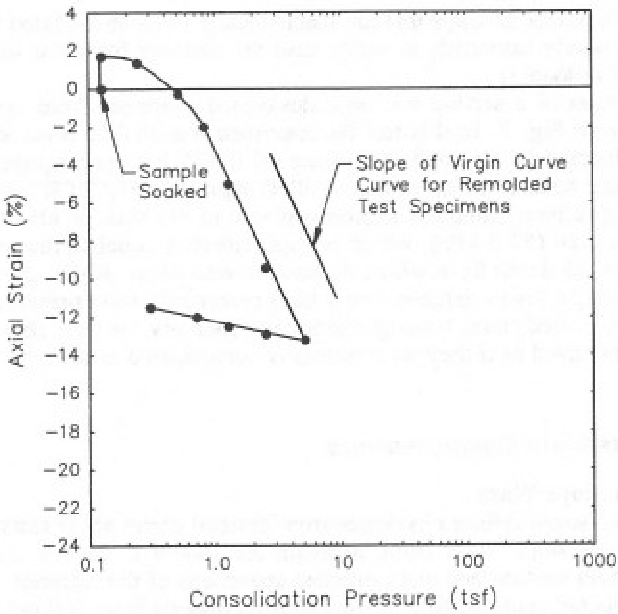

FIG. 7. Consolidation Test on Soaked Downstream Slope-Wash Specimen

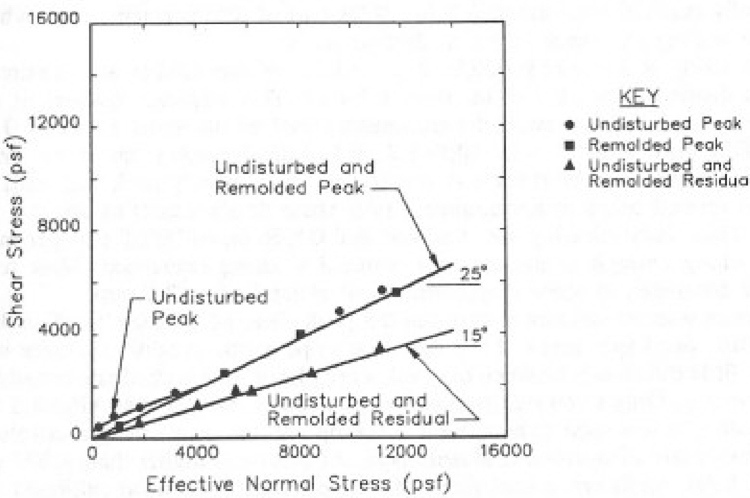

FIG. 8. Upstream Slope-Wash Failure Envelope

(2,395 kPa) in its desiccated state, and even though it was loaded mechanically to 100 tons per square foot (9,580 kPa) before soaking, it began to behave like a normally consolidated clay after it was soaked at an applied pressure of 3 tons per square foot (287 kPa). This behavior contrasts sharply with the behavior of clays that are mechanically overconsolidated while saturated or nearly saturated, in which case the memory for stress history persists after unloading.

The results of a second test on a desiccated specimen from downstream are shown in Fig. 7. In this test the specimen was soaked prior to loading, while subjected to an applied pressure of 0.125 tons per square foot (12 kPa). When soaked the specimen swelled approximately 2.0%. After soaking, the specimen exhibited a preconsolidation pressure of about 0.6 tons per square foot (57.5 kPa), which is approximately equal to the overburden pressure at the depth from which the sample was taken. Although the downstream test specimens exhibited very high preconsolidation pressures in their natural desiccated state, soaking erased their memory for their stress history, and they behaved as if they were normally consolidated as soon as they were soaked.

Shear-Strength Characteristics

Upstream Slope Wash

Effective stress failure envelopes from drained direct shear tests on specimens of the slope wash from upstream are shown in Fig. 8. Tests were performed on undisturbed and remolded specimens of the material. The tests were conducted using vertical normal stresses ranging from 100 psf (4.8 kPa) to 12,000 psf (575 kPa). By performing tests at various shearing rates, it was determined that 0.00096 in. (0.00038 cm) per minute was slow enough to allow full drainage during the tests. The peak shearing resistance was usually reached after about 0.2 in. (0.08 cm) of displacement, after which the shearing resistance began to decline slowly.

Shearing at a rate of 0.00096 in. (0.00038 cm) per minute was continued to a displacement of 2–3 in. (0.8–1.2 cm). This required reversal of the direction of shear because the maximum travel of the shear box was 0.55 in. (0.2 cm). After 2–3 in. (0.8–1.2 cm) of displacement, the motor drive was stopped, and the shear box was moved more rapidly back and forth by hand several times to accumulate a total shear displacement of about 8 in. (3.1 cm). Then shearing was resumed at 0.00096 in. (0.00038 cm) per minute—long enough to measure the residual shearing resistance. Most tests were continued to shear displacements of about 10 in. (3.9 cm).

There was no difference between the peak shearing resistances of soaked and unsoaked specimens of the upstream slope wash. In addition, there was very little difference between the peak strengths of undisturbed and remolded specimens. Only at normal stresses below about 3,000 psf (144 kPa) did the undisturbed test specimens exhibit a "bump" on the peak-strength-envelope characteristic of overconsolidated clays. At pressures higher than 3,000 psf (144 kPa), undisturbed specimens of the upstream slope wash exhibited the same behavior as remolded, normally consolidated specimens. Thus, at pressures above 3,000 psf (144 kPa), the peak strength of the upstream slope wash was equal to the fully softened strength as defined by Skempton (1964, 1970, 1977). Also, at those pressures, the peak strength parameters for the upstream slope wash were $c' = 0$ and $\phi' = 25°$, as shown in Fig. 8.

The residual strengths of undisturbed and remolded specimens of the upstream slope wash were also the same. As shown in Fig. 8, the residual strength envelope corresponds to $c' = 0$ and $\phi' = 15°$.

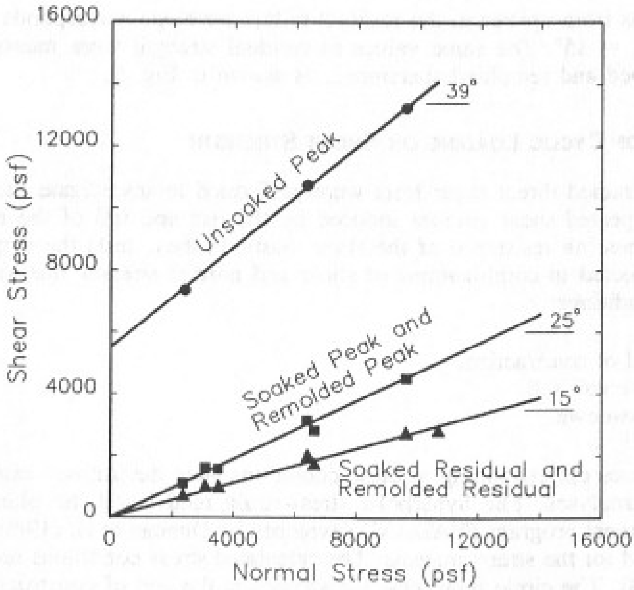

FIG. 9. Downstream Slope-Wash Failure Envelopes

Downstream Slope Wash

Failure envelopes from drained direct shear tests on undistrubed and remolded specimens of the downstream (desiccated) slope wash are shown in Fig. 9. The normal stress shown on the horizontal axis in Fig. 9 is the applied vertical stress. The effective stresses in the unsoaked specimens were probably considerably larger than the applied vertical stresses due to the negative pore pressures in the specimens. In the soaked specimens the effective normal stresses would be equal to the applied vertical stresses, since the negative pore pressures were eliminated by soaking.

The test procedures used in tests on the downstream slope wash were the same as described previously. The specimens were sheared at 0.00096 in. (0.00038 cm) per minute until the peak shearing resistance had been passed, usually to a shear displacement of 2–3 in. (0.8–1.2 cm). Then the shear box was moved back and forth more rapidly to accumulate shear displacement of about 8 in. (3.1 cm), and shearing at a rate of 0.00096 in. (0.00038 cm) per minute was resumed long enough to measure the residual strength value.

It can be seen that the desiccated slope wash was extremely strong. The peak strength envelope shown in Fig. 9 corresponds to $c' = 5{,}500$ psf (263 kPa) and $\phi' = 39°$. However, as soon as the desiccated material was soaked, its strength decreased to the value measured for the soaked specimens from upstream. As shown in Fig. 9, the peak strength envelope for soaked specimens corresponds to $c' = 0$ and $\phi' = 25°$. The same values were found for undisturbed and remolded specimens.

After shear displacements of 9–10 in. (3.5–3.9 cm), the shear strength of the slope wash decreased to its residual value. As in the case of the soaked specimens from upstream, the residual failure envelope corresponds to $c' = 0$ and $\phi' = 15°$. The same values of residual strength were measured for undisturbed and remolded specimens, as shown in Fig. 9.

Effect of Cyclic Loading on Shear Strength

Two drained direct shear tests were performed to investigate the effects of the repeated shear stresses induced by the rise and fall of the reservoir on the shearing resistance of the slope wash. In these tests the slope wash was subjected to combinations of shear and normal stresses that simulated these conditions:

- End of construction.
- Reservoir full.
- Drawdown.

The stresses corresponding to these conditions were determined using finite element analyses. The hyperbolic stress-strain model and the plane strain finite element program FEADAM, developed by Duncan et al. (1980, 1984), were used for the stress analysis. The calculated stress conditions are shown in Fig. 10. The circle represents the stresses at the end of construction, before the reservoir was raised. The square represents the stresses after reservoir filling. The effect of reservoir filling is to reduce both the shear stress and the effective normal stress in the slope wash. The triangle represents the stresses at the end of drawdown. The shear stress is the same as at the end of construction, but the effective stress is slightly lower because there were some undissipated excess pore pressures at the end of drawdown. These excess pore pressures were estimated using the finite element seepage program UNSAT1 developed by Neuman (1972). The seepage parameters were obtained from laboratory testing and calibrated using field piezometer data

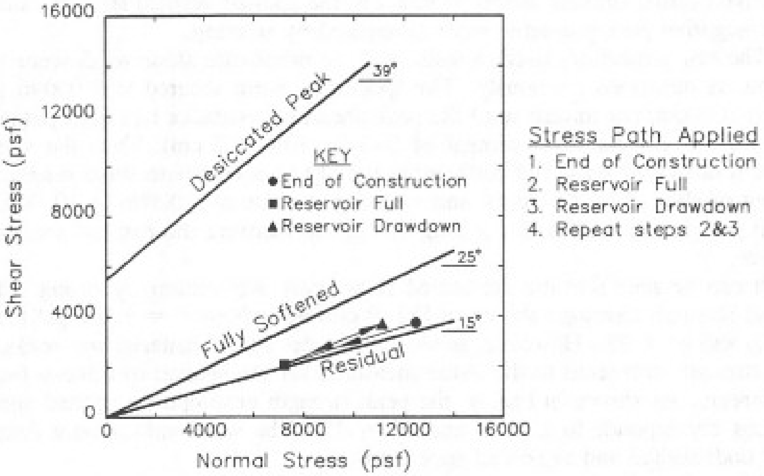

FIG. 10. Stress Path Applied in Repeated Load Direct Shear Tests

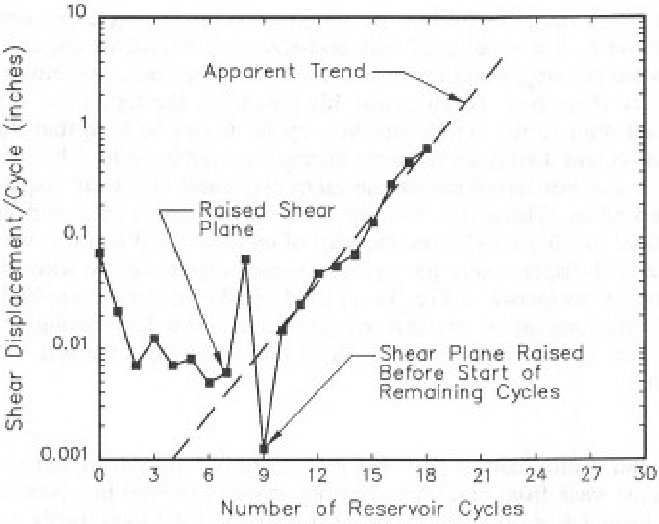

FIG. 11. Horizontal Displacement per Reservoir Cycle in First Repeated Load Direct Shear Test

measured during the 1984 and 1985 drawdowns, after the slide had been repaired.

Test 1

The first repeated load direct shear test was conducted using an undisturbed specimen of the desiccated slope wash from downstream. In the first stage of the test, the specimen was subjected to the end of construction stresses. These caused very little deformation because the specimen was dry and very strong. When the shear stress had increased to the end of construction value, the test was stopped and the sample was soaked for one day to simulate the filling of the reservoir. As soon as water was added to the shear box, the applied shear stress dropped approximately 50% with no change in shear displacement. When the effective normal stress had been reduced to the reservoir full value, a shear displacement rate of 3.8×10^{-5} in. (1.5×10^{-5} cm) per minute was required to maintain a shear stress equal to the reservoir full value.

After the reservoir's full stress condition had been maintained for 24 hours, the normal and shear stresses were increased to simulate drawdown. The drawdown shear stress was applied for three hours. During this period the slope wash had to be sheared continuously at a rate of 1.9×10^{-4} in. (7.5×10^{-5} cm) per minute to maintain the desired shear stress.

To simulate the rise and fall of the reservoir the slope wash was subjected to repeated cycles of the reservoir full and the drawdown stress conditions. During each reservoir cycle, the reservoir's full condition was maintained for three hours, because the specimen was able to maintain this value of shear stress with little or no displacement. The drawdown stress condition was applied for approximately 20 hours during each cycle.

Fig. 11 shows that the horizontal displacement per cycle decreased during the first seven reservoir cycles. Based on previous testing experience with the slope wash, it was believed that sand-size rock fragments might be binding between the upper and lower halves of the shear box, inhibiting movement of the shear box. To eliminate this possibility the top of the shear box was raised prior to the eighth reservoir cycle. It can be seen that the shear deformation was significantly larger during the eighth cycle. The top of the shear box was not raised before the ninth cycle and the shear displacement decreased again. Thereafter, the top of the shear box was raised approximately 0.03 in. (0.1 cm) before the start of each cycle. When this was done, the horizontal displacement per cycle increased exponentially with the number of cycles, as shown in Fig. 11. A total of 18 cycles were applied before a significant amount of the test specimen had been lost through the gap between the top and bottom of the shear box and before the test had to be terminated.

Test 2

A second cyclic loading test was performed on an undisturbed specimen of the slope wash from upstream. The first stage of the test involved shearing the specimen 1.9 in. (0.7 cm), approximately the final total displacement in the first test. The specimen was then subjected to eight load-unload cycles to simulate rise and fall of the reservoir. The displacement during the second cycle was quite small compared to that measured during the first cycle. Before the third and all subsequent cycles the top of the shear box were raised 0.03 in. (0.1 cm) to prevent binding on sand-size rock fragments. As in the first test, the displacement per cycle was found to increase with each cycle. After the eighth reservoir cycle, the specimen was sheared at 0.00096 in. (0.00038 cm) per minute using the standard direct shear procedure. The measured shear resistance was found to be halfway between the fully softened and residual strength envelopes.

Evaluation

The cumulative displacements measured in these tests are shown in Fig. 12. When the results of the first test are shifted to eliminate the first seven cycles where the shear box was binding on sand-size particles, the results of both tests are very nearly the same. On the basis of these results it can be seen that cyclic loading of the slope wash results in continual shear displacement. It appears that, with a sufficient number of cycles, the cumulative displacement could eventually become large enough [approximately 10 in. (3.9 cm)] to reduce the shearing resistance to its residual value. For the conditions in the cyclic loading tests discussed in the preceding paragraphs, about 15 cycles would be required.

During operation of the San Luis reservoir from 1968 to 1981, there were four large and eight small drawdown cycles. Thus, even if all 12 cycles are considered, Fig. 12 indicates that the cumulative displacement would probably not be more than 3 or 4 in. (1.2 or 1.6 cm), perhaps not enough to reduce the strength of the slope wash to its residual value.

The conditions involved in the cyclic loading tests do not correspond to the field conditions with respect to the length of time involved in the loading cycles, however. Because of the longer duration of the drawdown cycles in the field, it would be expected that more deformation would occur during each cycle in the field than in the laboratory. Each cycle of drawdown stress

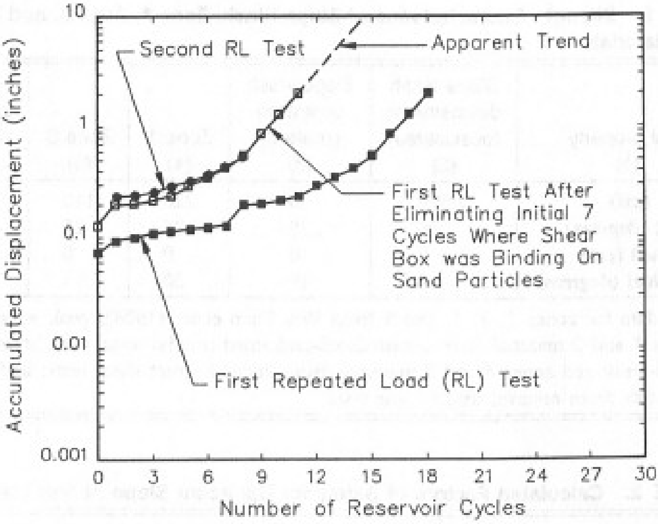

FIG. 12. Total Displacement per Reservoir Cycle in Both Repeated Load Direct Shear Tests

in the laboratory tests was applied for about 20 hours, and at the end of this time, when the reservoir full stresses were applied to the specimen, deformation was still occurring. Each cycle persisted for a longer period in the field, and would therefore have resulted in more shear displacement. It therefore seems likely that the shear strength of the slope wash in the field was reduced from its fully softened strength to the residual value as a result of cumulative shear displacements caused by cyclic loads due to filling and emptying the reservoir. It is possible that the cracks observed in the crest roadway between 1978 and 1981 were in fact early indications of the movements that led to the loss of strength and occurrence of the slide.

Stability Analyses

Values of the peak and residual strength parameters for the slope wash and the other materials in the dam are summarized in Table 1. The tests on the materials from zones 1, 3, 4, and 5 were performed by the Bureau of Reclamation and reported by Von Thun et al. (1984). These values were used to calculate factors of safety for the upstream slope at the end of construction, the reservoir full condition, and the drawdown condition. The analyses were performed using Spencer's method, which satisfies all conditions of equilibrium, and all calculated values of factor of safety for the observed slip surface, which is shown in Fig. 2. The results of the slope stability analyses are summarized in Table 2.

At the end of construction, with the slope wash dry and the reservoir down, the calculated factor of safety was 4.0. For the reservoir full condition, with the slope wash soaked and fully softened, the calculated factor of safety decreased to 2.0. At the end of drawdown, with the pore pressures determined from finite element transient seepage analyses and the fully soft-

TABLE 1. Strength Characteristics of Slope-Wash, Zone 1, Zone 3, and Zones 4 and 5 Materials

Soil property (1)	Slope wash downstream (desiccated) (2)	Slope wash upstream (soaked) (3)	Zone 1 (4)	Zone 3 (5)	Zones 4 and 5 (6)
c' peak (psf)	5,500	0	220	110	0
ϕ' peak (degrees)	39	25	25	25	40
c' residual (psf)	0	0	0	0	—
ϕ' residual (degrees)	15	15	20	15	—

Note: data for zones 1, 3, 4, and 5 from Von Thun et al. (1984); peak strength data for zones 1 and 3 material from consolidated-undrained triaxial tests; peak strength data for slope-wash and zones 4 and 5 materials from drained direct shear tests; and residual strength data from reversal direct shear tests.

TABLE 2. Calculated Factors of Safety for Upstream Slope of San Luis Dam

Condition (1)	Factor of safety (2)
End of construction (slope wash dry)	4.0
Reservoir full (slope wash soaked, fully softened strength)	2.0
Reservoir drawn down (slope wash soaked, fully softened strength)	1.3
Reservoir drawn down (slope wash soaked, residual strength)	1.0

ened strength in the slope wash, the calculated factor of safety was 1.3. When the drawdown condition was analyzed using the same pore pressures together with residual strength in the slope wash, the calculated factor of safety was 1.0.

Thus, if the fully softened strength is used to analyze the stability, the slide cannot be explained, because the calculated factor of safety is equal to 1.3. Only if the residual strength is used in the slope wash is the calculated factor of safety equal to 1.0. It thus seems clear that the strength of the slope wash must have been at its residual value during the 1981 drawdown, otherwise the slide would not have occurred.

As discussed previously, it seems likely that the strength of the slope wash was reduced from its fully softened strength to the residual value as a result of the cyclic loading imposed on it as the reservoir was filled and emptied. These cyclic stress changes appear to have resulted in cumulative deformations in the slope wash that were large enough to reduce the strength to residual.

Conclusions

This examination of the conditions leading to the slide in the upstream slope of San Luis Dam in 1981 has led to the following conclusions regarding mechanisms of strength loss in stiff clays:

1. The strength of the desiccated slope wash in the foundation of San Luis Dam was very high when the material was in its dry state. When the slope wash was wetted, its strength was reduced immediately to the fully softened value. This behavior is different from that of mechanically overconsolidated clays. The properties of these clays are strongly influenced by their preconsolidation history. Skempton (1977) analyzed a number of cases in which periods of time varying from 16 years to 80 years were required for the strengths of mechanically overconsolidated clays to decline to their fully softened values. In contrast, the slope wash, which is overconsolidated by drying, reverts almost immediately to a fully softened condition as soon as it is soaked. Only in tests at pressures below 3,000 psf (144 kPa) did the soaked slope wash show signs of its previous overconsolidation by drying.

2. When subjected to cyclic loading, the slope wash deformed continually. It appears highly likely that the slide in the upstream slope of San Luis Dam occurred as a result of deformations that developed when the reservoir was filled and emptied, eventually reducing the shearing resistance of the slope wash from its fully softened strength to the residual value. To the writers' knowledge, this mechanism of strength reduction to the residual value through cyclic loading has not been noted previously. It is, however, of potential importance any time a stiff clay will be subjected to cyclic variations in stress due to fluctuations in reservoir level or other causes. In such cases it appears advisable to perform tests on the material to determine if the cycles in loading to which it will be subjected will result in continual deformation and loss of strength. Alternatively, the loading on the clay can be reduced so that failure would not occur even if the strength was reduced to the residual value.

Acknowledgment

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Appendix. References

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