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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 through 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.

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.

<table>
<thead>
<tr>
<th>Distance in Ft. from Levee</th>
<th>100</th>
<th>50</th>
<th>0</th>
</tr>
</thead>
<tbody>
<tr>
<td>--- 13 March 1970</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--- 28 July 1970</td>
<td></td>
<td></td>
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</tbody>
</table>

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 (ø) 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 ø values of 13.4 to 18 degrees with cohesions ranging from 0.09 to 0.24 tsf. Two consolidated-drained (CD) direct shear tests resulted in residual drained internal friction angle (øR) 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 (øR) 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 øR of 8.6 degrees with no cohesion. Average results from these shear tests are shown in Figure 3.

Cyclic 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.

![Figure 4. Results of Cyclic Shrink-Swell Test.](image)

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 phenomenon 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.
Based on descriptions 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 apparent 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 suffi­
cient to create a smooth, even failure plane.

Unconfined compression tests on compacted
within the same slope. It can be concluded
undrained samples recovered from slides
indicated strengths slightly higher than 0.35 tsf. Consequently, the undrained shear
strength predicted by conventional laboratory
tests on undisturbed samples are not applicable
in analyzing the stability of the levee
slopes.

Effective stress stability analyses were per­
formed for two typical trenched slides to es­
mate 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 φ' = 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 4 feet. 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 effective shear strengths at failure es­
imated 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
stresses corresponding to a factor of safety
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.

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 sliding 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
shear 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 varying 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 115pcf. 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.

![Fig. 7. Profile of Partially Developed Slide.](https://example.com/fig7.png)
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 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 the 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 Braud, Jr. 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 erode, 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 an indepth study of this problem with some possible remedial measures. Testing was performed at the U.S. Army Engineer Waterways Experiment Station (WES), the Vicksburg District Soils Testing Laboratory (LMVD), and the Soils Testing Laboratory at Texas A&M University.

REFERENCES


