Long Term Failure in Compacted Clay Slopes

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SYNOPSIS The results of field, laboratory, and analytical investigations of recurring slope failures along the Mississippi River Levee are presented. Observations from slide trenching operations are described. Laboratory measured shear strengths are compared to effective strengths at failure "back-calculated" for two slides. Factors influencing the long term behavior of the levee materials and the mechanism of failure are discussed.

INTRODUCTION

Shallow slough slides have been occurring along the mainline Mississippi River Levees for the past 40 years. Approximately 160 such slides have been repaired by the U. S. Army Engineer District, Vicksburg, since 1964. Although these slides do not threaten the integrity of the levee system, they do pose recurring maintenance problems.

Levee sections in the Lower Mississippi River Levee system range in height from 25 to 30 feet with a 1V on 3.5H to 1V on 4.0H riverside slope. The landside slope varies from 1V on 5.5H to 1V on 6.0H. Most of the levee system was brought to current grade in the 1940's. Design specifications required an impervious riverside blanket of silt or clay with a minimum thickness of 5 feet to control thru-seepage. The thickness of impervious material generally exceeds 5 feet. Local borrow used for levee construction and enlargements was Recent alluvial deposits generally consisting of silts and clays. Common to these deposits are highly plastic clays (CH) known locally as "gumbo" and referred to as Sharkey clay by agronomists. It has been determined that slough slides occur only in reaches where the impervious blanket has been constructed entirely of highly plastic clay.

The highly plastic clay on the levee slope is highly overconsolidated by dessication. During dry periods, surface cracking is widespread and repeated cycles of wetting and drying produce a blocky slickensided structure to a depth of 5 to 7 feet. The slides occur in this weathered zone 20 to 40 years after construction and appear to be triggered by heavy rainfall. The maximum depth of the slides varies from 4 to 7 feet. They occur primarily in the riverside slope between the crown and a point midway down the slope and range in length from 100 to 300 feet. A typical slide is shown in Figure 1.

Fig. 1. Typical Levee Slough Slide.

FIELD INVESTIGATION

Trenched Slides

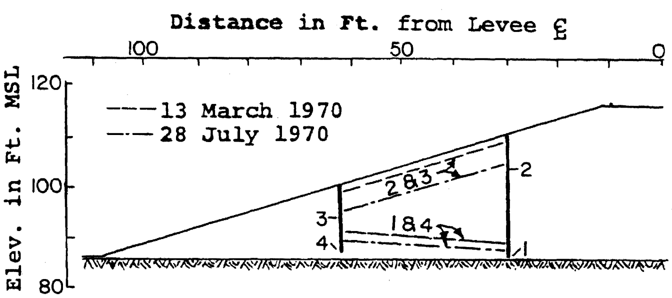
Eleven slides were trenched during the period 1968 to 1981. Eight were fully developed slides and three slides were in the initial stage of development. The trenches were excavated in order to observe the slide plane and soil composition and to obtain undisturbed samples. The trenches were excavated through the center of the slides in a direction perpendicular to the levee centerline. The maximum depth of excavation ranged between 8 and 10 feet and was generally governed by the depth to obviously competent material. Excavation was accomplished by either backhoe or dragline.

The material was carefully inspected as the excavation progressed to determine soil type, the distribution of fissures and slickensides, the location of the slide plane and any other features pertinent to analysis of the slide. Undisturbed and general soil samples were obtained in each of the trenches. The undisturbed samples were obtained by pushing a 5-inch O.D. by 1-foot-long stainless steel tube with either a dozer blade or backhoe bucket. After pushing, the tubes were removed manually. The samples were then extruded and sealed in cardboard jackets with paraffin.

Cross sections were run across each slide prior to trenching. After trenching, profiles were run along the bottom of the trenches and along the observed slide planes and all samples were located.

Piezometers

Four Casagrande-type piezometers were installed in the levee slope within an area that has experienced numerous slough slides. A schematic of the piezometer arrangement and tip elevations is presented in Figure 2. Readings were made over a 7-month period. Throughout this period the data indicated that a perched water table existed within the weathered zone above the intact unweathered material and that the water table was at its peak during the wettest season of the year. The piezometric surfaces shown in Figure 2 represent maximum and minimum piezometric conditions observed over the period the piezometers were monitored.



PIEZ NO.	RISER ELEV. FEET MSL	TIP ELEV. FEET MSL
1	111.9	87.4
2	111.9	104.9
3	100.7	94.7
4	100.7	88.7

Fig. 2. Piezometric Conditions in Highly Plastic Clay Embankment.

LABORATORY INVESTIGATION

Classification and Index Tests

The material in all slides investigated was a highly plastic clay having a liquid limit greater than 60 and a plasticity index greater than 40. According to Dakshanasmurthy and Raman (1973), Atterberg limits of the 39 specimens tested indicate the clay is highly expansive to extra highly expansive. X-ray diffraction analysis on samples recovered from nine slides showed that the most common clay mineral in each sample was montmorillonite. Montmorillonite and quartz were found to be the primary constituents of each sample. Grain size analyses indicated at least 90 percent passing the number 200 sieve with 20 to 60 percent finer than 0.002 mm. A majority of the samples tested indicated 50 percent or more finer than 0.002 mm.

Shear Strength Tests

Unconfined compression tests were performed on samples from three standard Proctor compaction tests and samples from a 7-blow compaction test. Shear strengths (c) at optimum water content achieved under these varied compactive efforts ranged from 0.70 tsf to 1.24 tsf.

Triaxial tests were performed on relatively undisturbed, intact samples recovered from the trenched slides to determine the available peak strengths under drained and undrained conditions. The samples tested were recovered from the vicinity of the slide plane. In general, the samples contained fissures and when broken exhibited a blocky structure. Unconsolidated-undrained (UU) tests indicated undrained internal friction angle (ϕ) values from 0 to 4 degrees with cohesions ranging from 0.23 tsf to 0.54 tsf. The measurement of a friction angle is attributed to the effects of fissures and slickensides. Results of consolidated-undrained (CU) tests indicated a range of undrained ϕ values of 13.4 to 18 degrees with cohesions ranging from 0.09 to 0.24 tsf. Two consolidated-drained (CD) direct shear tests indicated drained internal friction angle (ϕ') values of 18 and 16 degrees with associated cohesions of 0.05 and 0.16 tsf, respectively.

Residual strengths were determined for three samples by means of two drained repeated direct shear tests and one annular shear test. Consolidated-drained repeated direct shear tests resulted in residual drained internal friction angle (ϕ_R') values of 7.0 and 9.0 degrees with associated cohesions of 0.06 and 0.05 tsf, respectively. The annular shear test indicated a ϕ_R' of 8.6 degrees with no cohesion. Average results from these shear tests are shown in Figure 3.

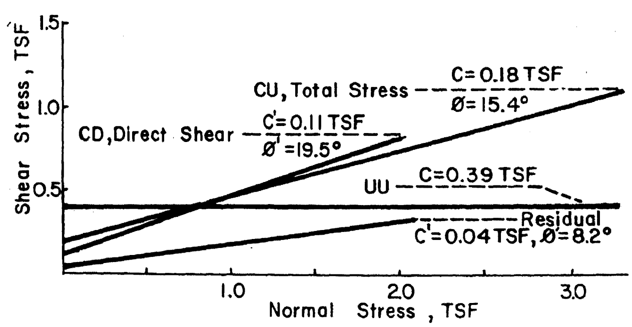


Fig. 3. Summary of Average Shear Strength Envelopes.

Cylic Shrink-Swell Test

A cyclic shrink-swell test was performed on an undisturbed sample recovered from near the slide plane of a trenched slide. The results of this test are presented in Figure 4.

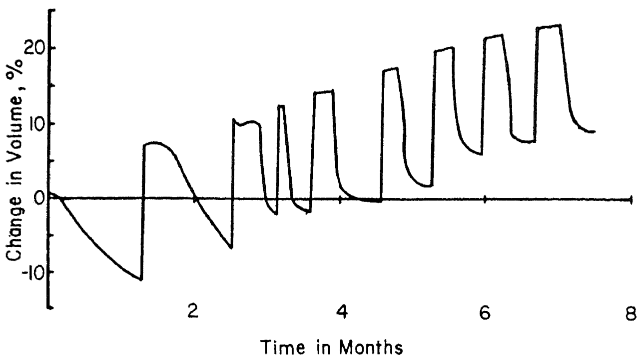


Fig. 4. Results of Cyclic Shrink-Swell Test.

It can be seen that as the number of wet-dry cycles increases there is a corresponding increase in swell volume and that a relative equilibrium where shrinking and swelling occur between constant limits had not been reached even after 9 cycles. Popesco (1980) attributes this phenomenom to progressive slaking which augments the swelling-shrinking ability of the soil. Tests presented by Popesco show that an equilibrium is eventually reached although it might take a number of years to reach this point of stabilization in the field.

OBSERVATIONS OF TRENCHED SLIDES

General

Many characteristics observed in trenching the fully and partially developed slides were common to both. The material type common to all slides investigated was a highly plastic clay (CH). No other material type was encountered above the slide plane in any of the slides trenched. Surface cracks were observed in areas adjacent to all slides except those slides trenched during wet weather. The cracks varied in width at the surface from a fraction of an inch to 2 to 3 inches. Generally, the width of the cracks appeared to be a function of the weather; the drier the weather, the wider the cracks.

Distinct soil structure patterns were observed in each slide trenched. Basically there were three different types of structures: (1) a "cubic" structure, (2) a "platey" structure, and (3) a "buckshot" structure. Both the cubic and platey structure patterns were observed in some of the slides, but usually one type would be much more prevalent than the other. In most slides only one of these structures was observed. The buckshot structure was observed in all slides, but was usually restricted to the top 3 to 4 inches at the surface.

The cubic structure in the profile view looked much like a wall constructed of square blocks. The size of cubes varied from approximately 1/4 to 3/4 inch. Generally, the material within the cube was stiff, but each cube had a layer of soft, wet material around the outside. It was noticeable that even though the individual cubes were stiff, when pressure was applied to a piece of soil containing an aggregate of many cubes, the consistency appeared to be that of a soft soil. Also free water was frequently encountered in the fissures between the cubes, and the surfaces of the cubes frequently showed traces of iron stains. The presence of free water could in part be attributed to the structure distortion which occurred during failure and rainfall that was trapped in the slide in the interim.

The platey structure was the most prevalent type of structure and similar to the cubic structure except for the shape. The plates generally had a thickness of approximately 1/4 to 1/2 inch and a length to width ratio of 4 to 5. The platey structure also showed traces of free water and iron stains within the structure.

The buckshot structure occurred in most slides at the ground surface in the top 1 to 4 inches. The size of the "buckshot" was generally on the order of a coarse sand to fine gravel. In many parts of the world the buckshot structure is referred to as a crumb structure (Popesco, 1980). Because it occurs at the surface, a large amount of research has been conducted in the areas of agricultural engineering and soil-water-plant relations. The buckshot structure is important only because of its granular characteristics and the fact that it is easily knocked into cracks by livestock and eroded into cracks by rains.

Fully Developed Slides

The primary differences in the fully developed slides and slides in the initial stages of development were in characteristics of the slide planes. The typical soil profile encountered in fully developed slides consisted of 1 to 4 inches of soil with a buckshot structure at the surface. Underlying this was soil with either a cubic or platey structure, occasionally combinations of both types, down to the slide plane. The cubic or platey structure was also noted below the slide plane in some of the slides but the structure was not nearly so well developed as above the slide plane. The material immediately above the slide plane, 1 to 3 inches, was usually very soft and wet and appeared to be highly remolded. The degree of difficulty in locating a distinct slide plane depended on the softness and thickness of the remolded zone surrounding the slide plane. If the remolded zone was relatively thin and only slightly soft, a distinct slide plane could be located easily. As the thickness and softness increased, the difficulty of locating a distinct slide plane increased. Immediately below the slide plane the material was generally stiff and showed no evidence of disturbance. The slide plane itself was generally shiny and smooth as shown in Figure 5.

Fig. 5. Downslope View of Trenched Slide Showing Exposed Slide Plane.

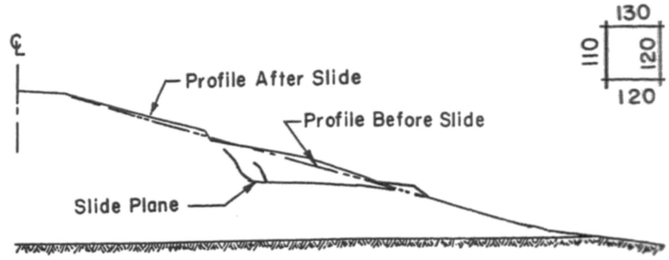


Fig. 6. Profile of Fully Developed Slide.

Based on discriptions by Skempton (1964), Morgenstern and Tchalenko (1967), and Skempton and Petley (1967) the slide planes indicated the material to be well within the residual strength range. Exceptions to this were noted in the developing slides trenched and will be discussed subsequently.

In addition to the distinct slide plane in each fully developed slide, numerous slickensides were encountered. The directions of the slickensides were generally random with some parallel to the slide plane and some intersecting it at various angles. All were above the slide plane. The slickensides varied in length from 2 to 18 inches. Their surface was shiny but irregular indicating that sufficient movement had occurred to orient the soil particles parallel to the surface but not enough translatory movement had occurred to create a smooth plane.

The typical slide broke about 15 feet down slope from the riverside crown and appeared to exit near the levee toe. Excavation revealed that the failure planes typically exited well above the levee toe, and the slide material below the slide plane exit was a "tongue" of material sliding along the levee slope. A cross section of a typical slide is presented in Figure 6.

From the surface of the scarp face down to a depth of 3 to 5 feet, no slide planes were located. Other than this, the slide planes were encountered throughout the slides and were generally quite pronounced.

In summary, the slide plane was located in all the fully developed slides excavated. The entire length of slide plane was located in each of the slides except for short segments in a few of the slides. Generally, it was easy to locate the well-defined portions of the slide plane. The less-defined segments could then be determined from the location of the adjacent segments. The shape of the failure planes appeared to be influenced by the type of soil structure. The slide planes in cubic-structured soil tended to have more circular arc-type segments. The failure surface also tended to vary more in the vertical direction, as evidenced by the variation in elevation from one side of the trench to the other. The platey type structure resulted in slide planes that were more planar in nature. The planes tended to be horizontal and to "stair step" down the slope.

Partially Developed Slides

Three slides were trenched that were in the initial stage of development. Many characteristics observed in these developing slides were similar to those observed in fully developed slides. The material type encountered was highly plastic clay (CH). Surface cracks were observed in the slide areas and the soil profile and structure were similar. The primary differences were in characteristics of the slide plane.

Prior to trenching one of the slides the only indication of movement was a scarp approximately 6 to 12 inches in height. A profile of this slide is shown in Figure 7. Downslope from the scarp there were no apparant deformations of the slope. After trenching and locating portions of the slide plane it was possible to identify a slight deformation in the slope at the toe of the slide. A fully developed slide plane was not located at this site. There were numerous slickensides in random directions throughout the slope that varied in length from a few inches to a foot. The slickensides connected to form continuous planes in some locations (see Figure 7) but the irregular surface of the slickensides indicated that the translatory movement was not sufficient to create a smooth, even failure plane.

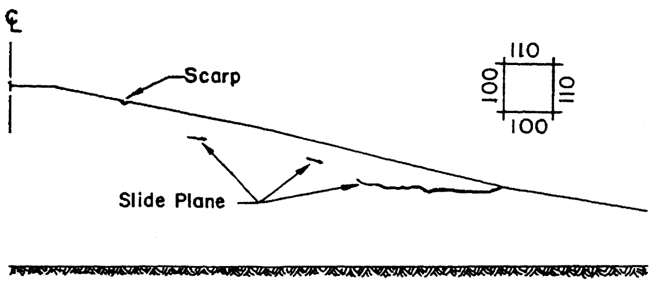


Fig. 7. Profile of Partially Developed Slide.

At the other two slides the only evidence of movement before trenching was a small "roll" approximately 6 inches high and 18 inches wide at the toe of the slide. Trenching revealed that fully developed slide planes had formed from the toe of the slide back into the levee slope for a distance of 30 to 35 feet. These fully developed portions of the slide plane appeared smooth and shiny, indicating the clay particles within the surface had attained their maximum degree of orientation and that the shearing resistance along the surface had decreased to near the residual. At one site the slide plane ended abruptly. Beyond this point several discontinuous slickensides 12 to 18 inches in length extended back into the slope. At the other site the slide plane appeared more irregular over the last 3 or 4 feet before it terminated in the slope. It appeared that movement experienced along the slide plane diminished from the toe to where it ended within the slope.

The significance of these observations is that the slide plane was developed to varing degrees within the same slope. It can be concluded that the slope is subjected to nonuniform stresses and local overstressing occurs. The result is concentrations of discontinuous slickensides that begin to connect at the toe of the slide and eventually form a continuous slide plane.

STABILITY ANALYSES

Undrained stability analyses were performed for a 1V on 3.5H slope assuming a homogeneous embankment with a cohesion of 0.35 tsf and a unit weight of 115 pcf. Factors of safety in excess of 3.0 were computed for failure planes similar to those located in trenched slides. Unconfined compression tests on compacted material from trenched slides resulted in shearing resistances twice that assumed in these analyses, indicating adequate factors of safety at the end of construction. Unconsolidated-undrained triaxial tests on undisturbed samples recovered from slides indicated strengths slightly higher than 0.35 tsf. Consequently, the undrained shear strength predicted by conventional laboratory tests on undistured samples are not applicable in analyzing the stability of the levee slopes.

Effective stress stability analyses were performed for two typical trenched slides to estimate the shear strength of the soil at failure and to determine the sensitivity of the slope to fluctuations in the piezometric surface. All analyses were made using the computer program SSTAB1, authored by Stephen G. Wright (1974). The program analyzes noncircular, piecewise linear slip surfaces by a limit equilibrium method of slices which satisfies all conditions of static equilibrium for each slice.

The sections analyzed were slope configurations existing before sliding occurred. The slip surfaces for which safety factors were computed were determined from data collected during trenching operations shortly after failure. Three unknown parameters defining the shearing resistance in terms of the Mohr-Coulomb criterion were assumed (or varied) to estimate conditions at failure. Since an infinite number of combinations of these parameters, cohesion (c'), friction angle (ϕ'), and pore pressure (u), will result in a factor of safety of unity, a constant value of $\phi' = 7.0°$ was assumed and the cohesion was varied until a safety factor of unity was calculated for the saturated to shell condition. The cohesion determined at the two slides analyzed were 0.044 tsf and 0.037 tsf.

The sensitivity of the factor of safety to fluctuations of the piezometric surface was investigated at both stations by lowering the phreatic surface 2 feet and 4 feet below the slope surface. In these analyses the parameters C' and ϕ' were assumed to be those corresponding to a factor of safety of unity for the saturated to shell condition. At both stations, lowering the phreatic surface 4 feet below the slope surface resulted in a factor of safety approximately equal to or greater than 1.25 which frequently used in the design of slopes for long-term stability.

The effective shear strengths at failure estimated in these analyses are much lower than the average peak strengths predicted by drained tests on undisturbed samples recovered from trenched slides. Skempton (1967) noted that the strength measured in these tests refers to the average peak strength which is certainly lower than the strength of the individual blocks and somewhat higher than the strength along existing fissures and shears. The shear strengths corresponding to a factor of safety of unity for the worst possible piezometric conditions are very near the residual values predicted by residual tests. Although it is difficult to make a reliable estimate of the average shear strength at failure, it is evident that a considerable reduction in shear strength occurred between construction and failure.

CONCLUSIONS

Sills (1981) reported that it was evident from results of shear strength tests and slope stability analyses that the peak strength is not applicable in analyzing the long-term stability of the embankment slopes. Observations from the trenched slides indicate the shear strength at failure is at or near the residual strength. Correlations presented in the literature indicate that the residual strength decreases with increases in clay content and plasticity index. The high clay content and high plasticity of the material recovered from slough slides indicate low values of residual strength which were confirmed through consolidated-drained repeated direct shear tests. Results of these tests indicate that the residual strength is less than half the peak strength.

The reduction in strength in the levee embankment results from weathering effects and strains induced by seasonal shrinking and swelling. During dry periods, shrinkage cracks open to a depth of 5 to 7 feet. These cracks expose the interior of the mass allowing desiccation to occur and fissures to form due to irregular shrinking. Subsequently, water from rainfall percolates through these cracks and fissures causing the material to swell and slake. Laboratory tests performed for this study indicate that this slaking results in a permanent increase in volume which must be accompanied by an increase in stress. This increase in volume and stress is augmented by the "self-swallowing" behavior of the soil (Allen and Brand 1966). During dry periods when cracks develop, soil material from the surface is knocked into the cracks by grazing cattle or surface runoff from rain. These stress increases are concentrated along discontinuities, and local overstressing occurs forming concentrations of slickenslides in zones experiencing the largest strains.

The slough slides appear to be triggered by heavy rainfall after an extended period of drying. The extensive network of cracks and fissures developed by years of weathering increases the mass permeability of the embankment. When these cracks fill with water, the exposed surfaces along the cracks and fissures soften, reducing the shear strength along these discontinuities. Piezometric data obtained from this study indicate that a perched water table forms above the intact clay zone located below the weathered zone. The increase in driving weight and concomitant softening of the exposed clay combined with the progressive loss of shear strength due to long-term seasonal shrinking-swelling effects result in a slough failure.

ACKNOWLEDGEMENTS

The original investigation reported herein was entitled "Long-Term Strength Reduction and Slough Slides in Mississippi River Levees." The study was conducted and the report prepared by Messrs. G. L. Sills and A. E. Templeton, Foundation and Materials Branch, Vicksburg District. Messrs. L. A. Cooley, Chief, Foundation and Materials Branch, and R. L. Fleming, Jr., Chief, Analytical Section, Foundation and Materials Branch, provided general supervision. This report includes a indepth study of this problem with some possible remedial measures. Testing was performed at the U. S. Army Engineer Waterwa Experiment Station (WES), the Vicksburg District Soils Testing Laboratory (LMVD), an the Soils Testing Laboratory at Texas A&M University.

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