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Department of Civil Engineering

CENTER FOR
GEOTECHNICAL PRACTICE AND RESEARCH

An Experimental Study of the Dynamic Behavior of Slickensided Surfaces

by

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CHAPTER 1: INTRODUCTION

The objective of this study is to investigate, through laboratory strength tests and centrifuge model tests, the shearing resistance that can be mobilized on slickensided rupture surfaces in clay slopes during earthquakes. The following case illustrates the limitations of our understanding regarding the seismic shear behavior of slopes in clay that contain slickensided rupture surfaces:

A 300-acre residential development at Rancho Solano, near Fairfield, California required significant mass grading. The grading resulted in steepened slopes at numerous locations across the site, and potential stability problems. Two geotechnical firms were engaged to study the potential stability problems, one on behalf of the developer and one on behalf of the property owners association.

Both firms recommended that the slopes should be stabilized using cut-and-fill operations. However, the grading and stabilization plans proposed by the firms differed in cost by a factor of approximately ten – one plan would cost about $2 million, the other about $20 million. This difference in cost was due primarily to the shear strengths chosen by the firms for use in the seismic slope stability analyses.

The uncertainty regarding the seismic shear strength of the Rancho Solano soil is related to the presence of slickensided rupture surfaces in the landslides. Because the landslides at the site had occurred in clayey soils, it was reasonable to expect that a large percentage of the platy clay particles along the rupture surfaces had become aligned in the direction of shear, forming "slickensided" shear surfaces. These slickensided rupture surfaces are inherently weaker than the surrounding soil mass (Skempton, 1964). Figure 1-1 is a sketch that shows a cross section through a slope that contains a slickensided rupture surface.

During an earthquake, ground shaking can cause additional landslide movement. For existing landslides or repaired landslides that contain slickensided rupture surfaces, it is reasonable to expect that the movement will occur along the existing slickensided rupture surfaces, because they are weaker than the surrounding soil. The amount of movement that occurs is controlled by the dynamic resistance that can be mobilized along the existing slickensided rupture surfaces.

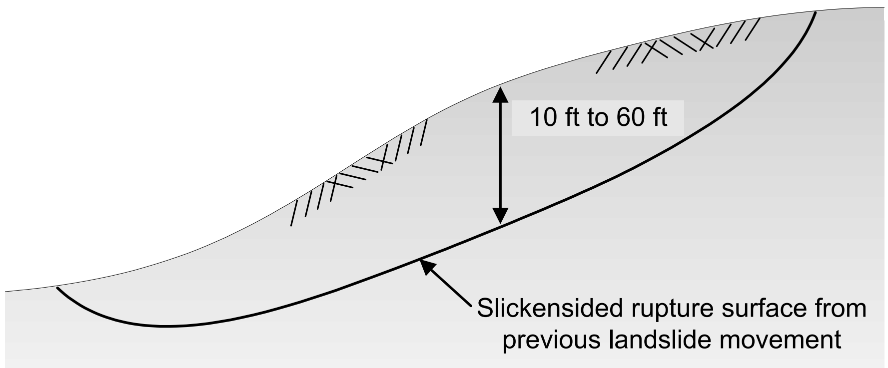


Figure 1-1. Cross section through slope containing slickensided rupture surface.

Little information is currently available concerning the dynamic shearing resistance that can be mobilized along existing slickensided rupture surfaces under seismic loading conditions. Given the present state of knowledge, it is not possible to say whether either of the Rancho Solano stabilization plans represented an optimum balance between safety and economy. Research is needed to provide a logical and supportable basis for evaluating the undrained cyclic shear strength that can be mobilized on pre-existing rupture surfaces in clay, so that projects like the one at Rancho Solano can be designed safely and economically.

Research Studies

The primary goal of the research project outlined in this report is to answer the following question:

> "What is the dynamic undrained shear resistance of a slickensided rupture surface that should be used in analyses of stability and deformation during earthquakes?"

To answer this question, a detailed investigation was undertaken, involving laboratory tests and analyses conducted at Virginia Polytechnic Institute and State University (Virginia Tech) and centrifuge tests at the University of California, Davis (UC Davis). At Virginia Tech, a method was developed for preparing slickensided rupture surfaces in the laboratory, and a series of ring shear tests, direct shear tests, and triaxial tests was conducted to study the static and dynamic shear resistance of slickensided rupture surfaces. At UC Davis, two dynamic centrifuge tests were performed to study the dynamic shear behavior of slopes that contain slickensided rupture surfaces. Newmark's method (Newmark, 1965) was used to perform seismic deformation analyses of the centrifuge model slopes. The results from the Newmark analyses were combined with the laboratory data from Virginia Tech and the centrifuge tests at UC Davis to develop design recommendations for analyzing the seismic stability of slickensided soil slopes.

The research studies described in this report were collaborative in nature, involving numerous contributions from researchers at Virginia Tech and UC Davis. Dr. Binod Tiwari, a post-doctoral researcher at Virginia Tech, performed triaxial tests on test specimens with pre-formed slickensided failure planes. Derek Martowska and Michael Wanger, Master's students at Virginia Tech, provided valuable assistance with the ring shear and direct shear testing programs. Raquel Miller, a Master's student at UC Davis, performed the initial feasibility studies for the centrifuge testing program.

CHAPTER 2: REVIEW OF PREVIOUS WORK

The objective of this chapter is to summarize the results of previous research on the shear behavior of slickensided soils. An overview of centrifuge model testing and seismic slope stability analysis methods is also provided, because of their relevance to the research program described in this report. The following categories of previous research are discussed:

- Slow Shearing of Slickensided Surfaces – Direct Shear
- Slow Shearing of Slickensided Surfaces – Triaxial
- Slow Shearing of Slickensided Surfaces – Ring Shear
- Fast Shearing of Slickensided Surfaces
- Cyclic Testing of Slickensided Surfaces
- Centrifuge Model Testing
- Seismic Slope Stability Analyses

Slow Shearing of Slickensided Surfaces – Direct Shear

Skempton (1964):

In his Rankine lecture, Skempton (1964) examined the behavior of stiff clays that are sheared slowly to large displacements. Shearing tests were conducted using a traditional Casagrande-type direct shear box, in which a thin, square soil specimen (2.4″ x 2.4″ x 1″) was subjected to monotonic, displacement-controlled shearing under a constant normal force. Failure occurred by rupture of the soil specimen at mid height, at the interface between the upper and lower shear boxes. The typical shearing behavior observed for the stiff clays in the Casagrande-type direct shear box is shown in Figure 2-1.

As the overconsolidated clay was sheared past its peak value of shear strength, the clay exhibited a “strain-softening” phenomenon, in which the ability of the clay to mobilize shearing resistance decreased due to softening and remolding of the clay on the failure plane. Additional shearing caused the platy clay particles located along the failure plane to orient themselves in the same direction, thereby decreasing the shear strength to its minimum value. This minimum mobilized shear strength is called the residual strength. Because shearing occurs slowly, shear-induced pore pressures have time to dissipate, and this residual strength can be characterized as a drained residual strength. As shown in Figure 2-1, Skempton (1964) demonstrated that drained residual strengths are typically much lower than drained peak strengths for clayey soils, and that consequently they can have a detrimental effect on the long-term stability of clay slopes.

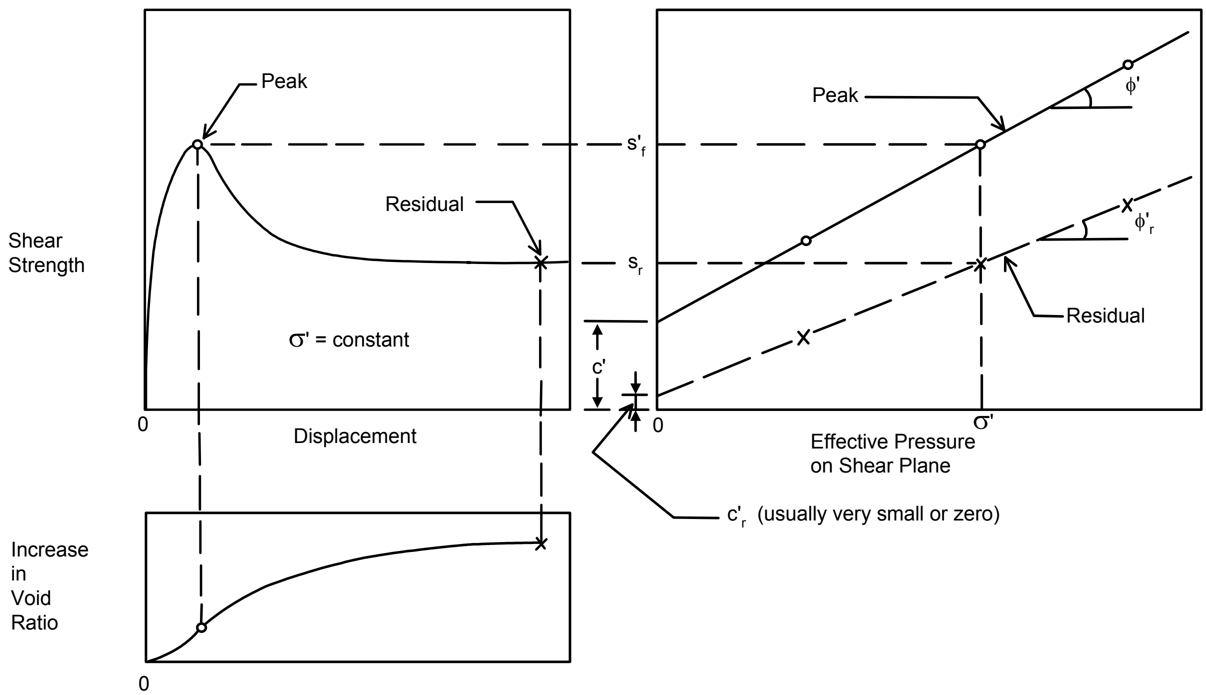


Figure 2-1. Shear characteristics of overconsolidated clay (Skempton, 1964).

Skempton (1964) found that once the drained residual strength has been reached, additional shearing will not change its value. As discussed above, this is due to the fact that the clay particles along the shearing plane become oriented in the direction of shear that corresponds to the lowest value of shear strength. Skempton (1964) refers to zones with shear-induced clay particle orientation as “slickensided”, and noted that slickensided features are often observed along the sliding plane in field landslides. In the field, these slickensided features often appear smooth and polished, with a lustrous sheen that is similar in appearance to the surface of a new bar of soap.

Direct shearbox tests indicate that once the peak strength has been reached, additional displacements on the order of 1 to 2 inches are enough to form slickensides and achieve the residual strength condition. Because the shear resistance of slickensided surfaces is smaller than that of the clay adjacent to the slickensided surface, shear displacements become localized on the slickensided surface once it forms. After the slickensided surface has formed, deformations involve one solid body sliding over another, along a well-defined interface between them.

Frequently, the amount of shear displacement that is necessary to reach the residual strength is greater than the maximum shear displacement that can be applied in a Casagrande-type direct shear box. Skempton (1964) suggested the use of "reversal" direct shear tests to address this issue. In a reversal direct shear test, once the maximum shearing displacement has been reached, the shear box is pushed back to its original position and sheared again. This process is repeated until the strength of the clay has dropped to a steady value, which is taken to be the residual strength. Typically, a small peak stress is observed after each reversal, as shown by the dotted line in Figure 2-2.

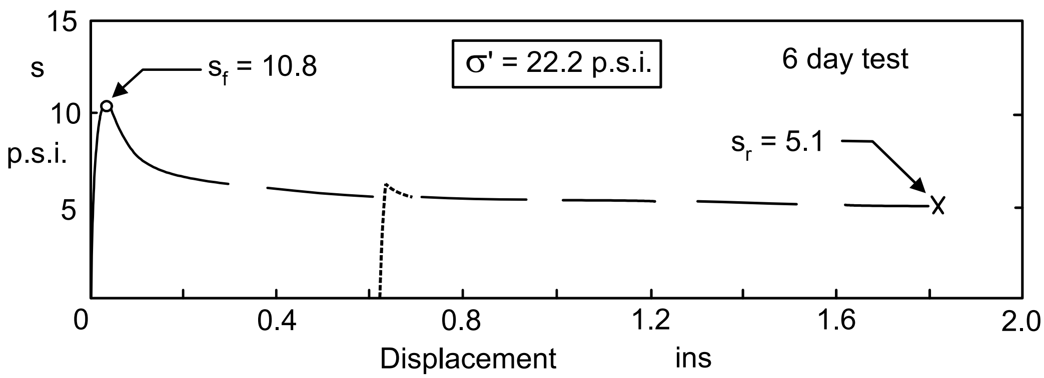


Figure 2-2. Effect of reversal in direct shear tests (Skempton, 1964).

Skempton (1964) found that if three different specimens of the same clay are tested at increasing effective normal stresses in the direct shear box, increasing values of peak and residual stress will be measured for each effective normal stress. As shown in Figure 2-1, the stress points that correspond to the peak and residual stresses can be plotted to form Mohr-Coulomb failure envelopes corresponding to the peak and residual strengths. Mohr-Coulomb envelopes from direct shear tests on four different clays show that the residual friction angle is always less than the peak friction angle ($\phi'_r < \phi'$) and that the residual cohesion (c'_r) is usually very small, and can be considered to be zero for most clays.

Skempton (1964) also stated that the residual strength of a clay under any given effective pressure is the same whether the clay is normally consolidated or overconsolidated, as shown in Figure 2-3. As a result, the residual strength is independent of stress history, and can be considered to be a fundamental property for a given clay soil. This statement is supported by the fact that residual strengths seem to correlate well with clay fraction. This observation led Skempton (1964) to conclude that, for a given normal stress, the residual strength will depend primarily upon the amount and the nature of the clay minerals that are present.

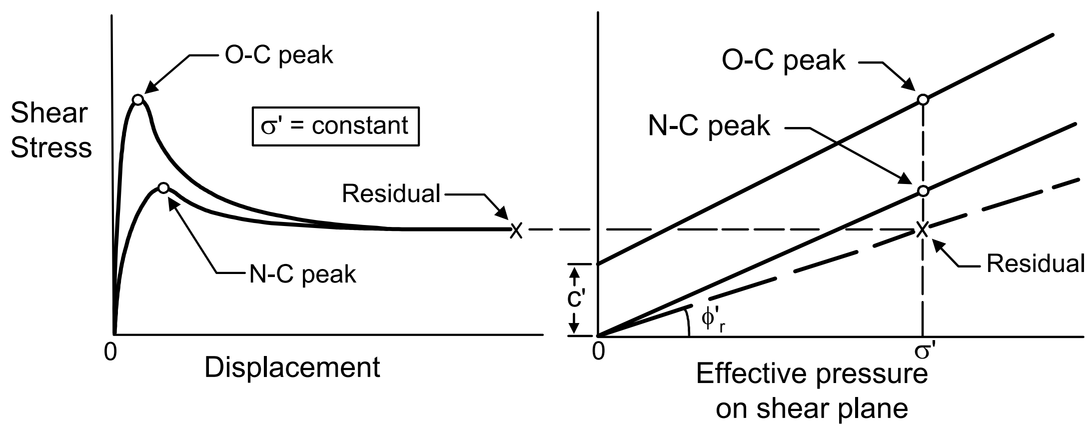


Figure 2-3. Simplified relation between normally and over-consolidated clay (Skempton, 1964).

Skempton's pioneering work in the area of drained residual shear strength successfully laid the foundation for much of the research that would be done in years to come. Although our understanding of drained residual shear strength has increased considerably since 1964, Skempton's research continues to define the state-of-the-art for determining drained residual shear strengths using conventional direct shear tests.

Skempton and Petley (1967):

Skempton and Petley (1967) performed a series of direct shear tests to measure the strength and stress-strain characteristics along existing slickensided discontinuities in stiff clays. In order to perform these tests, it was necessary to create direct shear test specimens that had slickensided discontinuities located at the likely plane of failure in the direct shear test. This was achieved by taking block samples of stiff clays that contained slickensided discontinuities, and trimming them to create direct shear test specimens that had slickensided discontinuities that coincided as nearly as possible with the separation plane in the shear box.

Typical test results for tests conducted on specimens trimmed from principal slip surfaces and joints are shown in Figure 2-4. Stress-displacement curves indicate that failure along a pre-existing slickensided discontinuity will start to occur once the residual strength along that discontinuity has been mobilized. As shown in Figure 2-4, a small peak strength is sometimes observed before the strength drops to the residual. This small peak could be caused by a number of contributing factors, such as: the slip surface not being planar, the slip surface containing some asperities, the clay particles not being fully oriented in the direction of shear along the discontinuity, or the development of a "bonding" effect after movement last occurred. Test results show that even when this small peak in strength occurs, the residual strength is still reached before completion of the first traverse of the shear box (as confirmed by subsequent reversals of the direct shear box). Consequently, it was concluded that strengths along principal slip surfaces and joints in the field are either at or very close to the residual strength.

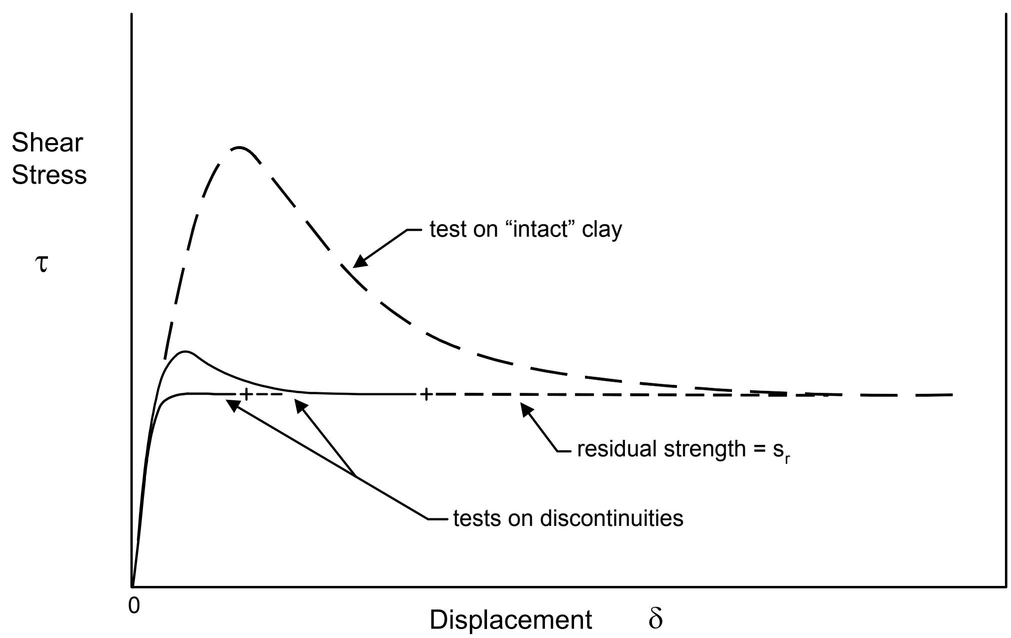


Figure 2-4. Stress-displacement curves from direct shear box tests on a discontinuity and on intact clay (after Skempton and Petley, 1967).

The direct shear tests conducted by Skempton and Petley (1967) also led to a number of significant additional observations regarding the behavior of existing slickensided discontinuities in stiff clays. From these tests, the authors discovered that:

- Direct shear box tests that measure the strength along existing slickensided discontinuities give the same value for residual shear strength as reversal shear box tests performed on intact specimens taken from a short distance away from the shear zone.

- The shear strength characteristics of freshly formed slickensides are the same as those of slickensides believed to be 10,000 years old. Therefore, it can be concluded that there is not a significant gain in shear strength over time for slickensided surfaces (the effect of thixotropy is minimal).

- The residual strength envelope is often nonlinear for clayey soils, especially at low normal pressures.

- The clay fraction content on a slickensided shearing surface is slightly greater than the surrounding soil. This indicates that there is some degree of clay-size enrichment, which may result from the physical breakdown of aggregations or silt-sized particles, or the migration of coarser grains from the shear zone during shear.

As was the case with Skempton's earlier work (Skempton, 1964), which established the standard for direct shear testing of intact overconsolidated soils, Skempton and Petley's (1967) direct shear testing approach has become the accepted method for measuring the drained residual strength along existing slickensided discontinuities.

<u>Kenney (1967):</u>

Kenney (1967) performed a series of reversal direct shear box tests to measure the residual strength of natural soils, pure clay minerals, and mineral mixtures. Both undisturbed and remolded specimens were tested following the approach outlined by Skempton (1964). Both the remolded and intact direct shear specimens were precut prior to shear. Use of this test approach was supported by the fact that tests conducted on precut specimens gave results that were more regular and reproducible than tests conducted on intact specimens.

Kenney (1967) found that the residual strengths of natural soils are primarily dependent on their mineral composition – both the quantity and type of clay minerals that are present. He also observed that, to a lesser extent, the residual strengths of natural soils are dependent on the magnitude of the normal effective stress and the overall system chemistry. Changes in pore water chemistry can affect the overall system chemistry, and were shown to have a direct effect on the measured residual strength. Kenney concluded that residual strengths do not correlate well to plasticity or grain size.

<u>Skempton (1985):</u>

By the mid-1980's, researchers had accumulated more than twenty years of experience measuring residual strengths in the laboratory. In 1985, Skempton published a paper that summarized what had been learned over the course of those twenty years about the laboratory measurement of residual strength, and the applicability of laboratory measurements of residual strength to active landslides in the field.

Skempton (1985) observed that reactivated landslides often move at varying rates of displacement that do not correspond to the usual laboratory testing rates that are used for the measurement of residual strength. However, he also observed that for typical variations in field displacement rate, the actual residual strength will be unlikely to vary by more than $\pm$5% from the value of residual strength that is measured in the lab. Since the fluctuation in actual residual strength is so small, a direct comparison between laboratory and back analysis strengths can be made in order to check the accuracy of the measured laboratory residual strengths.

Skempton (1985) stated that direct shear tests conducted on specimens that contain fully-developed landslide slip surfaces should be performed using the approach outlined by Skempton and Petley (1967). Test results show that if the slip surface is located exactly at the shear plane of the direct shear box, and if the specimen is arranged such that shearing follows the natural direction of shearing that was present in the field, then the residual strength will be recovered at virtually zero displacement. This measured value of residual strength agrees well with values derived from back analysis of reactivated landslides. Therefore, it can be concluded that landslide slip surface tests conducted in the direct shear box give accurate measurements for the value of residual strength in the field.

Skempton (1985) also stated that it is possible to measure the field residual strength by performing multiple reversal shear box tests on cut-plane samples trimmed from intact clay specimens. However, since the reversal direct shear test does not apply continual shearing to large displacements without reversal in the direction of shear, it is difficult to achieve complete particle orientation along the shearing plane. Consequently, multiple reversal shear box tests on cut-plane samples often give values of drained residual strength that are higher than the value of residual strength present in the field. Skempton (1985) observed that this effect is more pronounced in high clay-fraction soils, which give multiple reversal direct shear strengths that are higher than the strength that is measured in ring shear tests or calculated from back analysis of active landslides.

The displacements necessary to cause a drop in strength to the residual are usually far greater than those corresponding to the development of peak strength or fully softened strength in overconsolidated clays. Consequently, in the field, residual strengths are generally not relevant to first-time slides or other stability problems in previously unsheared clays and clay fills. The clay strength will be at or close to the residual value on slip surfaces in old landslides, in bedding shears and folded strata, in sheared joints or faults, and after an embankment failure. When pre-existing shear surfaces exist in the field, the residual strength should be used for engineering design.

<u>Slow Shearing of Slickensided Surfaces – Triaxial</u>

<u>Skempton (1964):</u>

Skempton (1964) performed drained triaxial tests on specimens from the Walton's Wood landslide shear zone. These specimens were prepared such that the landslide slip plane was inclined at 50º to the horizontal in the triaxial test specimen. Failure of the triaxial test specimen took place along the pre-existing slip surface. The measured strength on the slip plane corresponded closely to the residual strengths that were obtained by subjecting undisturbed clay specimens to large displacements in direct shear.

Skempton and Petley (1967):

Skempton and Petley (1967) performed triaxial tests on specimens that contained slickensided discontinuities, reporting measured residual strengths that agreed well with their direct shear testing results. These triaxial tests were carried out on specimens cut from landslide slip surfaces, with the landslide slip plane inclined at 50º to the horizontal in the triaxial test specimen. In nearly all of the cases tested, the residual strength was obtained before the displacement limit of the triaxial test had been reached.

Chandler (1966):

Chandler (1966) refined the approach used for testing slickensided discontinuities in the triaxial device by developing methods for correcting for the effects of membrane restraint and change in specimen area during shear. The effect of membrane restraint was determined by testing plasticene samples that contained a greased failure plane oriented at 55º from the horizontal. The effect of membrane restraint was found to vary with axial displacement and cell pressure. The recommended correction to deviator stress ranged from 1 psi to 15 psi for rubber membranes 0.008 inches thick. The recommended correction for the change in specimen area was based on the idealized change in contact area that occurs when two halves of a cylinder are displaced by each other along an inclined failure plane. Using the recommended correction, a triaxial specimen at 10% axial strain would experience a 20% decrease in contact area along a failure plane oriented 55º from the horizontal.

Chandler (1966) also recommended the use of ball bearings between the loading ram and the top cap, to give the top cap the freedom to move laterally during shear without tilting. This helped to maintain an even pressure along the failure plane, and reduced piston friction at the bushing.

Using the recommended membrane correction, area correction, and free platen test approach, Chandler (1966) reported residual friction angles for Keuper Marl that agreed well with the relationship between clay content and residual friction angle proposed by Skempton (1964). For the three specimens tested, the measured residual strength was reached at 2% to 4% axial strain.

Slow Shearing of Slickensided Surfaces – Ring Shear

Hvorslev (1939), Haefeli (1951), and others:

A number of early researchers in geotechnical engineering recognized the value of torsional shearing devices for their ability to measure the minimum value of shearing resistance in clayey soils that are sheared to very large displacements (Hvorslev, 1939; Haefeli, 1951; and others, as summarized by Bishop et al., 1971). As noted by these researchers, the primary advantage of the ring shear device over traditional direct shear and triaxial test equipment is that it allows for continual shearing to large displacements without reversal in the direction of shear. Because large displacements are often needed to achieve clay particle orientation along a shearing plane, torsional shearing devices are ideally suited for the measurement of residual strength. Unfortunately, the importance of residual strength and its effects on slope stability was not widely understood by the geotechnical engineering profession until the mid 1960's, so the practical importance of the pioneering work done by Hvorslev, Haefeli, and other early researchers was not appreciated until many years later.

Bishop et al. (1971):

The increased awareness of the importance of the post-peak shearing behavior in clayey soils brought about as a result of Skempton's Rankine lecture (Skempton, 1964) led to development of a torsional ring shear device by researchers at Imperial College and the Norwegian Geotechnical Institute (Bishop et al., 1971). This ring shear device, hereafter referred to as the NGI-type ring shear, is still widely used today for measuring drained residual strengths. In the NGI-type ring shear, an annular, ring-shaped soil specimen is subjected to torsional, displacement-controlled shearing under a constant normal force. Failure occurs by rupture of the soil specimen at mid height, at the interface between the upper and lower confining rings. Continued shearing results in clay particle orientation along the failure plane, and development of slickensides along which the residual strength is measured.

Since the width of the specimen is small compared to the diameter, uncertainties arising from an assumed non-uniform stress distribution across the shearing plane are reduced to an acceptable level. However, as noted by the authors, errors in stress measurement can arise as a result of friction at the contact between the rotating rings. As a result, accurate shear stresses can only be measured by leaving a gap between the confining rings. Unfortunately, as shearing progresses, soil particles extrude through this gap, resulting in some change in particle size at the shearing plane. This extrusion also makes calculation of vertical strains during shear subject to inaccuracy, because it is not possible to measure the volume of soil particles that are extruded through the gap.

With the NGI-type ring shear device, tests can be performed on remolded or undisturbed test specimens. "Multistage tests" can be performed by shearing the specimen to its residual state, changing the normal stress, and then shearing the same specimen to its residual state again. Using this approach with three or more normal stresses allows for the construction of a failure envelope with only one specimen. This reduces the amount of time necessary to develop a residual strength envelope.

Bishop et al. (1971) ran a series of tests using the NGI-type ring shear device on blue London Clay, brown London Clay, Weald Clay, Studenterlunden Clay, and remolded Cucaracha Shale. Comparison of their data with the data generated by other researchers testing the same clays (Agarwal, 1967; Garga, 1970; Hermann and Wolfskill, 1966; Kenney, 1967; La Gatta, 1970; Norwegian Geotechnical Institute, 1968; Petley, 1966; and Skempton and Petley, 1967) led to a number of important conclusions:

- The design of the NGI-type ring shear reduces mechanical friction and other types of "machine errors". A series of tests conducted on Blue London clay using the NGI-type ring shear agree well with a series of independent ring shear tests conducted by La Gatta (1970) using a "smear"-type ring shear device. Therefore, tests conducted using the NGI-type ring shear device give accurate measurements of the residual strength.

- Measurements of the ultimate residual friction angle are unaffected by the initial structure of the soil. Therefore, it is reasonable to use remolded specimens for measurement of drained residual shear strengths. This conclusion was also supported by La Gatta's (1970) ring shear tests; consequently, it has become common practice in the United States to use remolded specimens when measuring residual strengths (ASTM D 6467-99).

- In general, multiple reversal direct shear box tests give results which, in the case of clays, differ substantially from the true residual strength. For blue London clay in particular, NGI-type ring shear tests give much lower values of residual strength than those measured using multiple reversal direct shear box tests. The authors suggested that this is due to the inability of the direct shear box test to simulate the field condition of large relative displacements uninterrupted by changes in direction.

- 'Troughs' in direct shear stress-displacement curves may agree with residual strengths measured in the NGI-type ring shear device. Figure 2-5 shows the results from drained ring shear and multiple reversal direct shear tests on blue London clay. Note that the 'troughs' in the direct shear stress-displacement curves on the third and fourth travels agreed fairly closely with the residual strength measured in the NGI-type ring shear device.

- NGI-type ring shear tests give significantly lower values of residual strength than cut-plane triaxial tests. The two cut-plane triaxial tests on Ashford Common Shaft material (blue London Clay) suggest that even artificial polishing of slip surfaces by a spatula or glass plate does not establish maximum particle orientation.

<u>Bromhead (1979):</u>

By the mid-1970's, ring shear tests had become widely recognized as the best method for measuring the drained residual strength of clayey soils, due to their ability to apply large shear displacements without reversal in the direction of shear. However, ring shear tests conducted at that time using state-of-the-art ring shear equipment were too expensive and time-consuming to be widely used in engineering practice. In an attempt to address this issue, Bromhead (1979) developed a simple, robust, and relatively inexpensive ring shear device that was capable of running shearing tests more quickly than other ring shear devices on the market. As a result, the Bromhead ring shear has become widely used in engineering practice.

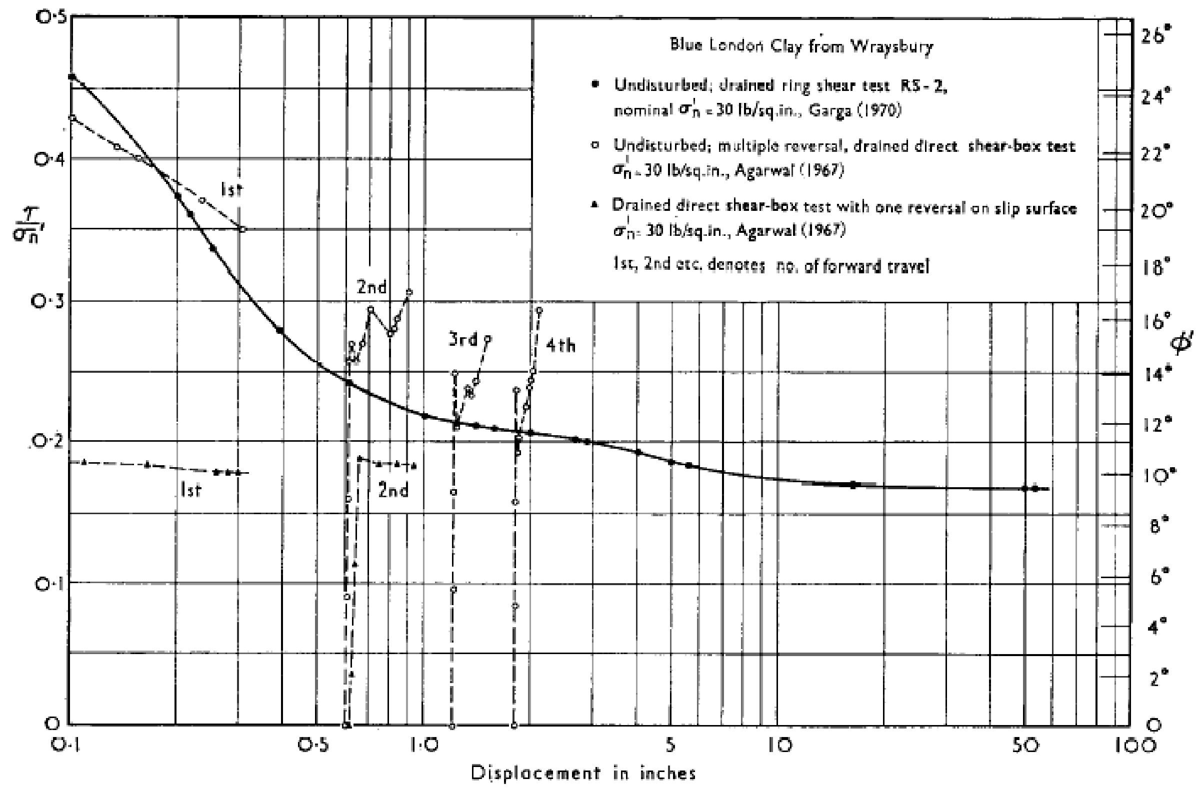


Figure 2-5. Drained ring shear and multiple reversal direct shear test results for blue London Clay (Bishop et al., 1971).

In the Bromhead ring shear, a thin annular soil specimen is subjected to torsional, displacement-controlled shearing under a constant normal stress. Failure occurs by rupture of the soil specimen along its upper surface, where a thin layer of clay particles that adhere to the roughened upper platen are displaced relative to clay particles below. Continued shearing results in clay particle orientation along the failure plane, and the development of slickensides along which the residual strength is measured. Because the failure plane is usually located at the top or close to the top of the specimen, the Bromhead ring shear is often categorized as a “smear-type” ring shear device.

Consolidation in the Bromhead ring shear device occurs rapidly, because the thin, annular specimen has a short drainage path length. Additionally, since the shearing plane is located close to the top of the specimen, the pore pressures generated during shear dissipate rapidly. Consequently, the shear rates that can be used in the Bromhead ring shear device are higher than those permissible in the NGI-type ring shear device. This allows for more rapid testing.

In the Bromhead ring shear, tests can be performed on remolded or undisturbed test specimens. As with the NGI-type ring shear device, "multistage tests" can be performed by shearing the specimen to its residual state, changing the normal stress, and then shearing the same specimen to its residual state again.

Bromhead (1979) reported that a series of Bromhead ring shear tests conducted on Gault Clay gave residual friction angles that agreed well with those measured in the NGI-type ring shear device. This data provides validation for the use of this type of ring shear device in engineering practice.

<u>Lupini et al. (1981):</u>

Lupini et al. (1981) performed an extensive review of previous studies done on the drained residual strength of cohesive soils, including various methodologies for measuring drained residual strength, and numerous correlations for drained residual strength with fundamental soil properties such as clay fraction and plasticity index. An extensive laboratory testing program was performed using the NGI-type ring shear, in which residual strengths were measured for sand and powdered mica mixtures, natural clay mixtures, and bentonite and sand mixtures. From this laboratory testing program, Lupini and his co-workers concluded that the mechanism of shearing at the residual condition could be classified as either "turbulent shear", "sliding shear", or "transitional shear". "Turbulent shear" occurs when soil particles pass by each other in a rolling, translatory fashion, with changing particle orientation. "Sliding shear" occurs when platy clay particles "slide" smoothly by each other during shear without reorientation. "Transitional shear" is the shearing state that occurs when both "turbulent shear" and "sliding shear" mechanisms occur simultaneously.

Turbulent shear is associated with soils that have low clay contents, and does not result in slickenside formation. Sliding shear occurs in clay-rich soils, and leads to the formation of slickensides. Transitional shear occurs at intermediate clay contents, and sometimes results in localized slickenside formation. All soils, when sheared to the residual state, exhibit one of these shearing mechanisms; however, it is those soils prone to the formation of slickensides that will be the focus of this report.

Lupini et al. (1981) also observed that the more highly plastic clays, which show preferred particle orientation and lower residual strengths, typically require displacements in excess of 4 inches (and sometimes up to 16 inches) to reach the residual state. Clays of lower plasticity require smaller displacements to reach the residual state.

<u>Anderson and Hammoud (1988), Anayi et al. (1989), Stark and Vettel (1992), and Stark and Eid (1993):</u>

As the Bromhead ring shear device became more popular, a number of researchers discovered that measured residual strengths were often dependent on details of the test procedure used (Anderson and Hammoud, 1988; Anayi et al., 1989; Stark and Vettel, 1992; and Stark and Eid, 1993). Of particular concern was the fact that multistage Bromhead ring shear tests did not agree well with single-stage Bromhead ring shear tests, because this was not consistent with the residual shearing behavior observed with the NGI-type ring shear device (Bishop et al., 1971). As a result, a number of constraints on the Bromhead ring shear test procedure and various modifications to the Bromhead ring shear device have been proposed in an attempt to improve the accuracy of the measured residual strengths.

Anderson and Hammoud (1988) were among the first to point out that different testing procedures could lead to different measured values of residual shear strength in the Bromhead ring shear device. To illustrate this, they performed Bromhead ring shear tests on identical specimens of two different clays at different normal stresses, using both single-stage and multistage testing techniques. They found that, for pottery clay, there is little difference in the results of single-stage and multistage tests. However, for kaolin, significantly lower values of residual stress were measured in multistage tests than in single stage tests. At higher stress levels, this difference was on the order of 20% to 25%. The authors theorize that this was due to the fact that the second and subsequent consolidation stages in a multistage test will tend to flatten out the "microkinks" along the shearing plane, with a consequent reduction in the stress necessary to mobilize residual shearing resistance. This difference is more pronounced with kaolin than with pottery clay because the platy particle orientation plays a more significant role with sliding shearing mechanisms than it does with transitional shearing mechanisms. Anderson and Hammoud (1988) concluded that the use of a multistage test technique is satisfactory for clays exhibiting "turbulent" or "transitional" shearing, but that a single stage test technique should be used for clays exhibiting a “sliding” shearing mode.

Anayi et al. (1989) reported problems testing Lias clay in the Bromhead ring shear apparatus. They found that, as shearing progressed, the forces in the proving rings became more and more imbalanced, with the force in one of the proving rings eventually falling to zero. The authors theorize that adhesion between the Lias clay and the porous bronze platen was not reliable, and some sort of remolding was taking place on part of the surface. This problem was overcome by modifying the specimen container and the loading platen to incorporate small vanes to transfer torque to the specimen, as shown in Figure 2-6. As a consequence of adding these vanes, the depth of the specimen container had to be increased and the shape of the torque arm had to be changed from rectangular in side elevation to tapered to allow for the increase in specimen thickness. With this modified specimen container, shear failure occurred at a plane at some depth within the specimen, instead of at the top of the specimen (as is typical for the unmodified Bromhead ring shear device). This helped to minimize extrusion as shearing progressed, but introduced additional side friction between the upper half of the specimen and the specimen container, decreasing the accuracy of the measured shear stress. Despite this limitation, the device modifications described above were successful in addressing the issues of imbalance between the proving ring forces, allowing for successful Bromhead ring shear testing of the Lias clay.

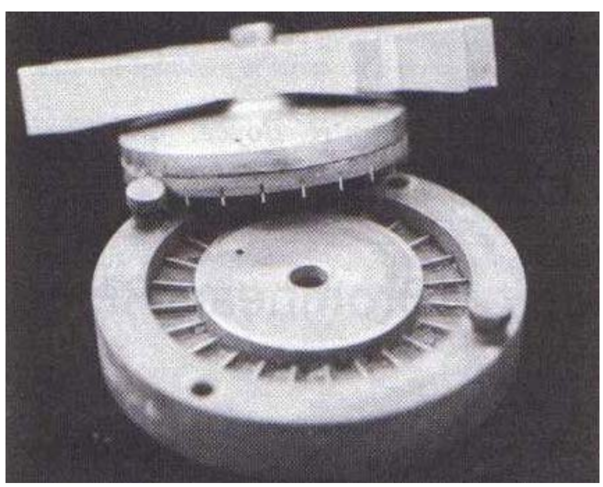


Figure 2-6. Modified specimen container for the Bromhead ring shear device (Anayi et al., 1989).

Stark and Vettel (1992) performed a series of ring shear tests to study the various device modifications and test procedures proposed for the Bromhead ring shear test (Anderson and Hammoud, 1988; Anayi et al., 1989; and Wykeham-Farrance, 1988). They observed that the main factor affecting the measured residual strength in the Bromhead ring shear apparatus is the magnitude of wall friction developed between the top porous stone and the walls of the specimen container. As shearing progresses, extrusion of the specimen causes the top porous stone to settle into the specimen container, thereby increasing the wall friction and decreasing the accuracy of the measured shear stress. This results in measured residual strength values that are higher than the actual residual strength values. Additionally, as the top porous stone settles into the specimen container, there is a greater chance that some of the extruded soil will become trapped between the top porous stone and the walls of the specimen container, introducing even larger errors into the measured shear stress. Test results show that the lowest residual strength is measured when the top porous stone remains at or near the surface of the specimen container (when little or no settlement occurs) and that this residual strength provides the best agreement with field case histories.

To address the issue of wall friction, Stark and Vettel (1992) concluded that the "flush" test procedure should be used for Bromhead ring shear testing, and that the use of shearing vanes, as proposed by Anayi et al. (1989), should be avoided. In the flush test procedure, after consolidation of the specimen has been completed, remolded soil is added to the specimen container, and reconsolidation of the specimen is performed to limit settlement of the top platen into the specimen container. Also, in order to reduce settlement of the top platen, specimens are not pre-sheared, and only one test is performed on each specimen. Since the flush testing procedure can be very time consuming, Stark and Vettel (1992) performed a sensitivity analysis, and found that measurements of residual strength are accurate using the Bromhead ring shear apparatus if the flush testing procedure limits top platen settlement to less than 0.03 inches.

Based on the results of the research conducted by Stark and Vettel (1992), Stark and Eid (1993) proposed a new specimen container for the Bromhead ring shear that allows multistage testing of overconsolidated specimens without excessive top platen settlement. Using this specimen container, remolded samples are overconsolidated and pre-cut prior to shearing, which decreases the amount of displacement necessary to reach the residual condition. This reduces the amount of top platen settlement for a given shearing stage, which makes it possible to run multistage tests without exceeding a top platen settlement of 0.03 inches. This reduction in top platen settlement minimizes the effect of wall friction, thereby satisfying the criteria for accurate measurements of shear stress that was established by Stark and Vettel (1992). Test results show that this modified specimen container allows for a much more rapid determination of the residual shear strength than the "flush" test procedure, and gives results that agree well with field case histories.

Stark and Eid (1994):

Stark and Eid (1994) conducted a series of drained ring shear tests using the Bromhead ring shear device, in order to examine the primary factors that influence the drained residual strength of cohesive soils. The modified specimen container proposed by Stark and Eid (1993) was used to minimize the effects of wall friction between the top platen and the side walls of the specimen container. Thirty-two different clays and clay shales were tested, and it was found that the drained residual strength is primarily influenced by the type of clay mineral and the quantity of clay-size particles present in the soil.

The results of these thirty-two ring shear tests revealed that the drained residual envelope is nonlinear, as shown in Figure 2-7. This agrees with the results of drained residual shearing tests conducted by numerous other researchers (Kenney, 1967; La Gatta, 1970; Bishop et al. 1971; Chowdhury and Bertoldi, 1977; Lupini et al., 1981; Bromhead and Curtis, 1983; Boyce, 1984; Hawkins and Privett, 1985; Gibo, 1985; Skempton, 1985; Anayi et al., 1988, 1989; Anderson and Hammoud, 1988; and Maksimovic, 1989). One method of addressing this failure envelope nonlinearity is to express the drained residual shear strength in terms of the secant friction angle. For a non linear failure envelope, the secant friction angle varies with the effective normal stress on the slip plane, as shown in Figure 2-7.

The ring shear test results showed that the drained residual strengths were influenced by the type of clay mineral and the quantity of clay-size particles that were present in the soil. Consequently, the authors concluded that drained residual strengths would correlate well with percent clay fraction and Atterberg limits. This conclusion is supported by the large number of correlations for drained residual strength that have been proposed by other researchers (Haefeli, 1951; Skempton, 1964; Kenney, 1967, 1977; Chandler, 1969; Voight, 1973; Kanji, 1974; Townsend and Gilbert, 1974; Kanji and Wolle, 1977; Lupini al., 1981; Skempton, 1985; Mesri and Cepeda-Diaz, 1986; Collotta et al., 1989; Müller-Vonmoos and Løken, 1989; Gibo et al., 1992; Mesri and Shahien, 2003; Wesley, 2003; and Tiwari and Marui 2005).

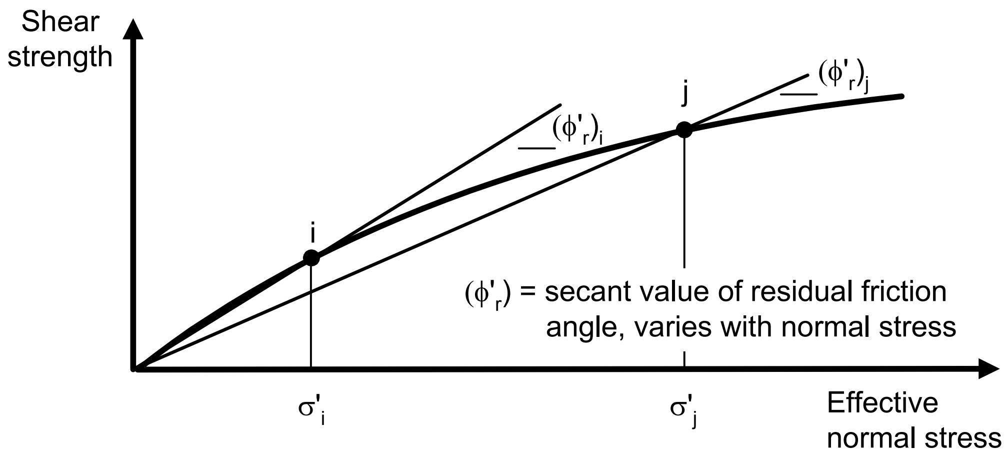


Figure 2-7. Shear strength envelope for a slickensided rupture surface.

Stark and Eid (1994) demonstrated that a good estimate for the drained residual shear strength of a clayey soil could be made based on its liquid limit and clay size fraction (percent by weight finer than 0.002 mm). This is a reasonable approach for estimating the drained residual strength of cohesive soils, because the liquid limit is an indicator of clay mineralogy and the clay size fraction is a measurement of the quantity of clay sized particles present in the soil. The correlation between residual friction angle, Liquid Limit, clay fraction, and effective normal pressure proposed by Stark and Eid (1994) is shown in Figure 2-8.

Tiwari et al. (2005), and others:

Tiwari et al. (2005) and numerous other researchers have compared residual strengths measured in the ring shear device with residual strengths back-calculated from analyses of failed landslides. Skempton (1985) and other researchers have compared residual strengths measured in the ring shear device with those measured in the direct shear device. The results from eight of these comparative ring shear studies are shown in Table 2-1.

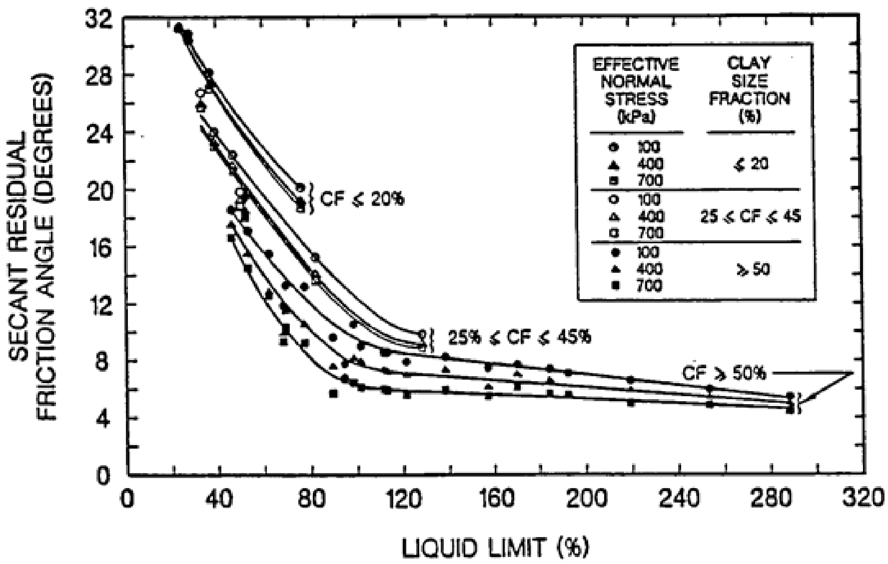


Figure 2-8. Correlation among residual friction angle, Liquid Limit, percent clay size, and effective normal pressure (from Stark and Eid, 1994).

Table 2-1 shows the range of results and conclusions regarding the nature of the relationship between the field residual strength and the residual strength measured in ring shear and direct shear. In most cases, the observed variations in the residual friction angle are less than 2°.

This review of available literature suggests the following conclusions:

- Ring shear tests measure residual strengths that agree quite closely with strengths back-calculated from analyses of landslides.
- Ring shear tests measure residual strengths that agree quite closely with strengths measured in direct shear tests on specimens cut from landslide slip surfaces.
- Ring shear tests measure residual strengths that are slightly lower than those measured in reversal direct shear tests.

Table 2-1: A Comparison of Drained Residual Strengths Measured Using the Ring Shear Device

Reference	Tests/Analyses Run	Ring Shear	Direct Shear	Notes
Bromhead (1979)	12 NGI-type RS 46 Bromhead RS	Bromhead ring shear agrees well with NGI-type ring shear.		
Boyce (1984)	48 Bromhead RS 6 reversal DS	Agrees well with reversal direct shear.	Agrees well with ring shear.	Seven soils tested.
Skempton (1985)	20 NGI-type RS 39 slip surface DS 13 back-analysis	Ring shear tests 1.5º lower than back-analysis	Slip surface tests agree well with back-analysis.	
Hawkins and Privett (1985)	6 Bromhead RS 18 reversal DS	Agrees well with reversal direct shear.	Agrees well with ring shear.	Results must be compared for the same effective stresses and subtle testing factors such as the effect of shear reversal, the effect of undulating failure plane, and the effect of specimen extrusion must all be understood and minimized.
Bromhead and Dixon (1986)	72 Bromhead RS 10 slip surface DS 20 back-analysis	Ring shear and back analysis data agree well.	Direct shear slip surface tests agree well with back analysis and ring shear tests.	Data contradicted Skempton's (1985) previous conclusion for London Clay.
Anayi et al. (1988)	22 Bromhead RS 20 reversal DS	Ring shear tests 1.5º lower than direct shear tests.	Direct shear tests 1.5º higher than ring shear tests.	Traditional Bromhead tests did not provide accurate results for Lias Clay. Bromhead ring shear modified to include vanes.
Stark and Eid (1992)	6 Bromhead RS 3 reversal DS 3 pre-cut reversal DS 1 back-analysis	Ring shear tests agreed with back-analysis.	Reversal direct shear tests yielded F that was 60% too high. Pre-cut reversal direct shear yielded F that was 10% too high.	Stark and Vettel (1992) test procedure used.
Tiwari et al. (2005)	187 NGI-type RS 6 back-analysis	Ring shear tests agreed with back-analysis.		Six different sites and six different soils tested.

Fast Shearing of Slickensided Surfaces

Skempton (1985):

Skempton (1985) performed a series of NGI-type ring shear tests on two slickensided clay soils, studying the effect of rate of loading on the measured strength. He found that tests on clays conducted at rates from 100 times slower to 100 times faster than the commonly used drained laboratory test rates of 0.01 mm/min gave residual strengths that increased by about 2.5% per log cycle increase in strain rate. This shows that variations in strength caused by loading rate effects are negligible within the usual range of slow laboratory tests (0.002 to 0.01 mm/min).

Skempton (1985) also conducted NGI-type ring shear tests on slickensided Kalabagh Dam clay to measure the effects of fast displacement on residual strength. Figure 2-9 shows the stress-displacement response measured for a Kalabagh Dam clay specimen.

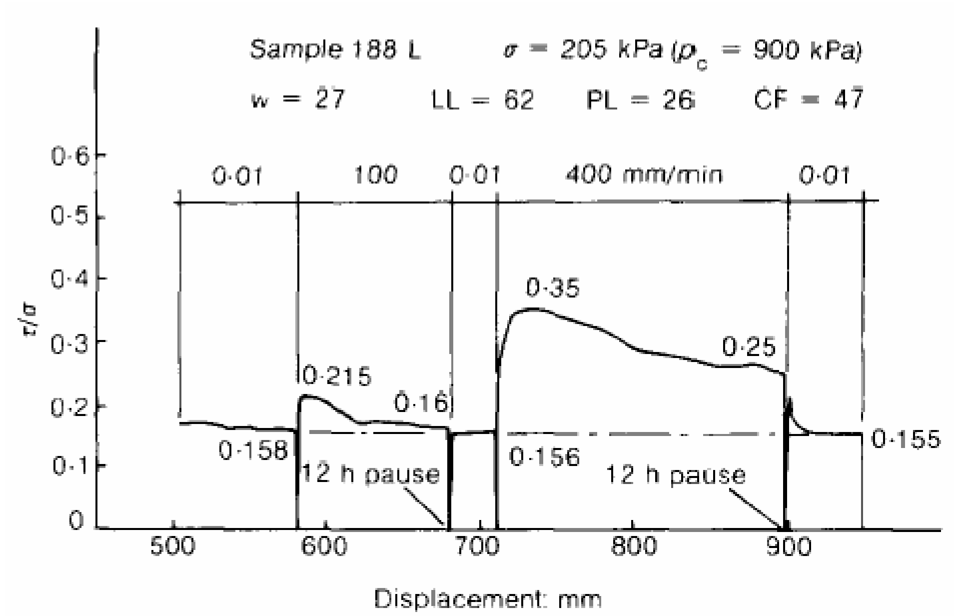


Figure 2-9. The stress-displacement response measured for a Kalabagh Dam clay specimen (Skempton, 1985).

During these tests, specimens were subjected to alternating slow and fast shearing stages, as follows:

- First, slow shearing was applied at a displacement rate of 0.01 mm/min to create a slickensided failure surface and measure the drained residual strength;

- Second, fast shearing was applied at 10, 100, 400, or 800 mm/min to measure the strength of the clay soil under rapid loading;

- Third, slow shearing was applied at 0.01 mm/min to measure the drained residual strength of the soil after a rapid loading event; and

- Fourth, additional fast and slow shearing were applied to measure the strength at a different loading rate.

As shown in Figure 2-9, rapid loading initially causes a significant increase in strength to a maximum value. As shearing continues, the shear resistance decreases to a steady minimum value. In clays and low clay fraction silts, the minimum value was higher than the slow residual strength. In clayey silts with clay fractions around 15-25%, this value was lower than the slow residual strength (in some cases, as low as one-half the slow residual value). A summary of the rapid loading strengths measured for Kalabagh Dam clay is provided in Figure 2-10.

Based on the data shown in Figures 2-9 and 2-10, Skempton (1985) states: "For clays the increase in strength becomes pronounced at rates exceeding 100 mm/min (Figure 2-10) when some qualitative change in behaviour occurs. This is probably associated with disturbance of the originally ordered structure, producing what may be termed 'turbulent' shear, in contrast with sliding shear when the particles are orientated parallel to the plane of displacement. It is possible, also, that negative pore pressures are generated and, as displacement continues, these are dissipated within the body of the sample thus leading to a decrease in strength. That some structural change has taken place in clays at ratios of 400 mm/min or more seems apparent from the fact that on reimposing the slow rate a peak is observed, the strength falling to the residual only after considerable further displacement (Figure 2-9), an effect not seen after shearing 100 mm/min or slower."

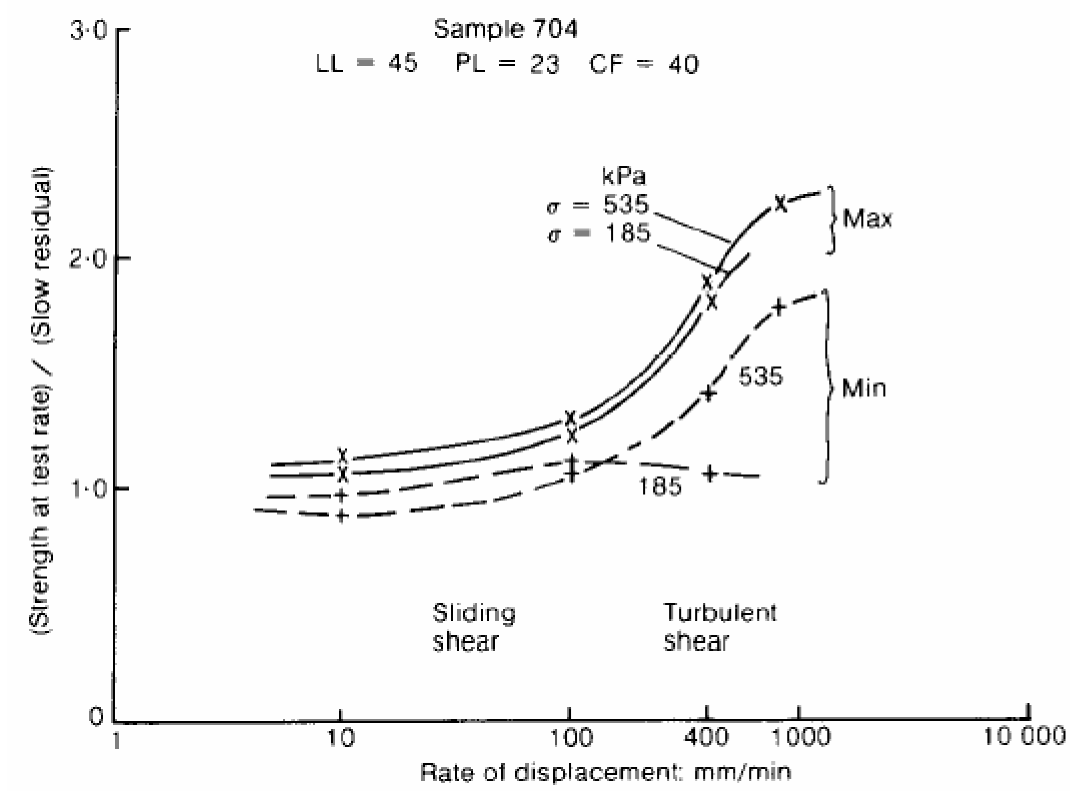


Figure 2-10. Rapid loading strengths measured for Kalabagh Dam clay (Skempton, 1985).

Lemos et al. (1985) and others:

Lemos et al. (1985) performed a series of fast ring shear tests on nine different soils using the test approach described by Skempton (1985). They observed different behavior for high clay fraction and low clay fraction soils. Figure 2-11 shows the shear response measured for high clay fraction soils, which are those prone to formation of slickensides and development of low residual strengths.

Figure 2-11 illustrates the shear strengths observed at different shear stages when pre-existing slickensided surfaces are rapidly sheared in a ring shear device. These different strengths were described by Lemos et al. (1985) as follows:

- Prior to fast shearing, the soil was sheared slowly to create the slickensided surface and to establish the drained residual strength. Point (a) in Figure 2-11 shows the slow shear behavior and the corresponding drained residual shear strength.

- Point (b) in Figure 2-11 marks the beginning of fast shearing. An increased threshold strength is observed when displacement on the shear surface recommences.

- As fast shearing is continued, an additional increase in strength is observed, as shown by Point (c) in Figure 2-11. This increased strength is called the fast maximum strength.

- As fast shearing continues to Point (d) in Figure 2-11, the strength drops from the fast maximum strength to a fast minimum strength.

- At Point (e) in Figure 2-11, fast shearing is stopped and slow shearing is resumed. A new slow peak strength is generally observed, which is higher than the drained residual strength shown at Point (a). This indicates that a structural change has taken place along the slickensided surface as a result of the rapid shearing, which supports the failure mechanism hypothesized by Skempton (1985).

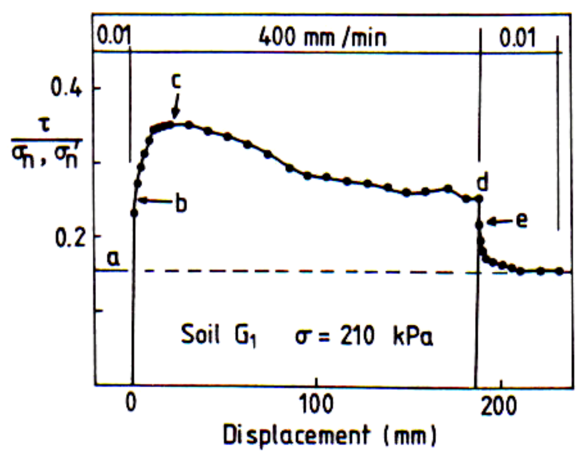


Figure 2-11. Typical results from rapid ring shear tests conducted along existing slickensided surfaces (from Lemos et al., 1985).

Additional laboratory testing performed by Tika et al. (1996), Vessely and Cornforth (1998), and Tika & Hutchinson (1999) using the NGI-type ring shear device agrees with the data reported by Skempton (1985) and Lemos et al. (1985). From this research, it appears that slickensided clays exhibit a significant increase in shear strength as strain rate is increased.

Cyclic Testing of Slickensided Surfaces

Yoshimine et al. (1999):

Yoshimine et al. (1999) conducted a series of cyclic ring shear tests along pre-existing shear surfaces in sixteen different soils from landslides. Shear surfaces were created in remolded, normally consolidated specimens by shearing the specimens to large displacements at a rate of 0.01 mm/min. The specimens were then subjected to gradually increasing shear rates up to 300 mm/min, to examine loading rate effects on monotonic shear strength. Three different types of stress-controlled cyclic loading were then applied to each of the specimens: (1) constant amplitude sinusoidal loading, shown in Figure 2-12, (2) increasing amplitude sinusoidal loading, shown in Figure 2-13, and (3) simulated earthquake loading, shown in Figure 2-14. For each of the cyclic loading phases, the applied cyclic stresses were imposed on top of an initial static shear stress that was equal to 70% of the static drained residual strength. Slow shearing (0.01 mm/min) was performed prior to each of the cyclic loading events to ensure that the shear surfaces that were tested were at their residual strength.

Fast monotonic test results showed that the residual strength generally increased with testing speed. Some of the soils with intermediate clay fractions (20% to 30%) exhibited smaller shear strengths at higher displacement rates. This behavior is consistent with the behavior observed by Skempton (1985), Lemos et al. (1985), Tika et al. (1996), and others.

The constant stress cyclic test results indicated that there is a threshold strength below which cyclic displacement does not occur. Above the threshold strength, constant deformation was observed for each cycle, as shown in Figure 2-12. No strain hardening or softening behavior was observed for any of the soils tested. The number of applied cycles did not appear to influence the cyclic behavior along pre-existing shear surfaces.

As shown in Figures 2-13 and 2-14, the increasing stress cyclic tests and earthquake loading tests show a direct relationship between the total applied shear stress (initial + cyclic) for each load pulse and the resulting displacement per pulse. The dynamic strength was defined as the stress level at which the stress-displacement curve became nearly flat. The measured dynamic strengths varied widely, with most soils exhibiting strengths that ranged from 20% to 100% higher than the slow residual strength. As the cyclic loading frequency was increased from 0.1 Hz to 1 Hz, most soils exhibited a 5% to 20% increase in dynamic strength.

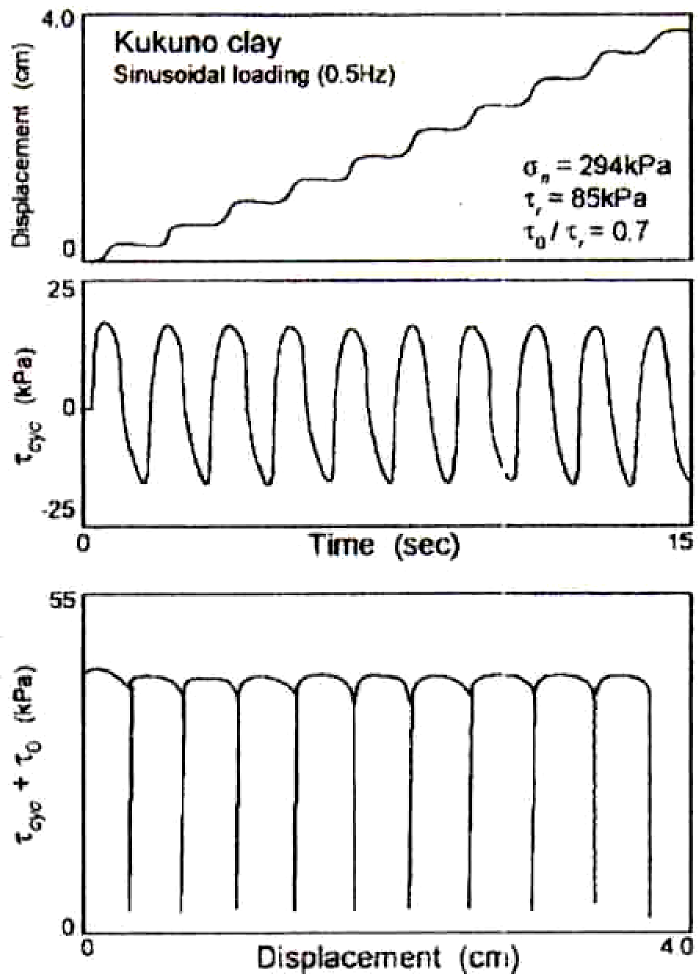


Figure 2-12. Constant amplitude sinusoidal loading of pre-sheared Kukuno clay (Yoshimine et al., 1999).

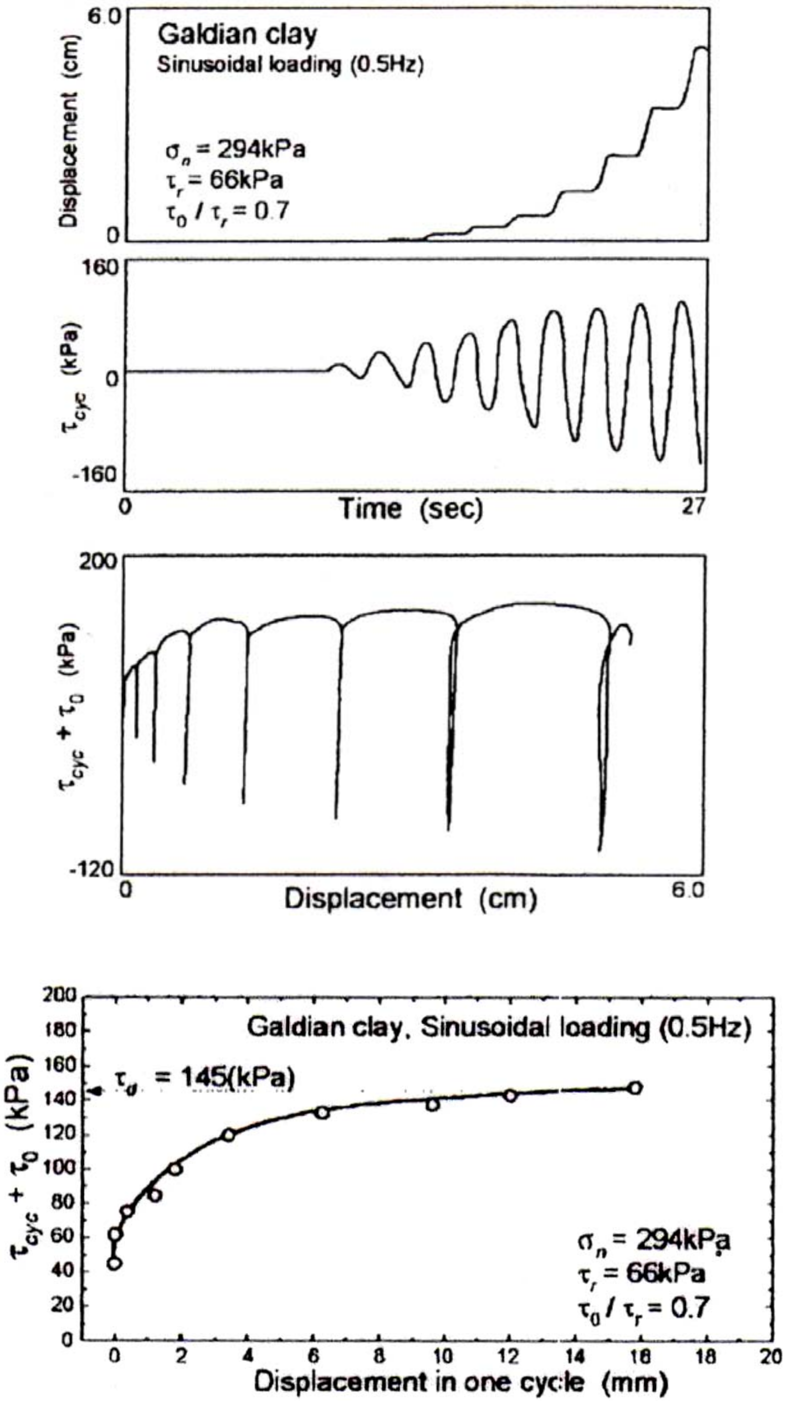


Figure 2-13. Increasing amplitude sinusoidal loading of pre-sheared Galdian clay (Yoshimine et al., 1999).

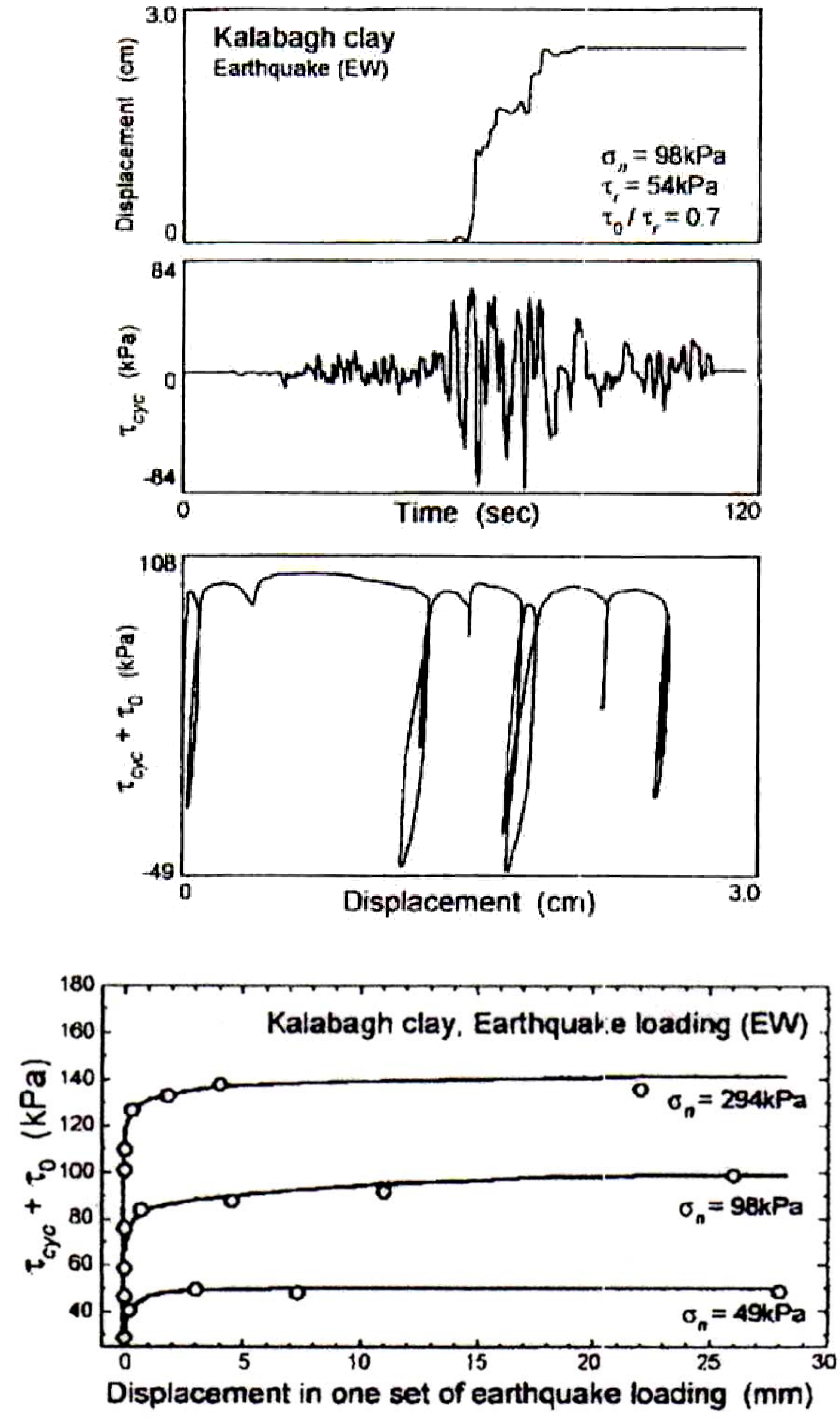


Figure 2-14. Simulated earthquake loading of pre-sheared Kalabagh clay (Yoshimine et al., 1999).

Of particular significance is the fact that all of the soils tested, including those that exhibited reduced shear strength during fast monotonic loading, showed an increase in strength over the drained residual condition during cyclic loading. These results provide justification for using dynamic strengths that are larger than the drained residual shear strength when performing seismic stability analyses of slickensided clay slopes. This represents a departure from the current state of practice, which is to use the drained residual shear strength as a "first-order approximation of the residual strength friction angle under undrained and rapid loading conditions" (Blake et al., 2002).

Centrifuge Model Testing

Kutter (1992):

Kutter (1992) explained the basic principles of dynamic centrifuge model testing, discussing the advantages and disadvantages inherent to this form of geotechnical laboratory testing. In order to conduct a dynamic centrifuge model test, a small scale model that represents a large geotechnical structure is "spun up" in the centrifuge. This "spin-up" process subjects the scale model to centrifugal accelerations that are significantly larger than the acceleration imposed by the earth's gravity. These centrifugal accelerations increase the self-weight of the soil, allowing the scale model to experience stresses that are the same in the model as they are in the field (the prototype). This stress "scaling effect" causes the scale model to behave much like the prototype structure it represents when subjected to dynamic loading.

Kutter (1992) described a series of centrifuge modeling laws. N is the size ratio between the prototype structure and the scale model. If the same soil is used in the model and the prototype, the "Density" relationship between the model and the prototype is 1/1. In order for the stresses in the model to be the same as the stresses in the prototype, the "Gravity" relationship between the model and the prototype must be N/1. From the scale factors for length, density, and gravity, the scaling relationship for other physical quantities such as mass, force, stress, strain, and time can be derived. The resulting scale factors for centrifuge model tests are given in Table 2-2.

Table 2-2: Scale Factors for Centrifuge Model Tests (after Kutter, 1992)

Quantity	Units	Model Dimension / Prototype Dimension
Length	L	1/N
Volume	L^3	$1/N^3$
Mass	M	$1/N^3$
Gravity	L/T^2	N
Force	ML/T^2	$1/N^2$
Stress	M/LT^2	1/1
Moduli	M/LT^2	1/1
Strength	M/LT^2	1/1
Acceleration	L/T^2	N
Time (dynamic)	T	1/N
Frequency	1/T	N
Time (diffusion)*	T	1/N or $1/N^2$

*Note: The diffusion time scale factor depends on whether the diffusion coefficient (e.g. coeffiecient of consolidation) is scaled. If the same soil is used in model and prototype, use $1/N^2$.

Centrifuge testing offers an advantage over traditional geotechnical laboratory strength tests like the ring shear, direct shear, and triaxial test because it can model the behavior of an entire geotechnical system instead of a single soil element. By modeling the behavior of an entire geotechnical structure, it is possible to capture soil-structure interaction behavior and failure mechanisms that cannot be measured in traditional laboratory “element” testing. Centrifuge testing is superior to other forms of scale model testing (such as shaking table tests), because the applied centrifugal g-field causes the stresses applied in the model to be the same as the stresses in the prototype.

Potential modeling problems regarding the effect of stress history can be avoided by constructing the model out of soil that has the same stress history as the prototype. Addressing the effect of loading rate is not so simple, because the assumption of rate-independent mechanical properties is embedded in the derivation of the scale factors (Kutter, 1992; Uzuoka and Furuta, 2001).

<u>Seismic Slope Stability Analyses</u>

Prior to 1965, the pseudo-static method was considered the state-of-the-art approach for performing seismic slope stability analyses (Seed and Martin, 1966). In engineering practice today, the pseudo-static method is still used as a screening procedure to evaluate the landslide hazard for slopes in earthquake prone areas (Seed, 1979; Duncan and Wright, 2005). Displacement analyses are recommended for those slopes that do not pass the pseudo-static screening procedure (Blake et al., 2002; Duncan and Wright, 2005). Because of its simplicity, Newmark's method (Newmark, 1965) is widely used to estimate earthquake-induced displacement of slopes.

Newmark (1965):

Newmark (1965) introduced a method for estimating earthquake-induced slope displacements based on the assumption that a sliding mass behaves as a rigid body with resistance mobilized along its sliding surface. Conceptually, Newmark's method is analogous to a block resting on an inclined plane – although the block is stable under static conditions, shaking causes the block to slide.

Figure 2-15 shows the approach used to calculate displacements with Newmark's method. First, a yield acceleration is calculated, which is the value of horizontal acceleration that would cause slope failure if it was applied at the center of the mass. The yield acceleration is then compared to the expected earthquake acceleration time history for the site. Earthquake accelerations in excess of the yield acceleration cause slope displacement. The magnitude of this displacement is calculated by double integration of the portion of the acceleration record that is larger than the yield acceleration.

Simplified calculation approaches based on Newmark's method have been proposed for use in engineering practice by various researchers (Newmark, 1965; Hynes-Griffin and Franklin, 1984; and others, as summarized in Cai and Bathurst, 1996 and Duncan and Wright, 2005). Other researchers have shown how Newmark's method can be applied to different slope failure mechanisms (Goodman and Seed, 1966; Chang et al., 1984; Ling and Leshchinsky, 1995; Michalowski and You, 1999; and Stamatopoulos et al., 2000). Modifications to Newmark's method have also been suggested to address the limitations associated with assuming rigid block response and rigid-plastic sliding behavior (Makdisi and Seed, 1978; Kutter, 1984; Kramer and Smith, 1997; Rathje and Bray, 1999; Razaghi et al., 1999; and Botero and Romo, 2001).

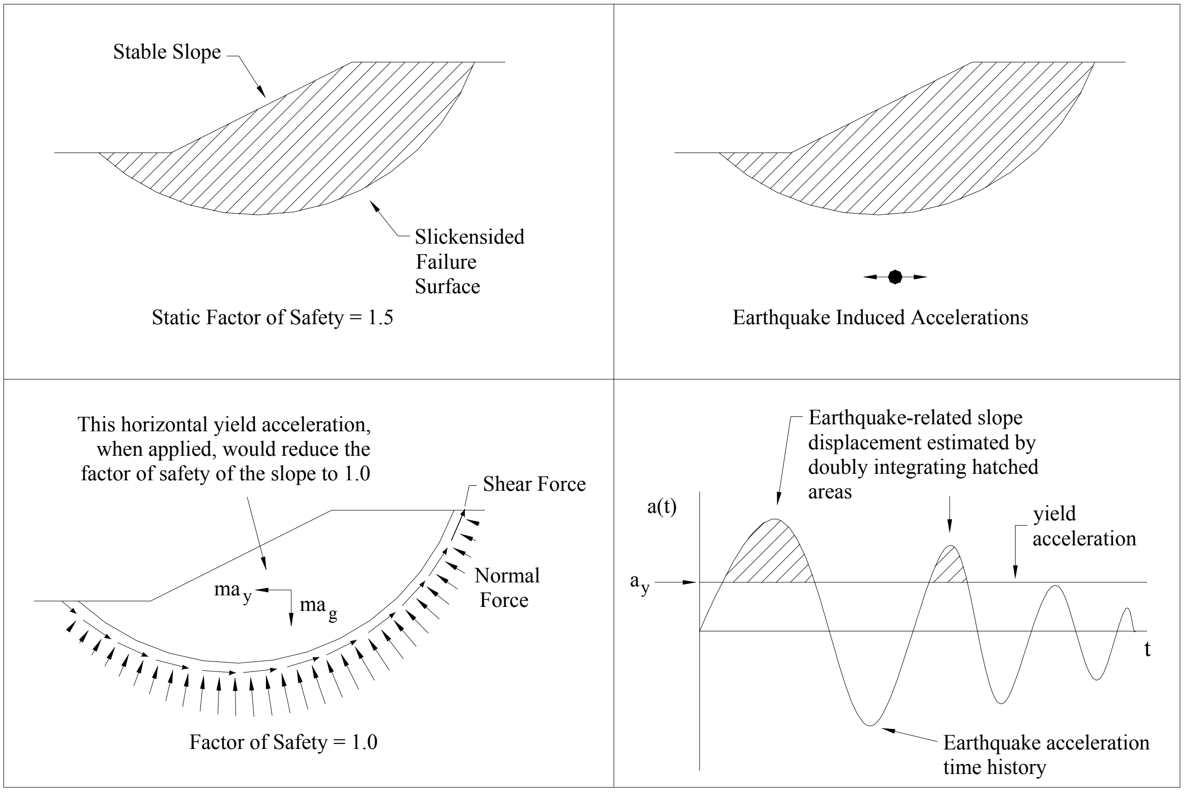


Figure 2-15. Newmark's method for calculating earthquake-induced slope displacements (Newmark, 1965).

<u>Blake et al. (2002):</u>

Blake et al. (2002) described the current state of practice for mitigating landslide hazards in California. With respect to dynamic displacement analyses for slopes containing slickensided surfaces, the following statements are made:

"The effect of strain rate on drained residual strengths was investigated by Skempton (1985) and Lemos et al. (1985). Their results suggest that the residual strengths of clay-rich materials (> 50% clay content, e.g., claystone, shale) are generally higher for rapid strain rates (> 100 mm/minute) than for ordinary strain rates. However, their testing also suggests that the residual strength for materials with intermediate clay contents (approximately 25%) can decrease with increasing strain rate. It is not clear from these papers whether the observed variations in strength from tests conducted at different strain rates are in fact resulting from pore pressure generation or true strain rate effects. Further research is needed on this topic. It is the judgment of the Committee that, based on the current state of knowledge, the residual strength friction angle from a drained test conducted at "normal" strain rates can be used as a first-order approximation of the residual strength friction angle under undrained and rapid loading conditions."

Pradel et al. (2005):

Pradel et al. (2005) document a case history of landslide movement during the Northridge earthquake. The unusually high quality site investigation data, shear strength data, and post-earthquake reconnaissance data at this site provided a unique opportunity for checking the accuracy of Newmark's method (Newmark, 1965) for slope displacement calculations.

The earthquake-induced landslide occurred in a weathered, previously sheared siltstone that was believed to be at its residual shear strength prior to the earthquake-induced slide movement. The sliding that occurred caused a break in a water main located at the head scarp of the slide, and measurement of the displacement between the broken pipe sections indicated that the slide had moved approximately 50 mm during the earthquake.

During a five-year period following the earthquake, additional rainfall-induced sliding occurred, resulting in litigation and a thorough analysis of the soil conditions at the site. Back-analysis of the rainfall-induced landslides gave residual shear strength values that agreed well with those measured in reversal direct shear tests.

As part of their case-history documentation, Pradel et al. (2005) performed a series of Newmark analyses using the measured and back-calculated residual strengths to estimate earthquake-induced slope displacements. These analyses were performed using drained residual strength parameters for the bedrock because, "Materials at residual strength are not expected to generate significant pore pressures during shear". Four input ground motions were used for the Newmark analyses, based on nearby recorded strong motion data.

The displacements calculated using Newmark's method ranged from approximately 20 mm to 90 mm, and were found to be highly sensitive to the position of the groundwater table. Using the best estimate of the groundwater table location at the time of the earthquake, the average predicted displacement is 46 mm, which agrees well with the 50 mm of displacement that was observed at the site.

Pradel et al. (2005) concluded that Newmark-type sliding block analyses can result in reasonable estimates of seismic displacements for landslides using site-specific geotechnical analyses. Because calculated seismic displacements are extremely sensitive to groundwater level and ground motion characteristics, uncertainties in those parameters (and in general, shear strength as well) should be considered when performing Newmark analyses to estimate seismic slope performance.

CHAPTER 3: CLAY PROPERTIES AND PREPARATION OF SOIL FOR TESTING

In order to study the dynamic undrained shear resistance of slickensided rupture surfaces, a laboratory testing program involving ring shear tests, direct shear tests, triaxial tests, and centrifuge tests was undertaken at Virginia Tech and UC Davis. Three different natural clay soils were studied during this laboratory testing program. Two of the soils were obtained from Rancho Solano in California. The third soil is San Francisco Bay Mud, which was obtained from Hamilton Air Force Base in California.

Previous research has shown that measurements of the residual friction angle are unaffected by the initial structure of the soil (Bishop et al., 1971). Therefore, it is reasonable to use remolded specimens for the measurement of residual shear strengths. By using uniform batches of remolded clays, and consolidating samples to the desired consistency in the laboratory, the scatter in results that would result from use of undisturbed samples can be avoided, and important questions regarding undrained strength on slickensided surfaces can be addressed more readily.

Initially, both of the Rancho Solano Clays and the San Francisco Bay Mud were obtained as bulk samples in five-gallon buckets. Batch mixing of all three soils was performed to ensure uniformity of the test specimens by thoroughly mixing and remolding the soils at water contents above their Liquid Limits. The soils were then pushed through the #40 sieve to remove larger particles that could interfere with the preparation of slickensided failure surfaces in the laboratory. The soils were then remixed to ensure uniformity. The measured index properties for the three "#40-pushed" soils are given in Table 3-1. The grain size curves for these soils are shown in Figure 3-1.

Table 3-1: Rancho Solano Clay and San Francisco Bay Mud Index Properties.

Clay	USCS Classification	LL	PL	PI	Clay Fraction	Specific Gravity
Rancho Solano Clay #1	Brown Fat Clay (CH)	61	25	36	53	2.65
Rancho Solano Clay #2	Brown Lean Clay (CL)	41	19	22	27	2.79
San Francisco Bay Mud	Grey Elastic Silt (MH)	85	38	47	47	2.70

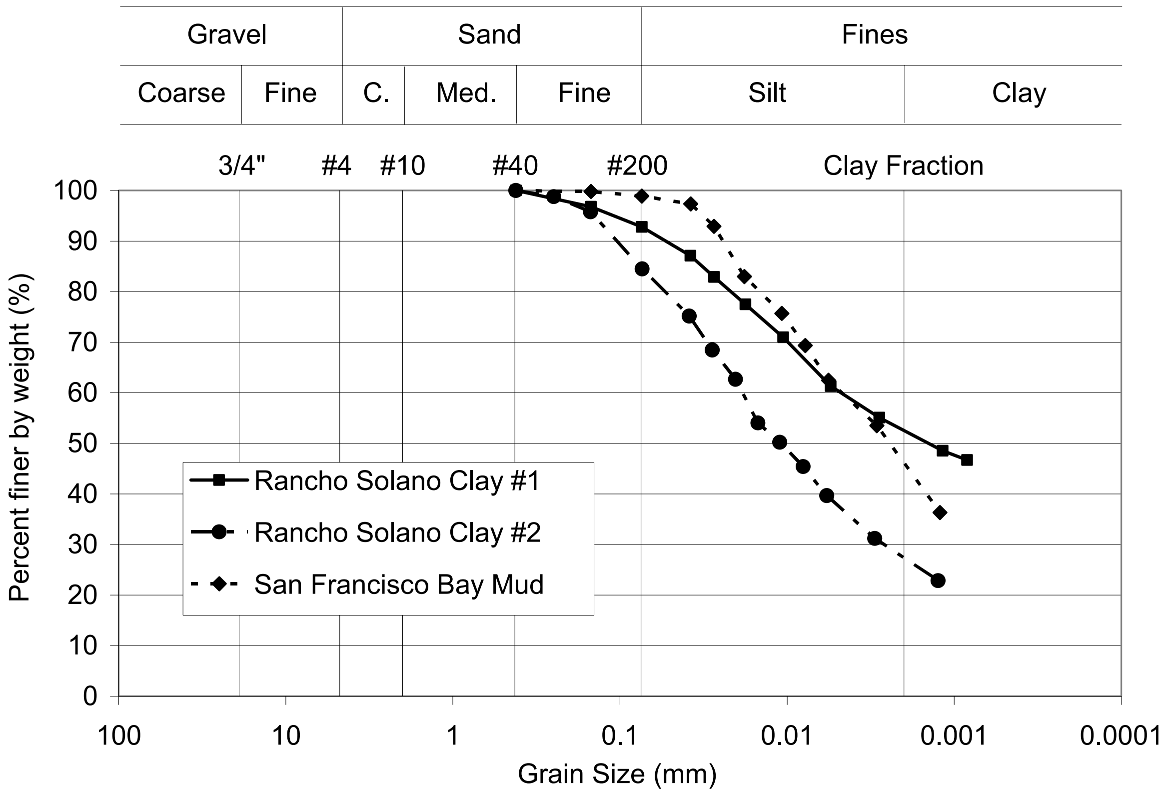


Figure 3-1. Rancho Solano Clay and San Francisco Bay Mud grain size curves.

Additional index tests were performed at frequent intervals throughout the duration of the laboratory testing program. Consistency of the measured index properties indicated that the batch mixing process was successful in creating uniform test specimens. Also, it should be noted that the clays were not oven dried at any point during the soil preparation process, because this has been found to affect the strength behavior and Atterberg Limits of clays.

Upon completion of batch mixing and soil preparation, the clays were consolidated using a batch consolidometer to reduce their water contents. A load-increment ratio of 1 was used for consolidation, and each batch of clay was consolidated to a maximum vertical consolidation pressure of 50 psi.

CHAPTER 4: RING SHEAR TESTING PROGRAM

The laboratory tests described in this chapter were conducted to measure the drained residual strength and the fast residual strength along slickensided discontinuities in the Rancho Solano Clay and the San Francisco Bay Mud. The drained residual strengths and fast residual strengths were measured by performing a series of strain-controlled ring shear tests at varying rates of shear. The drained ring shear tests are an essential part of the laboratory test program, because they provide an accurate baseline value for the drained residual strength that can be used to evaluate the accuracy of the direct shear and centrifuge test results. The fast ring shear tests are useful because they provide an improved understanding of the fast residual shear behavior along slickensided discontinuities.

Ring shear tests are the recommended method for developing the baseline values for drained residual strength, because of the ability of the ring shear device to apply large shear displacements without any reversal in the direction of shear. This allows for more complete particle orientation along the shearing plane, and a more accurate measurement of the drained residual strength than would be achieved in traditional direct shear or triaxial tests (Bishop et al., 1971).

The ring shear tests described in this chapter were performed at Virginia Tech using Bromhead ring shear devices (Bromhead, 1979) built by Wykeham Farrance Engineering Ltd. Figure 4-1 is a picture of the type of Bromhead ring shear apparatus that was used. Residual strengths measured in the Bromhead ring shear device agree well with residual strengths from back-analysis of failed slopes, which indicates that the Bromhead ring shear apparatus provides an accurate measurement of the drained residual shear strength (Bromhead and Dixon, 1986; Stark and Eid, 1992).

The Drained Residual Shear Strength of Rancho Solano Clay #1

In order to develop a baseline value for the drained residual shear strength of Rancho Solano Clay #1, a series of drained ring shear tests were conducted using the ring shear test procedure that is described in ASTM D 6467-99. ASTM D 6467-99 provides standardized guidance for drained ring shear testing of cohesive soils, and tests conducted using this approach should give results that are consistent with what would be measured by engineers in practice using the Bromhead ring shear device.

Figure 4-1. Bromhead ring shear apparatus.

Test specimens were prepared and tested according to the method described in ASTM D 6467-99. Using this approach, remolded specimens were first mixed at a water content near the liquid limit, and then pushed through the #40 sieve (which has an opening of 0.0165 inches) to remove larger particles that could get caught between the top platen and the side walls of the specimen container. The clay that passed the #40 sieve was then placed in the Bromhead ring shear specimen container, and consolidated using a series of load steps to the highest desired normal stress that would be on the shear strength envelope for that specimen. During consolidation, the normal force was applied by a dead-weight lever-arm system, and vertical displacements were recorded in order to ensure that pore pressures for a given load step were completely dissipated before the next load was applied.

Once consolidation was complete, the test specimen was unloaded to the lowest desired normal stress that would be on the shear strength envelope for that specimen, and allowed to come to pore pressure equilibrium. Once equilibrium was achieved, the specimen was presheared for one complete revolution (a shear displacement of 10.5 inches) at a rate of 0.58 in/min in order to create a slickensided failure plane. This allowed for a more rapid measurement of the drained residual shear strength, because a slickensided failure surface was already present in the specimen before slow shearing was begun.

Once the pore pressures that were induced by preshearing had dissipated, slow shearing was begun. In order to minimize shear-induced pore water pressures, slow-shear displacement rates were selected using the following equation (from ASTM D 6467-99):

$$\text{Displacement Rate} = \frac{\text{Displacement at Failure}}{\text{Time to Failure}} = \frac{0.2''}{50 \times t_{50}} \qquad (4\text{-}1)$$

In the above equation, t_{50} is the time required for the specimen to achieve 50% consolidation under the specified normal stress. Table 4-1 lists the calculated displacement rates for the ASTM standard ring shear tests on Rancho Solano Clay #1. Based on the data given in Table 4-1, slow shearing of all specimens was performed at a displacement rate of 0.00071 in/min. This is a conservative lower bound displacement rate that allowed for adequate pore pressure dissipation during shear. This displacement rate is also the lowest displacement rate that can be applied by the Wykeham Farrance Bromhead ring shear devices in the Virginia Tech laboratory.

Table 4-1: Calculated Displacement Rates for ASTM Standard Ring Shear Tests on Rancho Solano Clay #1

ASTM Standard Ring Shear Test Number(s)	Displacement Rate Calculated Using Casagrande t_{50} (in/min)	Displacement Rate Calculated Using Taylor t_{50} (in/min)
R1-052003-1, R1-052003-2, and R1-052003-3	0.0024	0.0024
R1-060303-1, R1-060303-2, and R1-060303-3	0.0027	0.0049
R1-061003-1, R1-061003-2, and R1-061003-3	0.0020	0.0033
R1-061903-1, R1-061903-2, and R1-061903-3	0.0012	0.0016

Slow shearing was continued at a displacement rate of 0.00071 in/min, until the stress-displacement curve had reached a constant minimum shear stress. Shearing was then stopped, because a constant measurement of minimum shear stress indicates that the residual strength state has been achieved.

After completion of the first shearing stage, the normal stress on the specimen was increased. Pore pressures induced by this increase in normal stress were allowed to dissipate, and the second shearing stage was begun. Once the residual strength state was achieved, the normal stress was increased again, for a third shearing stage. Upon completion of the third shearing stage, the three measured values of residual strength were used to construct a failure envelope. This "multistage" approach to testing reduced testing time considerably, because it was only necessary to prepare and consolidate one specimen in order to generate a three-point failure envelope.

A total of four ASTM standard ring shear tests were performed on Rancho Solano Clay #1. A multistage test approach was used, and each specimen was tested at normal stresses of 7.5 psi, 14.6 psi, and 28.8 psi. This resulted in a total of twelve different measurements of residual shear stress for this soil. Complete data sheets for each ring shear test are given in Appendix A. Statistical analysis results of the measured residual shear stresses are given in Table 4-2. A plot of average residual shear stress vs. normal stress is given in Figure 4-2. The standard deviations of the residual shear stresses were calculated using the nonbiased method, given by the following formula:

$$\text{Standard Deviation} = \sqrt{\frac{n\sum x^2 - (\sum x)^2}{n(n-1)}} \qquad (4\text{-}2)$$

where: x = sample value, and
n = total number of samples.

Table 4-2: Residual Shear Stresses Measured in ASTM Standard Ring Shear Tests on Rancho Solano Clay #1

Normal Stress (psi)	Number of Tests Performed	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.5	4	3.2	0.23	3.0	3.5
14.6	4	6.1	0.17	5.9	6.3
28.8	4	11.0	0.13	10.9	11.2

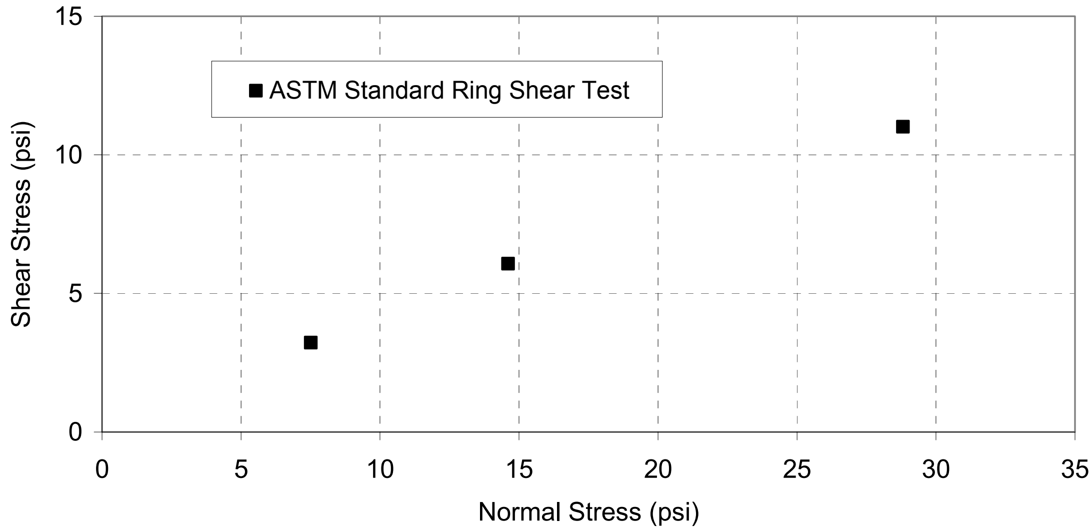


Figure 4-2. Average residual shear stresses measured in ASTM standard ring shear tests on Rancho Solano Clay #1.

Results from the ASTM standard ring shear tests can be interpreted using the secant phi approach discussed by Stark and Eid (1994). This approach assumes that there is no residual cohesion, which leads to the following formula for calculation of the secant residual friction angle:

$$\text{Secant Residual Friction Angle} = \tan^{-1}\left(\frac{\text{Residual Shear Stress}}{\text{Normal Stress}}\right) \qquad (4\text{-}3)$$

Statistical analysis results of the measured secant residual friction angles are given in Table 4-3. Standard deviations of the secant residual friction angles were calculated using Equation 4-2. A plot of average secant friction angle vs. normal stress is given in Figure 4-3. The bands surrounding each value of average secant friction angle in Figure 4-3 are the minimum and maximum secant residual friction angles measured at that normal stress.

Table 4-3: Values of Secant Residual Friction Angle Measured in ASTM Standard Ring Shear Tests on Rancho Solano Clay #1

Normal Stress (atm)	Number of Tests Performed	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	4	23.2	1.5	21.5	25.0
1.0	4	22.6	0.6	22.0	23.3
2.0	4	20.9	0.2	20.7	21.2

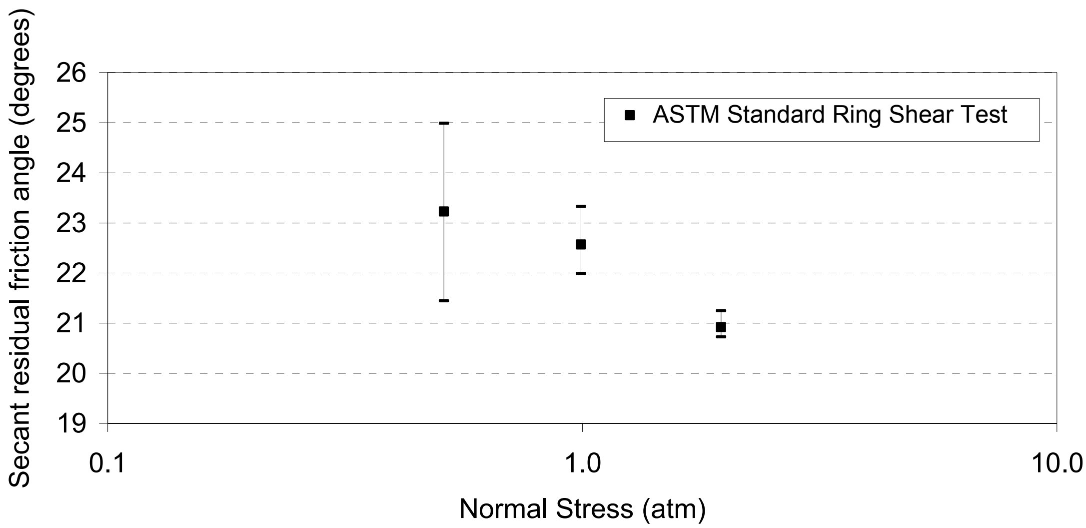


Figure 4-3. Values of secant residual friction angle measured in ASTM standard ring shear tests on Rancho Solano Clay #1.

Effect of Test Procedure on the Drained Residual Shear Strength of Rancho Solano Clay #1

In order to ensure the accuracy of the measured residual strengths, a series of drained ring shear tests were conducted on Rancho Solano Clay #1 using a ring shear test procedure that was designed to reduce the effects of friction in the Bromhead ring shear device. Of specific concern was the effect of wall friction between the top porous stone and the walls of the specimen container, which can lead to unconservative measurements of residual strength (Stark and Vettel, 1992). Because the magnitude of wall friction that is developed during shear is directly linked to the settlement of the top porous stone into the specimen container, the easiest way to reduce the effect of wall friction is to minimize the settlement of the top platen.

The three primary causes of top platen settlement in the Bromhead ring shear device are consolidation settlement, settlement due to extrusion during preshearing, and settlement due to extrusion during shearing. Although it is not possible to eliminate these sources of top platen settlement completely, a number of modifications to the ASTM test procedure can be made to reduce the overall top platen settlement during the tests. The modifications made to the ASTM test procedure are as follows:

- Test specimens were prepared at a lower water content in order to reduce the total amount of top platen settlement that occurs during consolidation. This was achieved by preconsolidating remolded test specimens in a batch consolidometer to a normal stress of 50 psi prior to their placement in the Bromhead ring shear specimen container.

- Preshearing of the specimens was not performed, in order to eliminate the top platen settlement that typically occurs during this phase of the test. Although some extrusion and top platen settlement does still occur when the slickensided failure surface is created during slow shear, its magnitude is significantly less than what is typically observed during the more rapid preshearing process.

- Multistage shearing of the specimens was not performed, in order to reduce the top platen settlement that occurs due to extrusion during shear. By testing a new specimen at each normal stress, it was possible to avoid the effect of accumulated extrusion and settlement that occurs at the second and third normal stresses in a multistage test.

A total of twenty-six "reduced platen settlement" ring shear tests were performed on Rancho Solano Clay #1 using the modifications to the ASTM ring shear test procedure discussed above. All specimens were sheared at a displacement rate of 0.00071 in/min. Specimens were tested at five normal stresses: 7.5 psi, 14.6 psi, 28.8 psi, 50.1 psi, and 85.6 psi. Complete data sheets for the "reduced platen settlement" ring shear tests are given in Appendix A. Statistical analysis results of the measured residual shear stresses for the "reduced platen settlement" ring shear tests are given in Table 4-4. A plot of average residual shear stress vs. normal stress for the "reduced platen settlement" ring shear tests and the "ASTM standard" ring shear tests is given in Figure 4-4. Statistical analysis results of the measured secant residual friction angles for the "reduced platen settlement" ring shear tests are given in Table 4-5. A plot of average secant friction angle vs. normal stress for the "reduced platen settlement" ring shear tests and the "ASTM standard" ring shear tests is given in Figure 4-5. Figure 4-5 also shows the minimum and maximum secant residual friction angles measured at each normal stress.

Table 4-4: Residual Shear Stresses Measured in "Reduced Platen Settlement" Ring Shear Tests on Rancho Solano Clay #1

Normal Stress (psi)	Number of Tests Performed	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.5	7	2.7	0.14	2.5	2.9
14.6	4	5.2	0.12	5.1	5.3
28.8	4	9.9	0.22	9.6	10.1
50.1	6	16.6	0.47	16.1	17.3
85.6	5	27.3	0.76	26.6	28.3

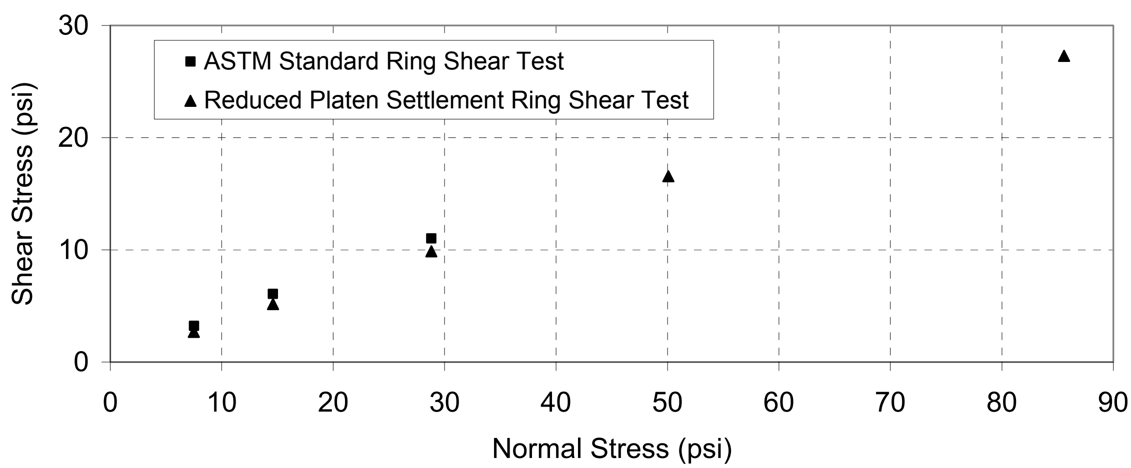


Figure 4-4. Residual shear stresses measured in "reduced platen settlement" ring shear tests on Rancho Solano Clay #1.

Table 4-5: Values of Secant Residual Friction Angle Measured in "Reduced Platen Settlement" Ring Shear Tests on Rancho Solano Clay #1

Normal Stress (atm)	Number of Tests Performed	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	7	19.7	1.0	18.6	20.8
1.0	4	19.5	0.4	19.1	19.9
2.0	4	18.9	0.4	18.4	19.3
3.4	6	18.3	0.5	17.9	19.1
5.8	5	17.7	0.5	17.2	18.3

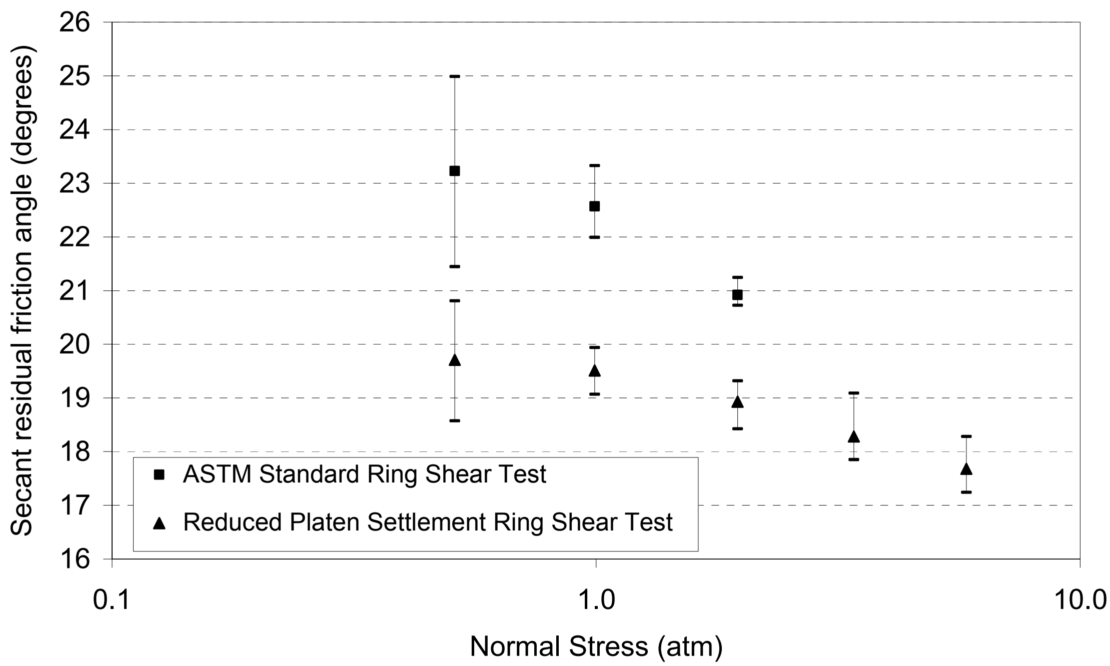


Figure 4-5. Values of secant residual friction angle measured in "reduced platen settlement" ring shear tests on Rancho Solano Clay #1.

The data presented in Figure 4-4 and Figure 4-5 shows the significant effect that wall friction has on residual strengths measured in the Bromhead ring shear device. These results show that the "reduced platen settlement" test approach reduces these wall friction effects. However, even if the "reduced platen settlement" test approach is used, wall friction in the Bromhead ring shear device will continue to affect measurements of the residual strength, because the procedure does not eliminate settlement of the top platen into the specimen container.

Effect of Device Modifications on the Drained Residual Shear Strength of Rancho Solano Clay #1

In order to check the accuracy of the residual strengths measured using the "reduced platen settlement" test approach, a series of drained ring shear tests was conducted on Rancho Solano Clay #1 using a modified Bromhead ring shear device designed to reduce the effects of wall friction.

Wall friction in the Bromhead ring shear device is developed as the top platen settles into the specimen container, due to the extrusion and entrapment of clay particles between the top platen and the side walls of the specimen container. By modifying the shape of the top platen, it is possible to reduce the entrapment of clay particles, thereby reducing wall friction. The modification involved beveling the inside and outside of the porous bronze top platen at a forty-five degree angle, as shown in Figure 4-6 and Figure 4-7. This reduced the possibility for entrapment of clay particles between the top platen and the walls of the specimen container. Consequently, even if top platen settlement occurs during the test, significant wall friction will not develop.

Figure 4-6. Side view that shows the difference between the original porous bronze platen (on the left) and the modified porous bronze platen (on the right).

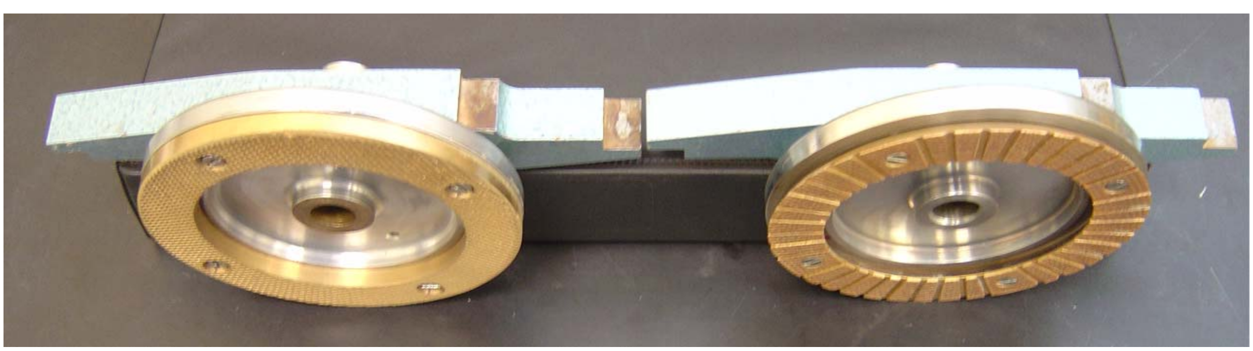


Figure 4-7. Angle view that shows the difference between the original porous bronze platen (on the left) and the modified porous bronze platen (on the right).

Twenty-six "modified platen" ring shear tests were performed on Rancho Solano Clay #1 using the "reduced platen settlement" test procedure in combination with the modification to the top platen described above. All specimens were sheared at a displacement rate of 0.00071 in/min. Specimens were tested at five normal stresses: 7.5 psi, 14.6 psi, 28.8 psi, 50.1 psi, and 85.6 psi. Complete data sheets for the "modified platen" ring shear tests are given in Appendix A.

Statistical analysis results of the measured residual shear stresses for the "modified platen" ring shear tests are given in Table 4-6. A plot of average residual shear stress vs. normal stress for the "modified platen" ring shear tests, the "reduced platen settlement" ring shear tests, and the "ASTM standard" ring shear tests is shown in Figure 4-8. Statistical analysis results of the measured secant residual friction angles for the "modified platen" ring shear tests are given in Table 4-7. A plot of average secant friction angle vs. normal stress for the "modified platen" ring shear tests, the "reduced platen settlement" ring shear tests, and the "ASTM standard" ring shear tests is given in Figure 4-9. Figure 4-9 also shows the minimum and maximum secant residual friction angles measured at each normal stress.

Table 4-6: Residual Shear Stresses Measured in "Modified Platen" Ring Shear Tests on Rancho Solano Clay #1.

Normal Stress (psi)	Number of Tests Performed	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.5	5	2.4	0.04	2.4	2.5
14.6	4	4.5	0.11	4.3	4.6
28.8	5	8.5	0.29	8.1	8.9
50.1	6	14.4	0.36	14.0	15.0
85.6	6	24.0	0.30	23.6	24.4

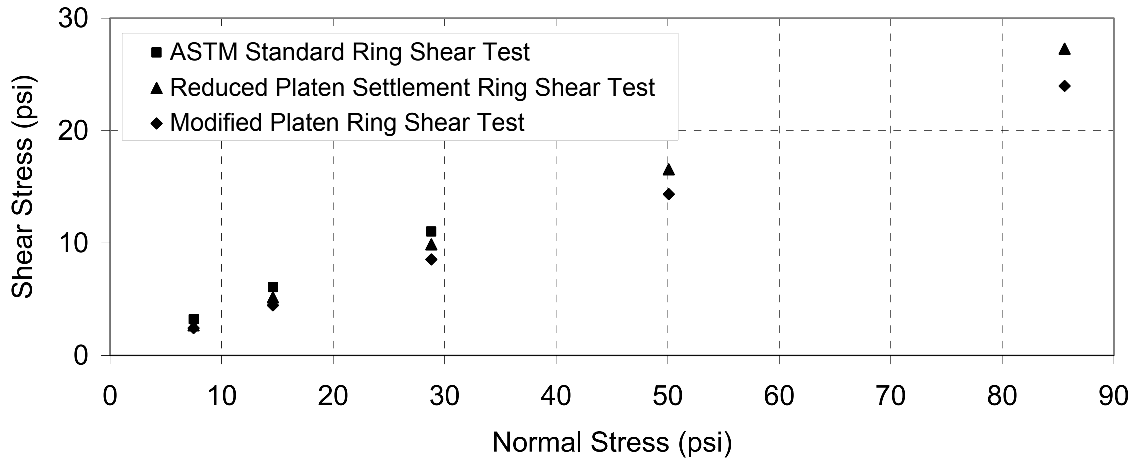


Figure 4-8. Residual shear stresses measured in “modified platen” ring shear tests on Rancho Solano Clay #1.

Table 4-7: Values of Secant Residual Friction Angle Measured in “Modified Platen” Ring Shear Tests on Rancho Solano Clay #1

Normal Stress (atm)	Number of Tests Performed	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	5	18.0	0.3	17.5	18.3
1.0	4	17.0	0.4	16.4	17.3
2.0	5	16.5	0.5	15.8	17.1
3.4	6	16.0	0.4	15.7	16.7
5.8	6	15.7	0.2	15.4	15.9

The data presented in Figure 4-8 and Figure 4-9 shows that the “modified platen” ring shear tests minimize the effect of wall friction in the Bromhead ring shear device, producing more accurate measurements of the drained residual strength than the “reduced platen settlement” test approach. Therefore, it appears that the most accurate results are achieved using the “modified platen” approach, which uses the “reduced platen settlement” test procedure in combination with the modified platen. The drained residual strength envelope for Rancho Solano Clay #1 determined using this technique is shown in Figure 4-10. This nonlinear failure envelope passes through the origin. The nonlinearity of the drained residual strength failure envelope agrees well with test data collected by other researchers (Stark and

Eid, 1994, and others). The assumption that the residual strength envelope passes through the origin is supported by Skempton's (1964) research on the residual strength of stiff clays.

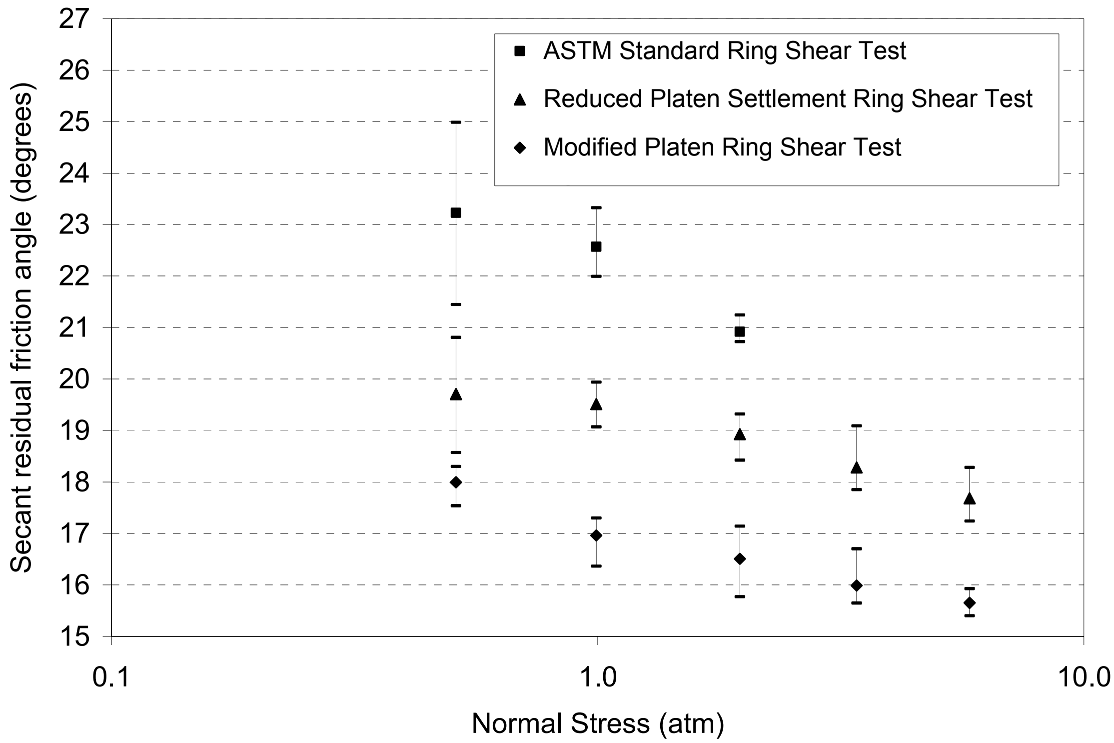


Figure 4-9. Values of secant residual friction angle measured in "modified platen" ring shear tests on Rancho Solano Clay #1.

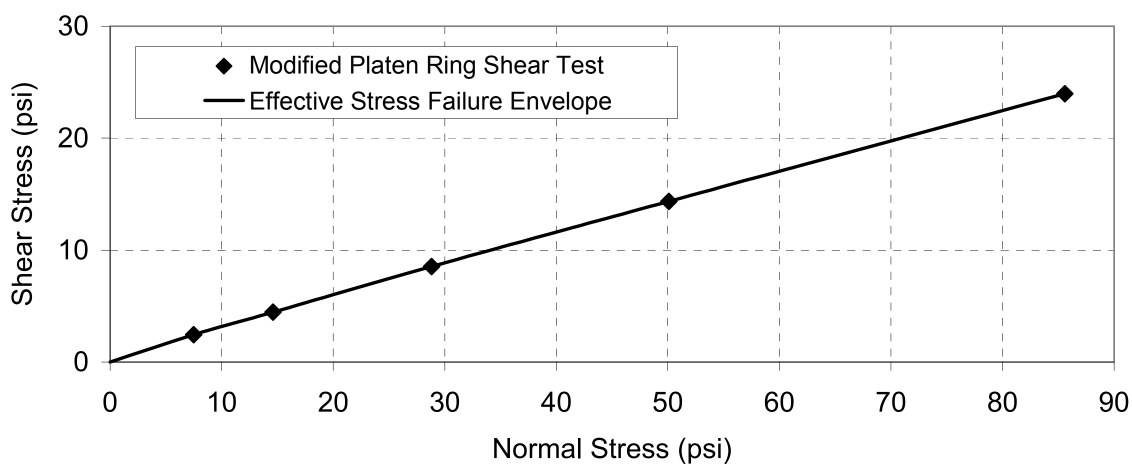


Figure 4-10. The drained residual strength envelope for Rancho Solano Clay #1.

One drawback to the "modified platen" test approach is the amount of soil extruded during shear. Because the top platen is beveled to reduce wall friction, there is less resistance to soil extrusion between the top platen and the side walls of the specimen container. Therefore, soil extrusion occurs more rapidly during shear, and the amount of specimen extruded during a test can be very significant. This limits the total shear displacement that can be applied to a specimen, because it is possible to extrude the entire specimen during shear. Additionally, since the amount of vertical displacement that occurs during shear is primarily controlled by the amount of soil extruded, measurements of vertical displacement cannot be correlated to change in soil volume or void ratio.

The Drained Residual Shear Strength of Rancho Solano Clay #2

The drained residual shear strength of Rancho Solano Clay #2 was measured using the "modified platen" ring shear test procedure described in the previous section.

A total of 15 "modified platen" ring shear tests were performed on Rancho Solano Clay #2. All specimens were sheared at a displacement rate of 0.00071 in/min. Specimens were tested at four normal stresses: 7.5 psi, 14.6 psi, 28.8 psi, and 50.1 psi. Complete data sheets for these ring shear tests are given in Appendix A. Statistical analysis results of the measured residual shear stresses are given in Table 4-8. A plot of average residual shear stresses vs. testing normal stresses is given in Figure 4-11. Statistical analysis results of the measured secant residual friction angles are given in Table 4-9. A plot of average secant friction angles vs. normal stresses is given in Figure 4-12. Figure 4-12 also shows the minimum and maximum secant residual friction angles measured at each normal stress.

Table 4-8: Residual Shear Stresses Measured in "Modified Platen" Ring Shear Tests on Rancho Solano Clay #2.

Normal Stress (psi)	Number of Tests Performed	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.5	9	3.3	0.30	2.8	3.6
14.6	3	5.8	0.25	5.6	6.1
28.8	1	10.6	N/A	N/A	N/A
50.1	2	17.1	0.25	16.9	17.2

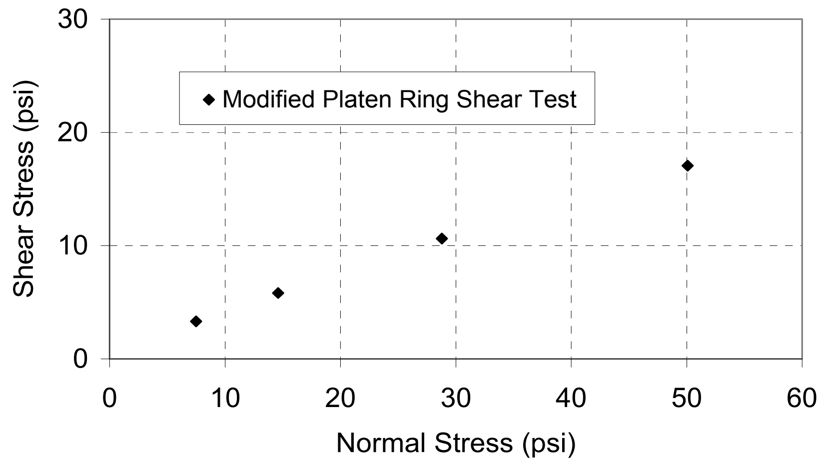


Figure 4-11. Residual shear stresses measured in "modified platen" ring shear tests on Rancho Solano Clay #2.

Table 4-9: Values of Secant Residual Friction Angle Measured in "Modified Platen" Ring Shear Tests on Rancho Solano Clay #2

Normal Stress (atm)	Number of Tests Performed	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	9	23.7	1.9	20.7	25.7
1.0	3	21.7	0.9	21.0	22.7
2.0	1	20.3	N/A	N/A	N/A
3.4	2	18.8	0.3	18.6	19.0

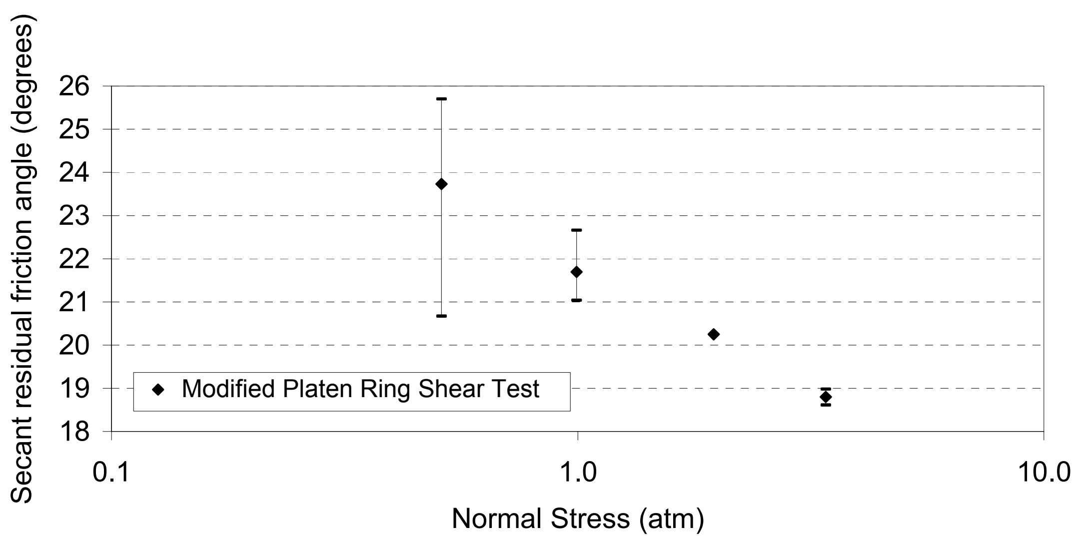


Figure 4-12. Values of secant residual friction angle measured in "modified platen" ring shear tests on Rancho Solano Clay #2.

Analysis of the data in Table 4-8, Table 4-9, Figure 4-11 and Figure 4-12 indicates that the residual strength failure envelope for Rancho Solano Clay #2 is nonlinear. Additionally, there is consistency between the residual strength values measured in each of the ring shear devices, which gives confidence in the measured drained residual strengths. Based on the test results, a drained residual strength envelope can be constructed using the same approach that was used for Rancho Solano Clay #1. The resulting drained residual strength envelope for Rancho Solano Clay #2 is shown in Figure 4-13.

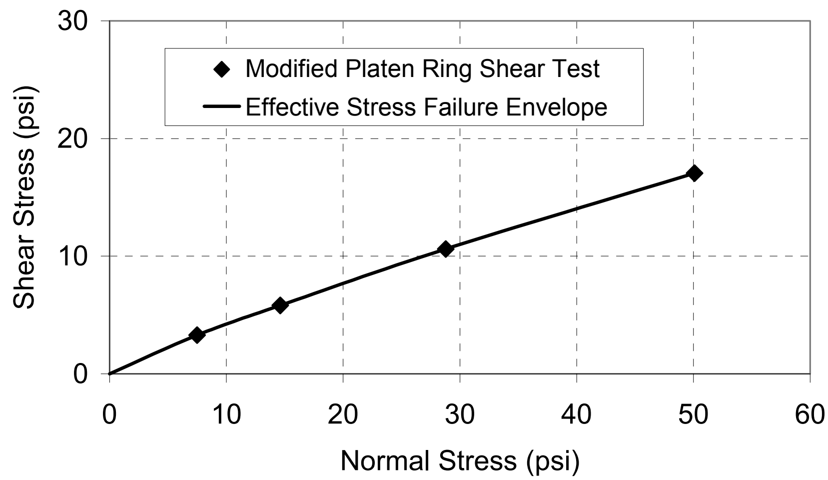


Figure 4-13. The drained residual strength envelope for Rancho Solano Clay #2.

The Drained Residual Shear Strength of San Francisco Bay Mud

Ring shear tests were also used to measure the drained residual shear strength of San Francisco Bay Mud. A total of twelve “modified platen” ring shear tests were performed on San Francisco Bay Mud using the “reduced platen settlement” test procedure and the modified platen. All specimens were sheared at a displacement rate of 0.00071 in/min. Specimens were tested at five normal stresses: 7.5 psi, 14.6 psi, 28.8 psi, 50.1 psi, and 85.6 psi. Complete data sheets for these ring shear tests are given in Appendix A. Statistical analysis results of the measured residual shear stresses for these ring shear tests are given in Table 4-10. A plot of average residual shear stress vs. normal stress for these ring shear tests is given in Figure 4-14. Statistical analysis results of the measured secant residual friction angles are given in Table 4-11. A plot of average secant friction angle vs. normal stress is given in Figure 4-15. Figure 4-15 also shows the minimum and maximum secant residual friction angles measured at each normal stress.

Table 4-10: Residual Shear Stresses Measured in "Modified Platen" Ring Shear Tests on San Francisco Bay Mud.

Normal Stress (psi)	Number of Tests Performed	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.5	2	2.7	0.03	2.7	2.8
14.6	4	4.9	0.05	4.8	4.9
28.8	2	8.7	0.02	8.7	8.7
50.1	2	14.7	0.36	14.4	14.9
85.6	2	24.9	0.25	24.7	25.0

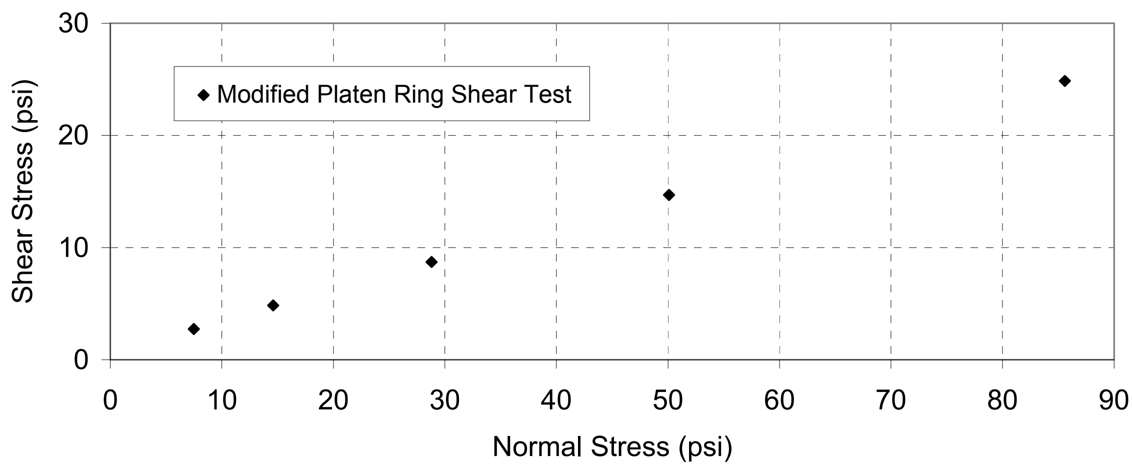


Figure 4-14. Residual shear stresses measured in "modified platen" ring shear tests on San Francisco Bay Mud.

As shown in Table 4-10, Table 4-11, Figure 4-14 and Figure 4-15, there was very little scatter in the test data. Additionally, good agreement was observed between the residual strength values measured in each of the ring shear devices, which gives a high degree of confidence in the measured drained residual strengths. Based on the test results, a drained residual strength envelope can be constructed using the same approach that was used for the two Rancho Solano Clays. The resulting nonlinear drained residual strength envelope for San Francisco Bay Mud is shown in Figure 4-16.

Table 4-11: Values of Secant Residual Friction Angle Measured in "Modified Platen" Ring Shear Tests on San Francisco Bay Mud

Normal Stress (atm)	Number of Tests Performed	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	2	20.1	0.2	19.9	20.2
1.0	4	18.4	0.2	18.2	18.6
2.0	2	16.8	0.04	16.8	16.9
3.4	2	16.3	0.4	16.1	16.6
5.8	2	16.2	0.2	16.1	16.3

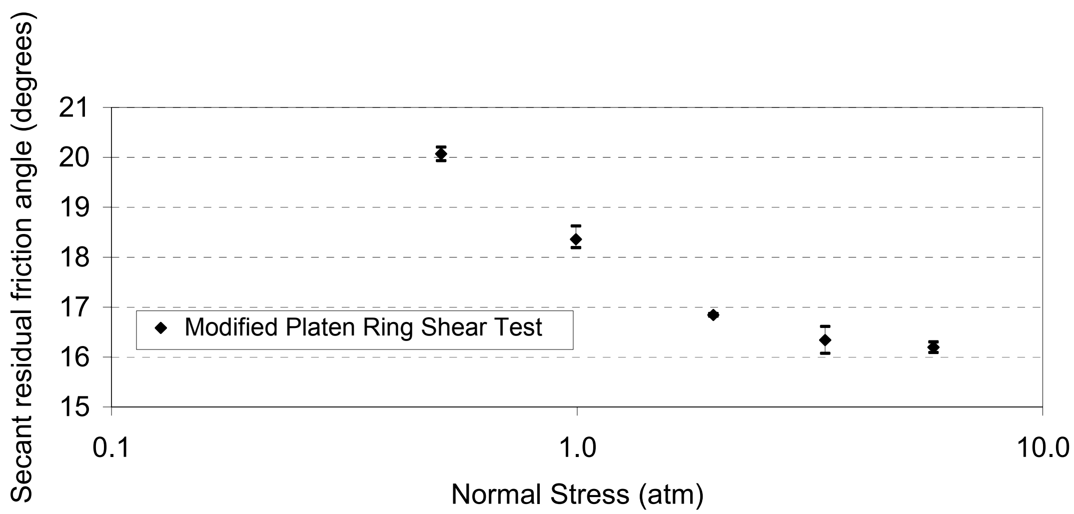


Figure 4-15. Values of secant residual friction angle measured in "modified platen" ring shear tests on San Francisco Bay Mud.

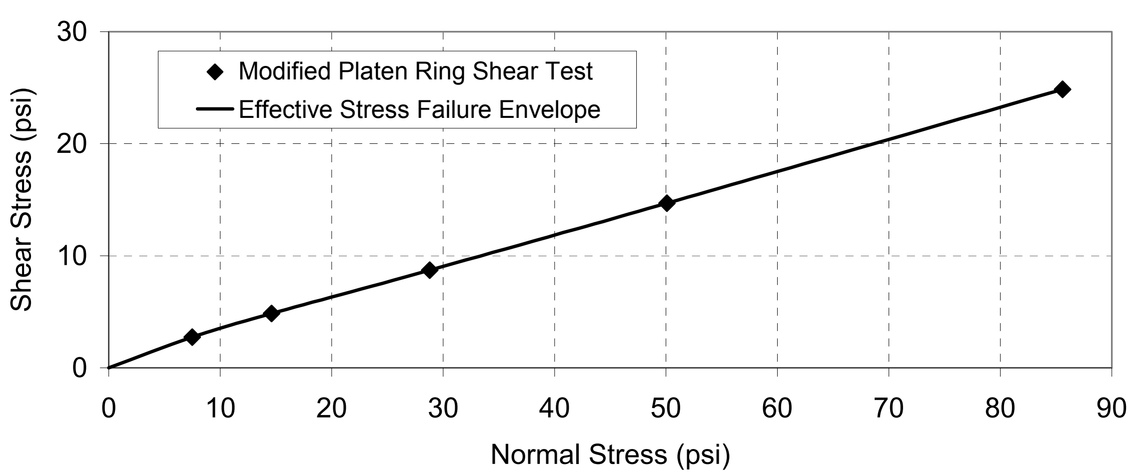


Figure 4-16. The drained residual strength envelope for San Francisco Bay Mud.

The Fast Residual Shear Strength of Rancho Solano Clay #1

As part of the ring shear testing program for Rancho Solano Clay #1, a series of fast ring shear tests were also conducted in the Bromhead ring shear device. The purpose of these tests was to try to develop an understanding of the fast residual shear strength along existing slickensided discontinuities.

Although it is not possible to control boundary drainage conditions directly in the Bromhead ring shear device, it was hoped that fast shearing would provide reasonably accurate measurements of undrained strength along the slickensided surface. This appears to be a reasonable expectation because dissipation of shear-induced pore pressures is inhibited by the low permeability of the clay soil, combined with the short test duration.

The fast residual ring shear testing program was developed using the same rationale and test approach that was first proposed by Skempton (1985) for fast-shear testing in the NGI-type ring shear device. Using Skempton's (1985) approach, a clay specimen is first sheared slowly to create a slickensided failure surface, then sheared rapidly to measure the undrained shearing response along the slickensided shear surface, and then slowly again to re-establish the drained residual condition. This approach has also been employed successfully by other researchers using the NGI-type ring shear device (Lemos et al, 1985; Tika et al, 1996; Vesseley and Cornforth, 1998; and Tika and Hutchinson, 1999), as discussed in Chapter 2. The usefulness of this test approach for measuring fast residual strengths in the Bromhead ring shear device has not been explored.

In order to measure the fast residual shear strength along slickensided surfaces in Rancho Solano Clay #1, it was first necessary to create a slickensided failure surface within the ring shear test specimen. Slickensided failure surfaces were created in the ring shear specimens by preparing and testing specimens using the "modified platen approach". For each fast shear test, initial drained shearing was performed at a displacement rate of 0.00071 in/min (Stage 1 shearing), to create a slickensided failure surface within the ring shear test specimen.

During a typical fast shear test, the initial drained shearing stage was continued until the residual condition had been reached. At that point, drained shearing was stopped, and fast shearing (Stage 2 shearing) was begun. Fast shearing was performed at a rate of 1.75 in/min. Fast shearing was continued for two full revolutions in the ring shear device, which corresponds to a shear displacement of approximately 21 inches. Fast shearing was then stopped, and drained shearing was recommenced at a displacement rate of 0.00071 in/min (Stage 3 shearing). The third shearing stage was continued until the drained residual condition had been achieved. Figure 4-17 shows the fast shear response of three Rancho Solano Clay #1 specimens that were tested at normal stresses of 28.8, 50.1, and 85.6 psi.

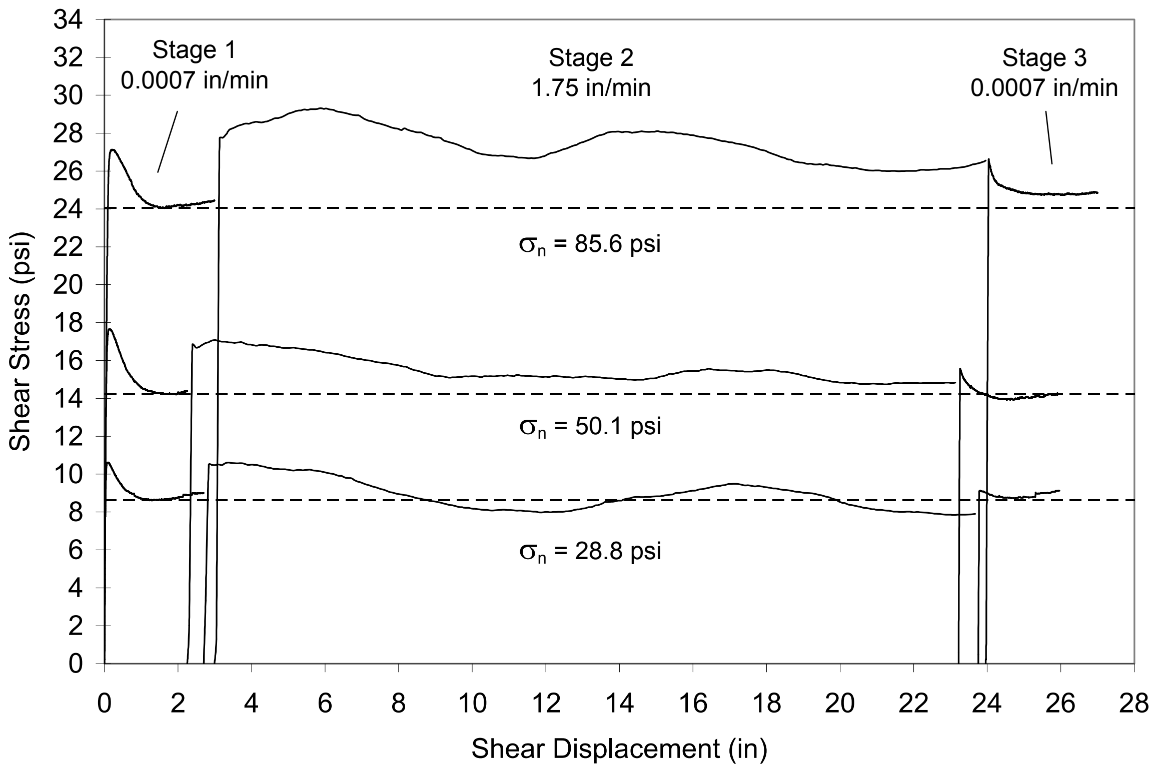


Figure 4-17. The fast shear response of Rancho Solano Clay #1.

The shear stress plots shown in Figure 4-17 are representative curves selected from the results of 11 tests. The fast shear tests indicated that there is an increased "threshold strength" at the beginning of fast shearing. As fast shearing is continued, there is an additional increase in strength, to the "fast maximum" strength. As fast shearing continues past the fast maximum strength, the strength drops to a "fast minimum" strength. As shown in Figure 4-17, the fast minimum strength was sometimes higher than the drained residual strength and sometimes lower than the drained residual strength. At the end of fast shearing, once drained shearing is recommenced, a new slow peak strength is observed, which is higher than the drained residual strength. The strength then drops again to the drained residual strength, which coincides with the initial drained residual strength.

These results agree in most respects with what has been observed for clayey soils in the NGI-type ring shear apparatus (Lemos et al., 1985; Tika et al, 1996; Vesseley and Cornforth, 1998; and Tika and Hutchinson, 1999). However, there is one significant difference between the fast minimum shear behavior in the Bromhead ring shear device and what has been observed in the NGI-type ring shear device. This difference is the cyclic up-and-down nature of the stress-displacement curve, which is clearly evident in Figure 4-17. This cyclic increase and decrease in shear stress is probably not a true soil behavior phenomenon, and could be caused by either of the following mechanisms:

One possibility is the replacement of soil particles along the shearing plane, which might occur as follows: As soil is extruded from the shearing plane, oriented clay particles are replaced by clay particles that have not been completely sheared to the residual condition. The strength of these non-oriented particles would be higher, and additional shearing would be necessary to orient the particles along the shearing plane. Cycles of clay particle extrusion, replacement by non-oriented particles, and orientation of clay particles along the shearing plane might cause variations in the measured shear stress. Unfortunately, the top platen modifications that are necessary to reduce wall friction in the Bromhead ring shear device also allow soil extrusion at a more rapid rate than usual, which would exacerbate this behavior, if it does occur.

A second possible mechanism for the observed "pumping" behavior is a machine effect that may be caused by subtle shifting of the top platen during shear. Figure 4-18 shows the fast shear response of a Rancho Solano Clay #1 specimen that was sheared to large displacements in the Bromhead ring shear device. The observed peaks and troughs in the stress-displacement curve occur on a cyclic basis, with approximately one full revolution (360°) of the specimen between successive peaks in the measured shear stress. This strongly suggests that the cyclic increase and decrease in measured shear resistance is a machine effect, probably wobbling of the top platen, and does not represent real soil behavior.

This phenomenon makes it impossible to quantify the value of the fast minimum residual strength. Even from a qualitative standpoint, in some cases it is not clear whether the fast minimum strength is higher than or lower than the drained residual strength (as shown by the σ_n = 28.8 psi test in Figure 4-17).

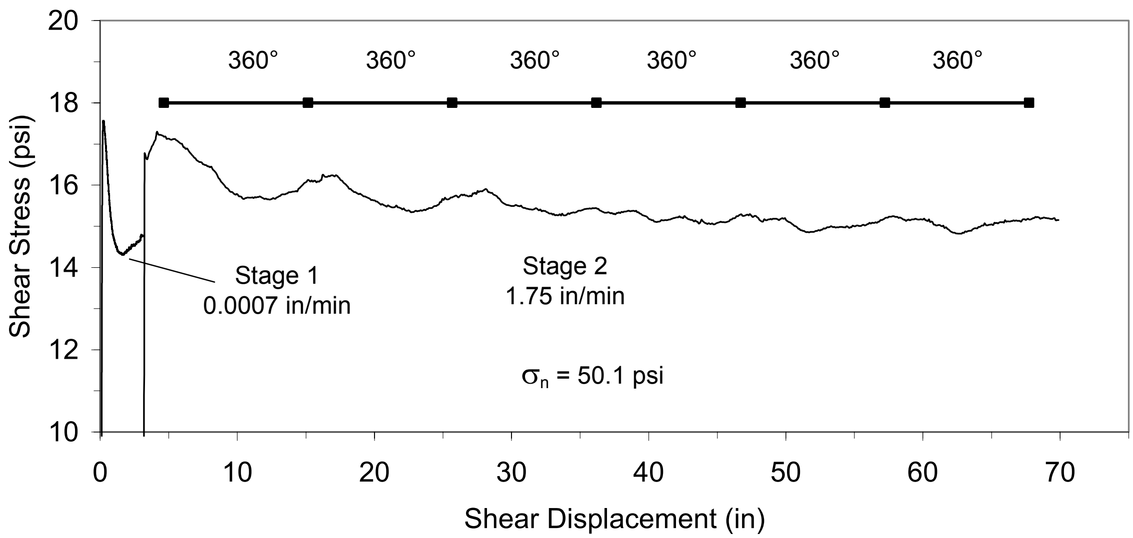


Figure 4-18. The fast shear response of Rancho Solano Clay #1 sheared to large displacements in the Bromhead ring shear device.

In addition to the physical problems of extrusion and top platen shifting during fast shear tests, there is also one significant theoretical problem with using the Bromhead ring shear device to measure the fast strength of clays. This has to do with the pore pressure response of the soil surrounding the slickensided failure plane.

As discussed in Chapter 2, Skempton (1985) has shown that rapid shearing along pre-existing slickensided discontinuities can lead to significant gains in strength above the drained residual strength condition. Skempton (1985) hypothesized that this strength gain is due to disturbance of the originally ordered clay particles, which causes a transition from the sliding mode to the turbulent mode of failure. As the clay particles along the shearing plane are disturbed, negative pore pressures are developed along the shearing plane, which leads to the development of a negative pore pressure gradient into the surrounding soil. These negative pore pressures are dissipated as shear continuous, which is what causes a decrease in strength from the “fast maximum” condition to the “fast minimum” condition.

With the NGI-type ring shear device (used by Skempton and others for fast ring shear testing), there is a significant amount of clay on either side of the failure plane. The presence of this clay is essential for the development of negative pore pressures along the failure plane during fast shear. However, in the Bromhead ring shear device, shearing takes place at or very close to the top platen. Therefore, the drainage path length from the shearing plane to the closest free-draining boundary is very short, and any negative pore pressures developed are dissipated relatively quickly. This leads to a pore pressure response that is different than what would occur in the field, or in the NGI-type ring shear device. Consequently, the fast residual strengths measured in the Bromhead ring shear device may not match the fast-shear strengths in the field or in the NGI-type ring shear device.

In conclusion, a number of practical and theoretical problems are involved in using the Bromhead ring shear device to measure fast residual shear strengths. Consequently, the fast ring shear test results for Rancho Solano Clay #1 were discarded, and the fast ring shear testing program was discontinued. The NGI-type ring shear device appears to be better suited for this type of test.

CHAPTER 5: LABORATORY TESTING OF SLICKENSIDED SURFACES

The laboratory tests described in this chapter were conducted to measure drained residual strength, fast residual strength, and cyclic shear strength along pre-formed slickensided discontinuities in Rancho Solano Clay and San Francisco Bay Mud. Drained residual strengths were measured by performing strain-controlled direct shear tests and triaxial tests at slow rates of shear on specimens that contained pre-formed slickensided failure surfaces. Fast residual strengths were measured by performing strain-controlled direct shear tests at fast rates of shear on slickensided specimens. Cyclic shear strength was measured by performing stress-controlled cyclic direct shear tests on slickensided specimens.

The drained direct shear tests and triaxial tests are a valuable part of the laboratory test program because they provide a means of evaluating the effectiveness of the slickenside preparation and polishing process. The fast direct shear tests are useful because they provide improved understanding of the fast shear behavior along slickensided discontinuities. The cyclic direct shear tests provide an indication of how slickensided rupture surfaces behave under seismic loading conditions.

The direct shear tests described in this chapter were performed at Virginia Tech using two direct shear devices. Figure 5-1 shows the strain-controlled direct shear device used to perform the drained direct shear tests and fast direct shear tests. This device was built by Wykeham Farrance Engineering Ltd. Figure 5-2 shows the stress-controlled direct shear device that was used to perform the cyclic direct shear tests. This device was designed and constructed at Virginia Tech by modifying an existing simple shear device so that it could apply cyclic loading to a direct shear specimen. The triaxial tests described in this chapter were performed at Virginia Tech using automated triaxial test equipment manufactured by the GeoComp Corporation. The triaxial device that was used for testing is shown in Figure 5-3.

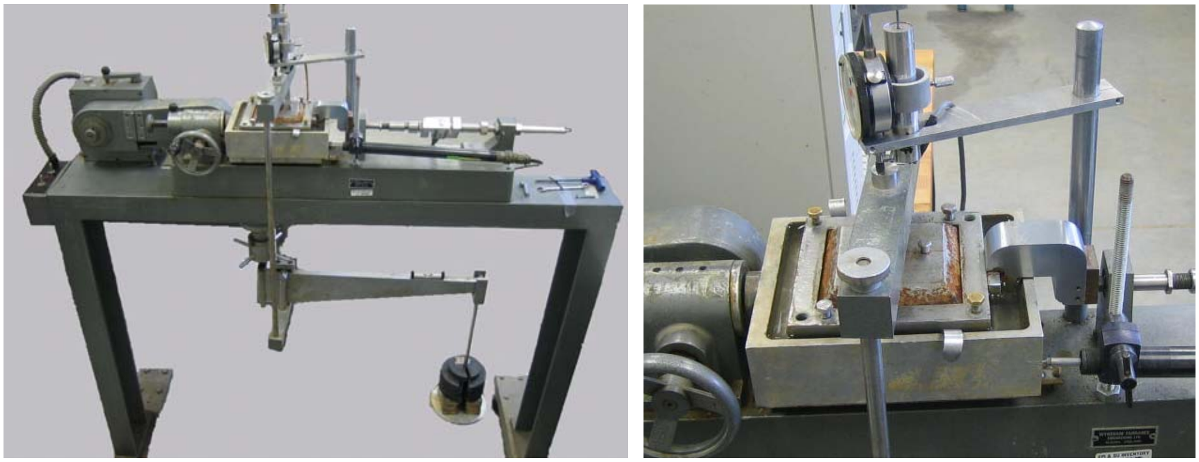


Figure 5-1. Wykeham Farrance direct shear apparatus.

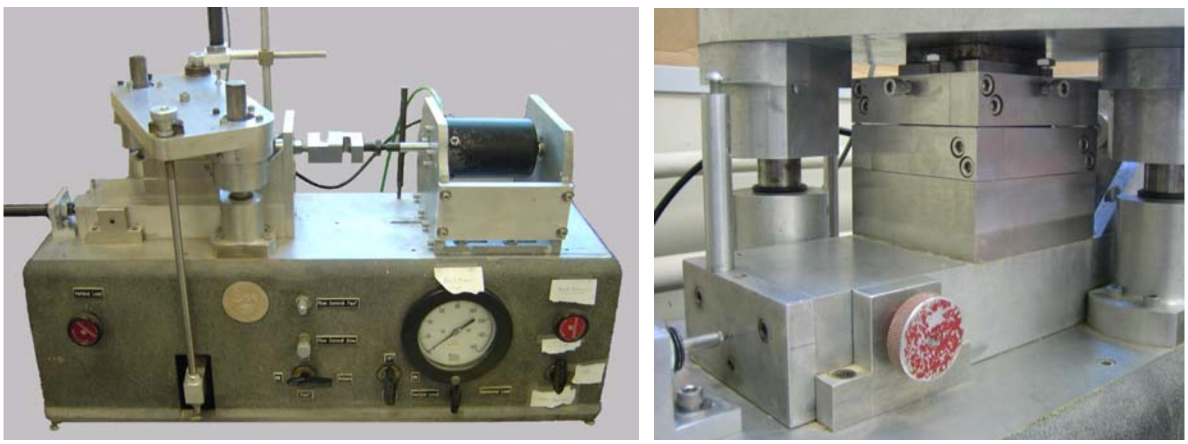


Figure 5-2. Virginia Tech cyclic direct shear device.

Figure 5-3. GeoComp automated triaxial test equipment.

Drained Direct Shear Testing of Rancho Solano Clay #1

In order to measure the drained residual shear strength along slickensided failure surfaces in Rancho Solano Clay #1, drained direct shear tests were conducted in general accord with the direct shear test method described in ASTM D 3080-98.

Direct shear tests can be used for measuring the shear strength along existing discontinuities in clayey soil (Skempton and Petley, 1967). Because remolded specimens were used in the testing program, it was necessary to develop a method for creating slickensided rupture surfaces in the laboratory. The effectiveness of the slickenside preparation process was evaluated by comparing the residual strengths measured along the prepared slickensided surfaces with those from the Bromhead ring shear testing program. This step was essential to ensure the validity of the slickenside preparation process.

As discussed in Chapter 3, the clay for the direct shear test specimens was prepared by first mixing it at a water content near its liquid limit. The clay was then pushed through the #40 sieve (opening size = 0.0165 inches) to remove larger soil particles that could interfere with the slickenside preparation process. The resulting clay slurry was then consolidated to 50 psi in a batch consolidometer to lower its water content.

Each of the direct shear test specimens was created by pressing the stiff clay from the batch consolidometer into the direct shear box and trimming it to the desired height. This formed test specimens that were 4″ x 4″ square, with heights of 0.5″. After trimming, the specimens were consolidated to 100 psi to stiffen the clay for easier slickenside formation.

After consolidation, the test specimen was repositioned so that its center coincided with the separation between the upper and lower shear boxes. The specimen was then wire cut to create a shear plane at the interface between the upper and lower shear boxes. The specimen could then separated into two pieces, an upper half and a lower half, which were polished to align clay particles in the direction of shear.

A specimen half was polished by shearing it along the entire length of a wet 12-inch frosted glass plate under moderate hand pressure. Four passes along the frosted glass plate were used for each half of the test specimen, taking care to remove the test specimen from the plate after each pass by shearing it off the edge of the glass, in order to not disturb the clay particles along the shearing plane. Care was taken to ensure that the direction of polishing coincided with the direction of shear that the specimen would experience in the direct shear device.

Once the two halves of the test specimen were polished, they were placed in the direct shear device, and the specimen was aligned such that the preformed shearing plane coincided with the shear plane between the two halves of the shear box. A bit of judgment was necessary at this stage, because the vertical position of the shear plane could change as a result of the specimen consolidation that occurred when the specimen was loaded to the desired testing normal stress. Achieving the appropriate vertical alignment of the shear plane took significant experience, and was critical for measuring the residual strength using this approach. Figure 5-4 shows the approach used to prepare the direct shear test specimens, and the final appearance of the failure plane after wet polishing.

Figure 5-4. Preparing a direct shear test specimen; (a) wire-cutting a direct shear specimen, (b) rubbing the cut plane on frosted glass to align clay particles, (c) the polished failure plane.

Once the polishing process was completed for each half of the test specimen, the two halves were reassembled and the specimen was placed in the direct shear device. The direct shear test was then begun by consolidating the specimen to the desired testing normal stress. During consolidation, the normal force was applied by a dead-weight lever-arm system, and vertical displacements were recorded in order to ensure that pore pressures were completely dissipated before the commencement of shear.

Upon completion of consolidation, the specimen was sheared using slow, displacement-controlled loading. In order to minimize shear-induced pore water pressures so that the test could be considered “drained”, slow-shear displacement rates were selected using the following equation (from ASTM D 3080-98):

$$\text{Displacement Rate} = \frac{\text{Displacement at Failure}}{\text{Time to Failure}} = \frac{0.5''}{50 \times t_{50}} \qquad (5\text{-}1)$$

In the above equation, t_{50} is the time required for the specimen to achieve 50% consolidation under the applied normal stress. The value of t_{50} was determined using data from consolidation tests and early direct shear tests. A displacement rate of 0.000123 in/min was used for drained direct shear testing of Rancho Solano Clay #1. This value is believed to be a conservative displacement rate that would ensure full pore pressure dissipation during shear.

Test specimens were sheared until the stress-displacement curve showed that a constant minimum shear stress had been reached. In all cases, shearing was continued for at least 0.3 inches and for no more than 0.5 inches (the maximum permissible travel of the shear box).

A total of 13 drained direct shear tests were performed on Rancho Solano Clay #1. Specimens were tested at four initial normal stresses: 7.9 psi, 14.5 psi, 28.8 psi, and 50.4 psi. Data sheets for each direct shear test are given in Appendix B. A typical friction ratio vs. displacement curve for Rancho Solano Clay #1 is shown in Figure 5-5. Friction ratios in the strain-controlled direct shear device were calculated using the following equation:

$$\text{Friction Ratio} = \frac{\text{Corrected Shear Stress}}{\text{Corrected Normal Stress}} = \frac{\text{Shear Force}}{\text{Normal Force}} \qquad (5\text{-}2)$$

This soil typically exhibits a small peak in shear resistance, possibly due to a “healing” effect on the shear plane. The shear resistance then drops to a nearly constant value, which can be considered the residual strength for the soil. A gradual increase in shear strength is often observed as the specimen is sheared to larger displacements, as shown in Figure 5-5. This “saddle” shape has been observed by other researchers testing clays in the direct shear device (Bishop et al., 1971), and is thought to be caused by the combined effects of extrusion and machine friction.

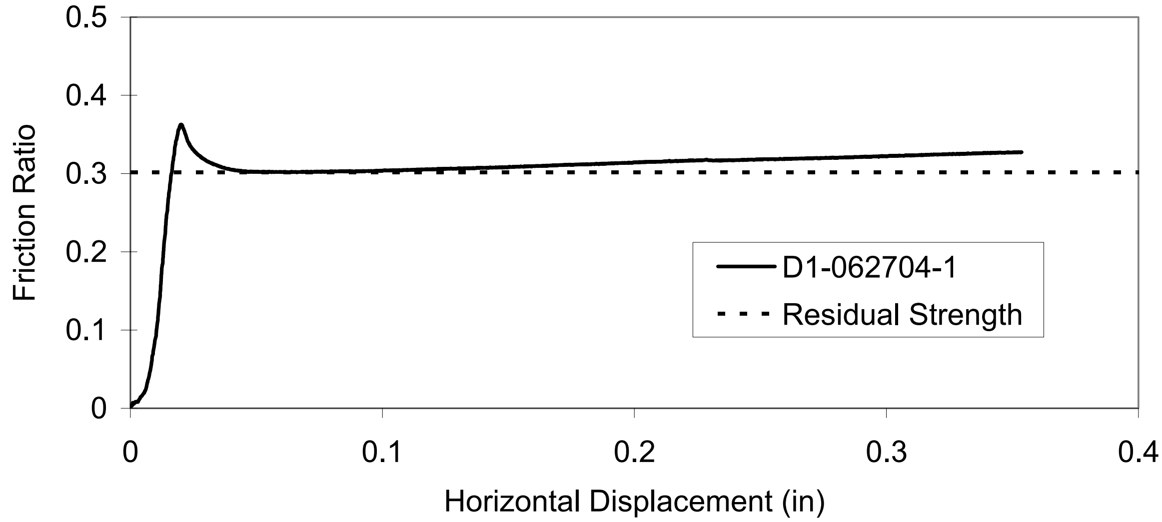


Figure 5-5. Friction ratio vs. displacement for direct shear test D1-062704-1.

Statistical analysis results of the measured residual shear stresses are given in Table 5-1. Statistical analysis results of the measured secant residual friction angles are given in Table 5-2. Secant residual friction angles were calculated using the following formula, which assumes that there is no residual cohesion:

$$\text{Secant Residual Friction Angle} = \tan^{-1}(\text{Residual Friction Ratio}) \qquad (5\text{-}3)$$

The standard deviations of the residual shear stresses and the secant residual friction angles were calculated using the nonbiased method, given by the following formula:

$$\text{Standard Deviation} = \sqrt{\frac{n\sum x^2 - (\sum x)^2}{n(n-1)}} \qquad (5\text{-}4)$$

where: x = sample value, and
n = total number of samples.

Comparisons between the average residual shear stresses and secant residual friction angles measured in the Bromhead ring shear device and the direct shear device are given in

Figure 5-6 and Figure 5-7. The bands surrounding each value of average secant friction angle in Figure 5-7 are the minimum and maximum secant residual friction angles measured at that normal stress.

Table 5-1: Residual Shear Stresses Measured in Direct Shear Tests on Rancho Solano Clay #1

Initial Normal Stress (psi)	Number of Tests Performed	Average Residual Normal Stress (psi)	Average Residual Shear Stress (psi)	Standard Deviation of Measured Residual Shear Stress (psi)	Minimum Measured Residual Shear Stress (psi)	Maximum Measured Residual Shear Stress (psi)
7.9	4	8.4	2.9	0.32	2.5	3.2
14.5	4	14.8	4.7	0.26	4.5	5.0
28.8	3	29.6	8.7	0.33	8.4	9.0
50.4	2	52.6	15.5	2.51	13.7	17.3

Table 5-2: Secant Residual Friction Angles Measured in Direct Shear Tests on Rancho Solano Clay #1

Initial Normal Stress (atm)	Number of Tests Performed	Normal Stress at Failure (atm)	Average Secant Residual Friction Angle (degrees)	Standard Deviation of Measured Secant Residual Friction Angle (degrees)	Minimum Measured Secant Residual Friction Angle (degrees)	Maximum Measured Secant Residual Friction Angle (degrees)
0.5	4	0.6	18.7	1.1	17.2	19.6
1.0	4	1.0	17.6	0.8	16.8	18.6
2.0	3	2.0	16.4	0.8	15.8	17.2
3.4	2	3.6	16.4	2.0	15.0	17.7

As shown in Figure 5-6 and Figure 5-7, good agreement was obtained between the residual strengths measured in the Bromhead ring shear device and the direct shear device. This provides experimental validation for use of the wet polishing method with Rancho Solano Clay #1.

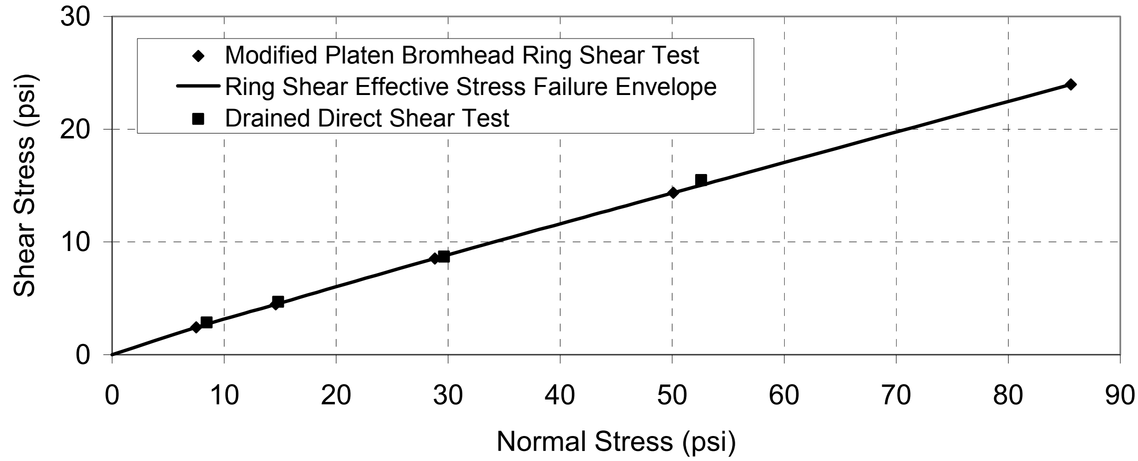


Figure 5-6. Comparison between Bromhead ring shear and direct shear test results for Rancho Solano Clay #1.

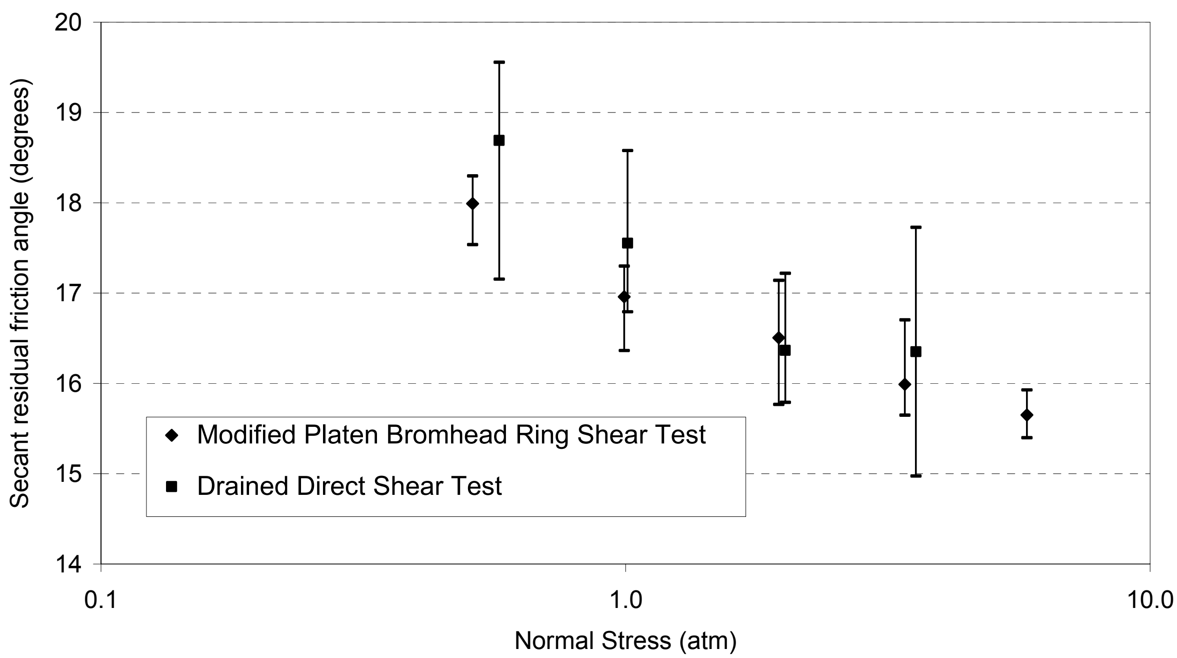


Figure 5-7. Comparison between Bromhead ring shear and direct shear test results for Rancho Solano Clay #1.

Drained Direct Shear Testing of Rancho Solano Clay #2

Direct shear tests were also used to measure the drained residual shear strength of Rancho Solano Clay #2. Specimens were prepared using the same "wet polish" method that had worked well for Rancho Solano Clay #1. The appearance of the Rancho Solano Clay #2 failure plane after wet polishing was indistinguishable from the Rancho Solano Clay #1 wet polished failure plane.

Two tests were performed, at an initial normal stress of 10.1 psi and a displacement rate of 0.000123 in/min. Data sheets for these direct shear tests are given in Appendix B. The friction ratio vs. displacement curves for these tests are shown in Figure 5-8. Figure 5-8 also shows the range of peak and residual friction ratios measured for Rancho Solano Clay #2 in the Bromhead ring shear device.

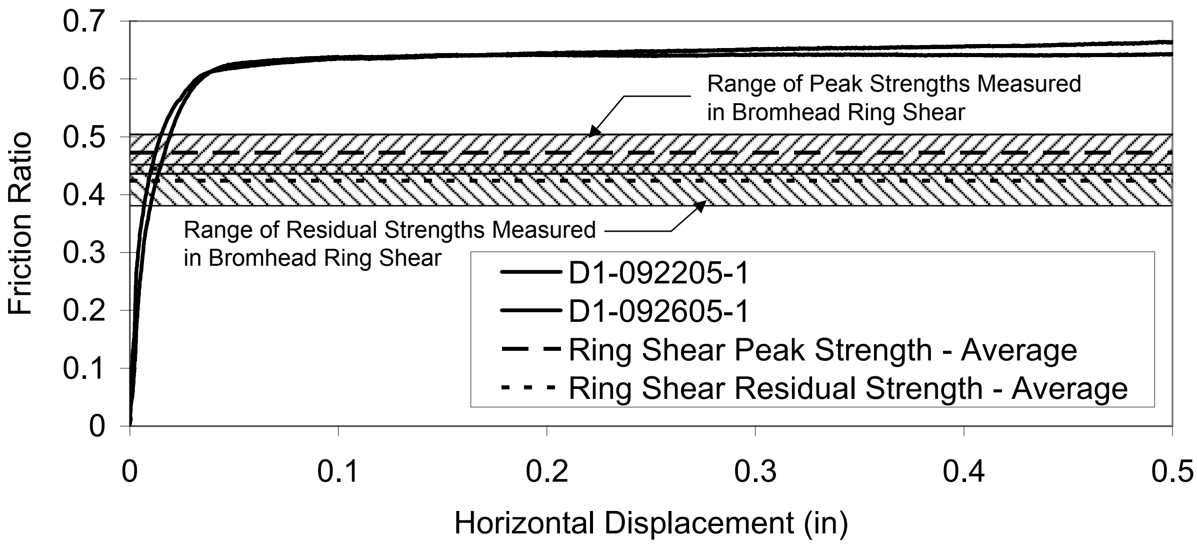


Figure 5-8. Comparison between Bromhead ring shear and "wet polish" direct shear test results for Rancho Solano Clay #2.

As shown in Figure 5-8, the shape of a typical friction ratio vs. displacement curve for Rancho Solano Clay #2 is significantly different than the curve for Rancho Solano Clay #1 (shown in Figure 5-5). Even more significant is the fact that the measured residual friction angle does not agree with the residual strength from the Bromhead ring shear device. The magnitude of this difference is quite large: 32.5° for the direct shear tests, as compared with 22.8° for the ring shear tests.

It was hypothesized that the use of a wet polishing method might have stripped fine particles from the pre-formed shearing plane in Rancho Solano Clay #2, effectively changing the grain size distribution at the shear interface. Such a change would alter the shear behavior of the soil, causing it to behave more like a silt or fine sand when sheared. This could explain why the residual friction angles are unusually high, and why the curve is shaped differently than what was observed for Rancho Solano Clay #1. It is not clear why wet polishing might have had this effect on Rancho Solano Clay #2, and why it did not have a similar effect on Rancho Solano Clay #1.

To explore this hypothesis, a series of direct shear tests were conducted on Rancho Solano Clay #2, using two different "dry" polishing methods. Using these methods, direct shear specimens were consolidated and wire-cut using the same approach that was used for the "wet" polish tests. The wire-cut test specimens were then polished on dry Teflon and dry glass surfaces, to orient the clay particles in the direction of shear.

For the dry Teflon polish method, a specimen half was polished by shearing it along the entire length of a dry 24-inch Teflon sheet under moderate hand pressure. A total of ten passes along the Teflon sheet were performed for each half of the test specimen. Figure 5-9 shows the dry Teflon polishing process, and the resulting slickensided failure plane.

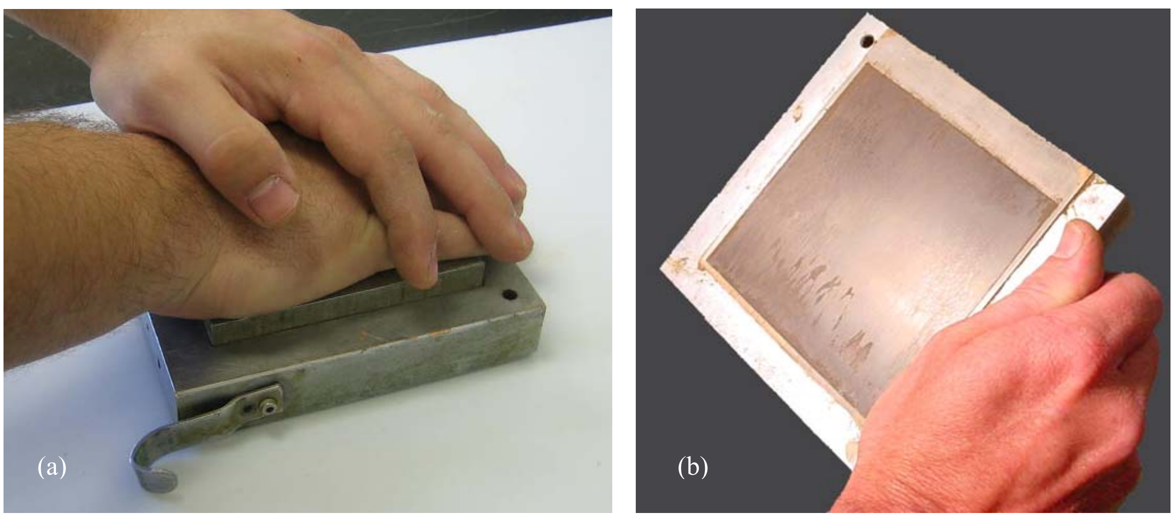


Figure 5-9. The dry Teflon polishing process; (a) rubbing the cut plane on dry Teflon to form slickensides, (b) the slickensided failure plane.

For the dry glass polish method, a specimen half was polished by shearing it along the entire length of a dry 12-inch frosted glass plate under moderate hand pressure. A total of ten passes along the glass were performed for each half of the test specimen. Figure 5-10 shows the dry glass polishing process, and the resulting slickensided failure plane. Note that the glass-polished specimen does not appear as slickensided as the specimen that was prepared using the dry Teflon polishing process.

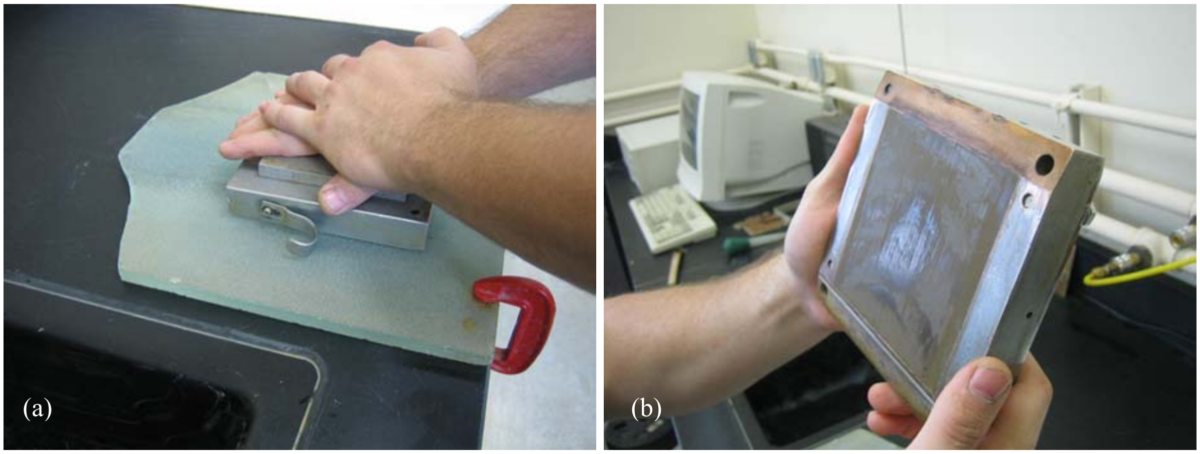


Figure 5-10. The dry glass polishing process; (a) rubbing the cut plane on dry glass to form slickensides, (b) the slickensided failure plane.

Dry polish direct shear tests were performed at an initial normal stress of 10.1 psi and a displacement rate of 0.000123 in/min. Data sheets for these direct shear tests are given in Appendix B. The friction ratio vs. displacement curves for these tests are shown in Figure 5-11. Figure 5-11 also shows the friction ratio that corresponds to the residual shear strength measured in the Bromhead ring shear device.

As shown in Figure 5-11, the dry polish method yields residual strengths for Rancho Solano Clay #2 that are lower than those measured in the Bromhead ring shear device – 11.8º to 12.3º for the direct shear tests, as compared with 22.8º for the ring shear tests. The shape of the friction ratio vs. displacement curves is more consistent with what was observed for Rancho Solano Clay #1 (shown in Figure 5-5). This supports the hypothesis that the wet polish method may have changed the grain size distribution on the shearing plane for Rancho Solano Clay #2. It is not clear why the residual strengths from dry polishing are so much lower than the ring shear residual strengths.

Additionally, as shown in Figure 5-11, the increase in strength that occurs as the specimen is sheared to large displacements is more pronounced for the specimen that was dry polished on Teflon than for the specimen that was dry polished on glass. This increase in strength is believed to be a testing artifact that was caused by slight misalignment of the preformed failure plane with the gap between the two halves of the direct shear box. The difficulty of aligning preformed shear planes in the direct shear device is consistent with what was observed by Skempton and Petley (1967). Good agreement was observed between the residual strengths measured in dry polish tests on Teflon and glass, despite the difference in behavior at large shear displacements.

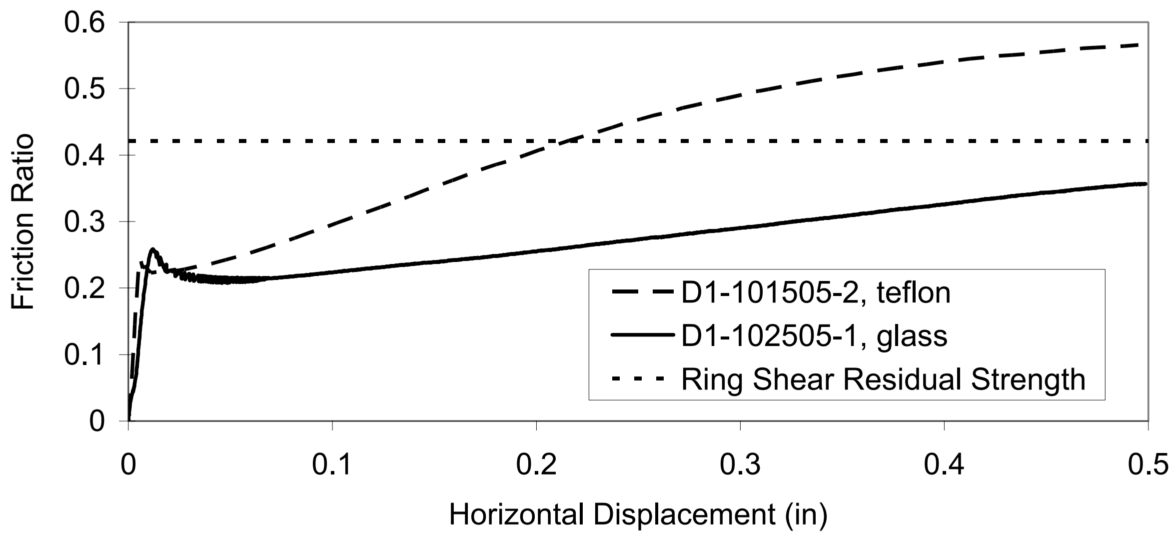


Figure 5-11. Comparison between Bromhead ring shear and "dry polish" direct shear test results for Rancho Solano Clay #2.

For Rancho Solano Clay #2, neither wet nor dry polishing techniques gave direct shear test results that agreed with the residual strengths measured in the Bromhead ring shear device. This result is unsatisfactory, and further research is necessary to identify why the direct shear test results deviated so significantly from the ring shear test results. Until the reason for this deviation is more clearly identified, the use of artificially prepared slickensides is not recommended for use in geotechnical engineering practice.

Drained Direct Shear Testing of San Francisco Bay Mud

Direct shear tests were also used to measure the drained residual shear strength of San Francisco Bay Mud. Specimens were prepared using the glass "wet polish" method and the Teflon and glass "dry polish" methods that were used to test Rancho Solano Clay #2. Figure 5-12 shows the appearance of the polished failure planes for three different test specimens.

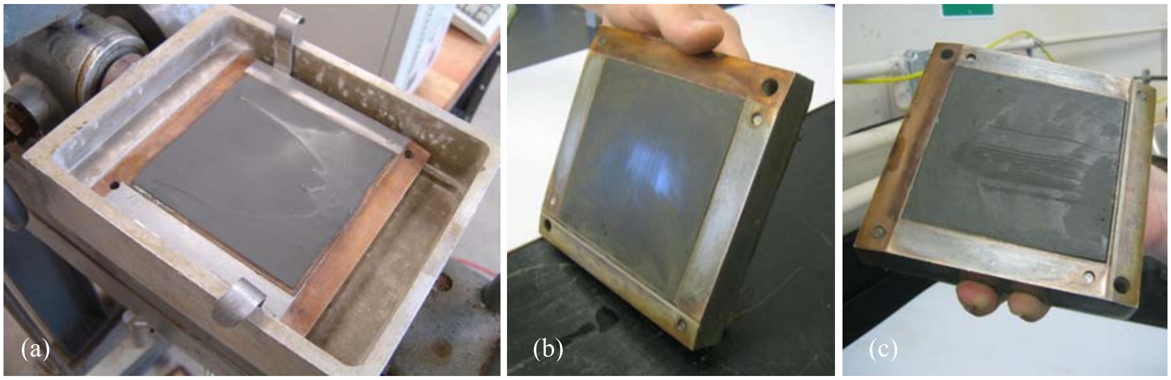


Figure 5-12. Appearance of slickensided failure planes in San Francisco Bay Mud after: (a) wet polishing on glass, (b) dry polishing on Teflon, and (c) dry polishing on glass.

Three direct shear tests were performed, at an initial normal stress of 14.9 psi and a displacement rate of 0.000123 in/min. Data sheets for these direct shear tests are given in Appendix B. The friction ratio vs. displacement curve for the "wet polish" test is shown in Figure 5-13. The friction ratio vs. displacement curves for the two "dry polish" tests are shown in Figure 5-14. Figures 5-13 and 5-14 also show the friction ratio that corresponds to the residual shear strength measured in the Bromhead ring shear device.

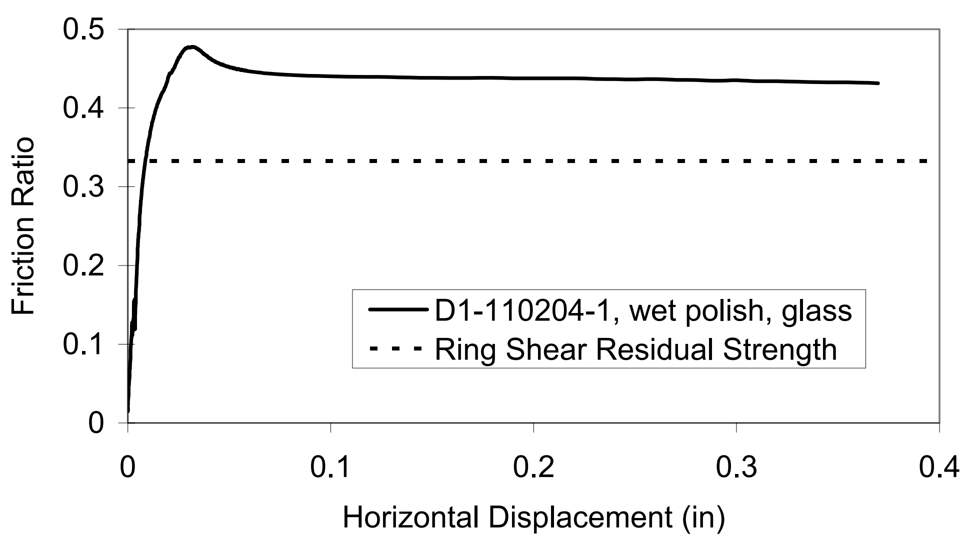


Figure 5-13. "Wet polish" direct shear testing on San Francisco Bay Mud.

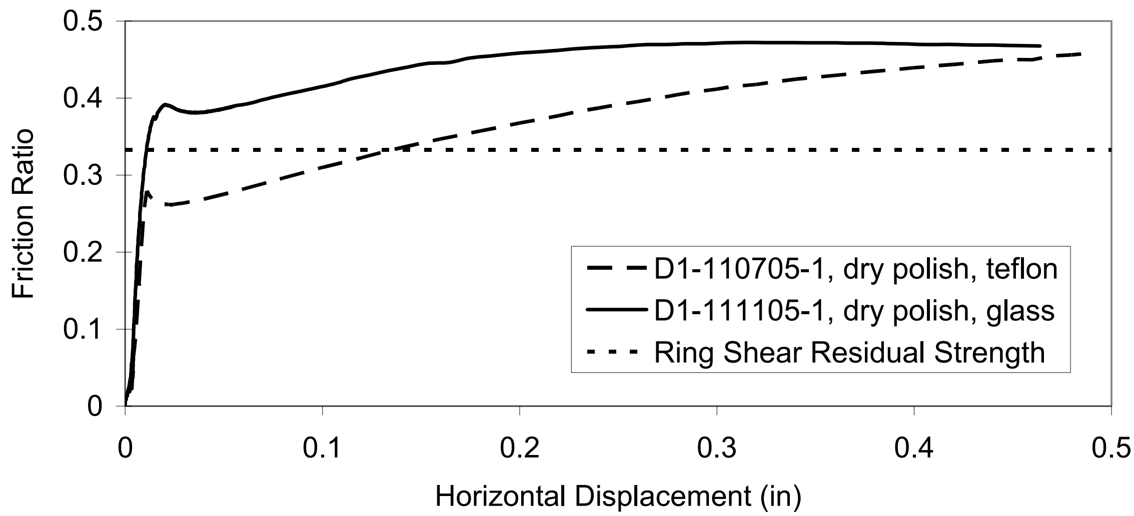


Figure 5-14. "Dry polish" direct shear testing on San Francisco Bay Mud.

As shown in Figure 5-13, the residual strength measured for the glass "wet polish" direct shear tests is higher than the residual strength measured in the Bromhead ring shear device – 23.3º for the direct shear tests, as compared with 18.4º for the ring shear tests. It is believed that this increased strength is due to a change in the grain size distribution of the soil along the shear interface, brought about by the use of a wet preparation process that strips clay fines from the shear interface during polishing. This mechanism is the same as the one used to explain the increased strengths measured for wet polished Rancho Solano Clay #2. The increase in strength of the wet polished San Francisco Bay Mud over the ring shear residual strengths is not as pronounced as what was observed for Rancho Solano Clay #2.

As shown in Figure 5-14, the Teflon dry polish method yields residual strengths for San Francisco Bay Mud that are lower than those measured in the Bromhead ring shear device – 14.7º for the direct shear tests, as compared with 18.4º for the ring shear tests. As was observed in Teflon dry polish tests on Rancho Solano Clay #2, the cause of this low strength value is unknown. It is believed that the Teflon dry polishing process somehow fundamentally alters the nature of the shear interface, either by causing changes in the physio-chemical interaction between clay particles or by increasing the amount of clay particles along the shearing interface.

The glass dry polish method yields residual strengths for San Francisco Bay Mud that are higher than those measured in the Bromhead ring shear device – 20.9º for the direct shear tests, as compared with 18.4º for the ring shear tests. This increased value of strength was likely caused by adhesion of the Bay Mud particles to the glass polishing plate, which resulted in an extremely poor quality polish along the prepared shearing plane. Visually, glass dry-polished San Francisco Bay Mud specimens appear the least slickensided of all the soil and polishing interface combinations that were tested.

<u>Triaxial Testing of Preformed Slickensided Surfaces</u>

In addition to the drained direct shear tests described in the previous sections, a series of triaxial tests were also performed to measure the drained residual shear strength along pre-formed slickensided failure surfaces. These tests were performed at Virginia Tech by Dr. Binod Tiwari, who has significant experience with soil laboratory testing. Dr. Tiwari has encountered significant obstacles during the triaxial testing program, including:

- Difficulties with the effect of end platen restraint on specimens that fail along a well-defined failure plane,

- Uncertainties involving the appropriate area correction and membrane correction to use when reducing the triaxial data, and

- Long test times for consolidated-drained triaxial tests, which has tied up equipment and made it difficult to run the desired number of triaxial tests in a timely fashion.

As a result of these challenges and uncertainties, it has proven far more difficult to use triaxial tests than direct shear tests to measure the shear strength along pre-formed slickensided discontinuities. Because of the difficulties encountered, the triaxial test is not recommended for future testing of this type.

Because of the uncertainty surrounding the triaxial test results at this time, useful conclusions cannot be drawn from the triaxial test data regarding the residual strength behavior of pre-formed slickensided surfaces. Consequently, the results from Dr. Tiwari's triaxial testing program are not included in this report. Dr. Tiwari plans to continue triaxial testing.

Discussion of Experience with Laboratory Testing of Preformed Slickensided Surfaces

At the beginning of this research project, it was envisioned that is would be a straightforward process to measure the residual strengths along preformed slickensided surfaces using traditional direct shear and triaxial testing equipment. This assumption was based on previous research performed by Skempton (1964), Chandler (1966), and Skempton and Petley (1967). The potential payoff to this approach was that it would allow geotechnical practitioners to simulate earthquake loading along slickensided surfaces using simple test equipment available in most geotechnical laboratories. However, in order to measure accurate dynamic strengths along preformed slickensided surfaces, it is essential to first establish a laboratory method for preparing slickensided surfaces that behave like those formed in the field.

As is evident from the discussion in the previous sections, it was found to be significantly more difficult than anticipated to prepare slickensided surfaces that exhibited the expected drained residual strength behavior. Table 5-3 shows how the results from the drained direct shear testing program compare with the residual strength values measured in the Bromhead ring shear device.

Table 5-3: Comparison of Drained Direct Shear Test Results with Bromhead Ring Shear Test Results for Different Polishing Methods

Soil	Wet polishing on glass	Dry polishing on Teflon	Dry polishing on glass
Rancho Solano Clay #1	Excellent agreement between Direct Shear and Bromhead Ring Shear	Not performed	Not performed
Rancho Solano Clay #2	Direct Shear 43% Higher than Bromhead Ring Shear	Direct Shear 46% Lower than Bromhead Ring Shear	Direct Shear 48% Lower than Bromhead Ring Shear
San Francisco Bay Mud	Direct Shear 27% Higher than Bromhead Ring Shear	Direct Shear 20% Lower than Bromhead Ring Shear	Direct Shear 14% Higher than Bromhead Ring Shear

As shown in Table 5-3, the effectiveness of a given polishing technique varied greatly for the three soils tested. Consistency was not obtained between different soils for any of the polishing methods explored in this study. The polishing approach that worked well for Rancho Solano Clay #1 did not work at all for Rancho Solano Clay #2, despite the fact that the soils are from the same area and are quite similar. Because the test results were so sensitive to soil type and to the polishing process used, a single method for preparing slickensided surfaces in the laboratory could not be identified. Consequently, the use of artificially prepared slickensides is not recommended for use in geotechnical engineering practice.

For research purposes, it is possible to form slickensided surfaces in the laboratory that behave similarly to those created by soil-on-soil shearing processes (as illustrated by the Rancho Solano Clay #1 test results). When a polishing process is used to prepare slickensides, the effectiveness of the preparation method should be confirmed by comparison with Bromhead ring shear strength test results. It is recommended that a number of tests be performed for this purpose, to explore the sensitivity of the measured strengths to the preparation method for the soil being studied.

As shown in Table 5-3, Rancho Solano Clay #1 was the only soil that gave drained strengths that compared consistently well with those measured in the Bromhead ring shear device. As a result, fast direct shear and cyclic direct shear tests were only performed on wet polished Rancho Solano Clay #1 specimens.

Fast Direct Shear Testing of Rancho Solano Clay #1

Fast direct shear tests were performed on Rancho Solano Clay #1 test specimens to explore the effect of loading rate on the shear behavior along slickensided discontinuities. Test specimens were prepared using the "wet polish" preparation method, which forms slickensides that behave similarly to those formed by soil-on-soil shear.

Four strain-controlled direct shear tests were performed at a displacement rate of 0.048 in/min, which is the maximum loading rate that can be applied in the Wykeham-Farrance direct shear device. Two test specimens were tested at a normal stress of 14.5 psi and two were tested at 28.8 psi. Data sheets for these direct shear tests are given in Appendix

B. The friction ratio vs. displacement curves for the 14.5 psi normal stress tests are shown in Figure 5-15. The friction ratio vs. displacement curves for the 28.8 psi normal stress tests are shown in Figure 5-16. For comparison purposes, Figures 5-15 and 5-16 also show friction ratio curves measured in slow direct shear tests.

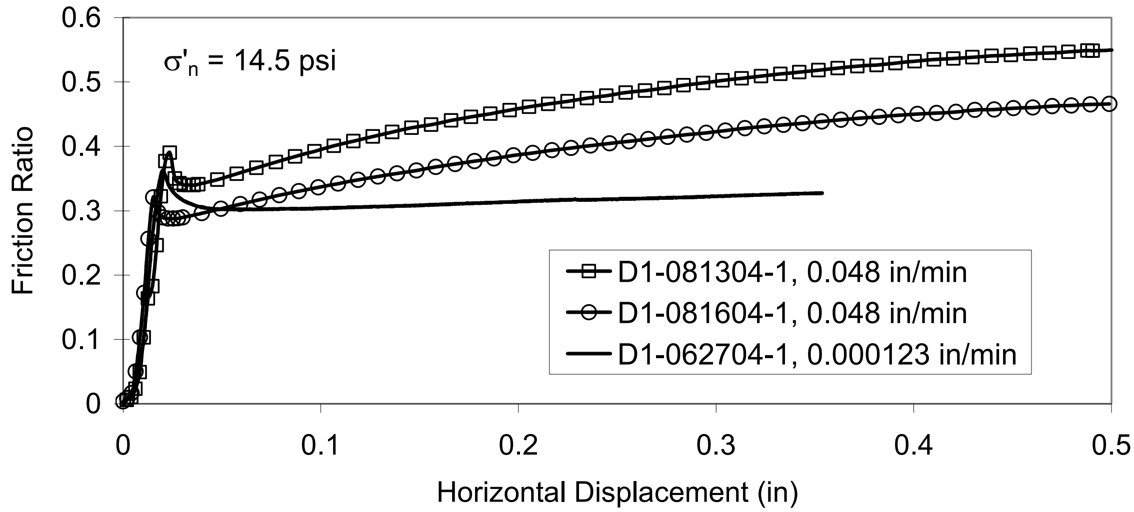


Figure 5-15. Comparison between fast and slow direct shear tests conducted on Rancho Solano Clay #1 at a normal stress of 14.5 psi.

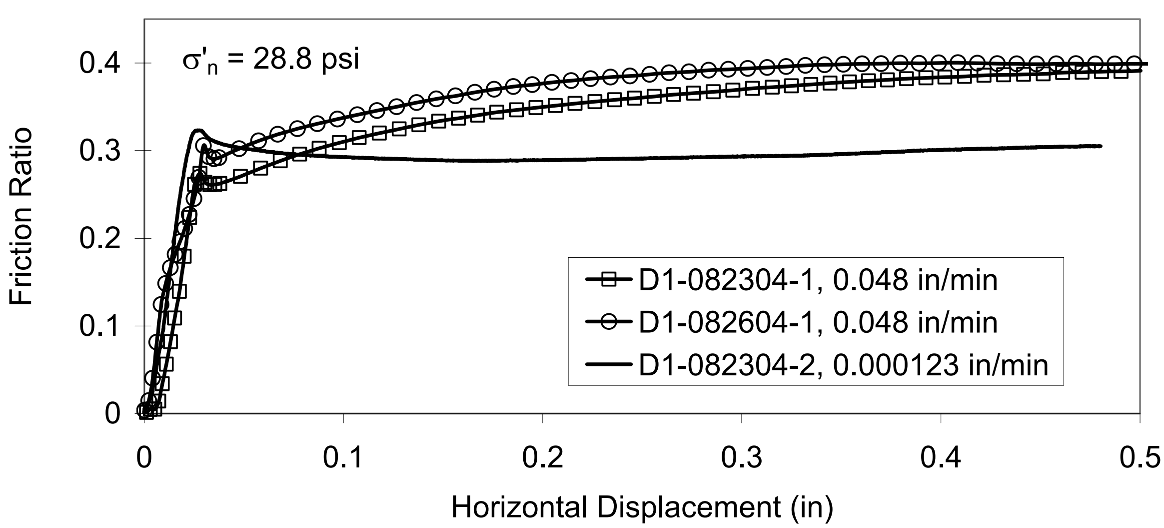


Figure 5-16. Comparison between fast and slow direct shear tests conducted on Rancho Solano Clay #1 at a normal stress of 28.8 psi.

As shown in Figures 5-15 and 5-16, fast direct shear tests exhibit different behavior than drained direct shear tests. Initially, there is an increase in shear resistance, possibly due to a “healing” effect on the shear plane. The shear resistance then drops to a post-peak minimum value, which can be considered the “fast residual strength” for the soil. Upon additional shearing, there is a significant increase in shear resistance that continues throughout the remainder of the test.

The initial peak shearing resistances from the fast direct shear tests were equal to or slightly lower than the peak shear resistances observed in the drained direct shear tests. The fast residual strengths varied significantly, coming in both lower and higher than the residual strengths measured in the drained direct shear tests. Consequently, the results do not show conclusively how the residual strength changed as the loading rate was increased from 0.000123 in/min to 0.048 in/min.

The increase in mobilized shear resistance that occurred after the fast residual strength is likely due to a combination of soil strengthening and machine effects in the direct shear device. Skempton (1985) reported an increase in residual strength of 2.5% per log cycle increase in strain rate, so it is reasonable to expect an approximately 6% increase in shear resistance as the displacement rate is increased from 0.000123 in/min to 0.048 in/min. Additionally, it is possible that the increased loading rate led to disturbance of the smoothly polished failure plane, which caused a corresponding increase in measured shear resistance that became more pronounced with increased displacement (Lemos et al., 1985; Skempton, 1985; Tika et al., 1996). This effect is likely minimal however, as the aforementioned researchers have reported that this increase typically does not become pronounced until displacement rates on the order of 0.4 in/min to 4 in/min.

It is likely that a large portion of the increased shearing resistance that occurs past the fast residual strength is due to machine effects in the direct shear device. Frictional forces between the shear boxes and soil extrusion caused saddle-shaped curves in the slow direct shear tests, and it is likely that their effect was more pronounced during the fast shear tests. Additionally, as shown in Figure 5-11, slight misalignments of the preformed shearing plane in the direct shear box can cause a significant increase in the mobilized shear resistance at large displacements.

Cyclic Direct Shear Testing of Rancho Solano Clay #1

Cyclic direct shear tests were performed on Rancho Solano Clay #1 test specimens to examine the behavior of slickensided rupture surfaces under cyclic loading conditions. Test specimens were prepared using the "wet polish" preparation method, which forms slickensides that behave similarly to those formed by soil-on-soil shear.

Eight stress-controlled cyclic loading direct shear tests were performed in the cyclic direct shear device at Virginia Tech. Specimens were tested at a normal stress of 14.9 psi and a cyclic load frequency of 0.5 Hz. Data sheets for the cyclic direct shear tests are given in Appendix B.

Prior to application of the cyclic loading, specimens were subjected to a static shear stress in the direct shear device. This allowed cyclic loading to be applied around a sustained static stress, which mimics the state of stress mobilization that exists in a slope in the field. A target static shear stress ratio of 0.2 was used for these tests, which corresponds to approximately 60% of the drained residual shear strength. Table 5-4 lists the applied static load for each test and the resulting displacement.

Table 5-4: Applied Static Load and Resulting Displacement for the Cyclic Direct Shear Tests

Test Number	$\tau_{static}/\sigma'_{fc}$	Displacement Upon Load Application
D2-040805-1	0.21	0.0015
D2-042905-1	0.22	0.001
D2-062105-1	0.19	0.0017
D2-062705-1	0.20	0.0018
D2-062805-1	0.21	0.001
D2-090105-1	0.19	0.001
D2-092705-1	0.24	0.0026
D2-092905-1	0.17	0.0011

As shown in Table 5-4, only a very small amount of displacement occurred upon application of the static load.

After each specimen had come to equilibrium under the applied static load, cyclic loading was applied. Each specimen was subjected to 500 constant-amplitude sinusoidal stress pulses, with a cyclic load frequency of 0.5 Hz. A plot of applied shear stress vs. time for cyclic direct shear test D2-090105-1 is shown in Figure 5-17. A close-up view of the applied shear stress pulses for test D2-090105-1 is shown in Figure 5-18. The resulting horizontal and vertical displacements for test D2-090105-1 are shown in Figure 5-19.

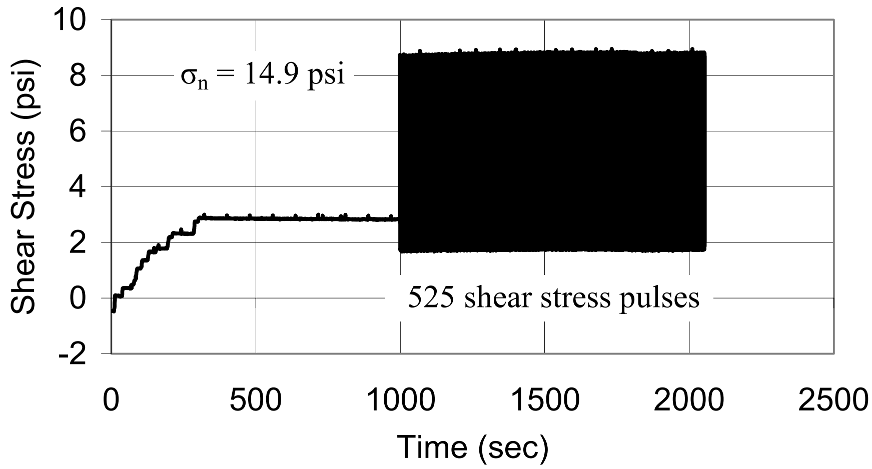


Figure 5-17. Applied shear stress vs. time for cyclic direct shear test D2-090105-1.

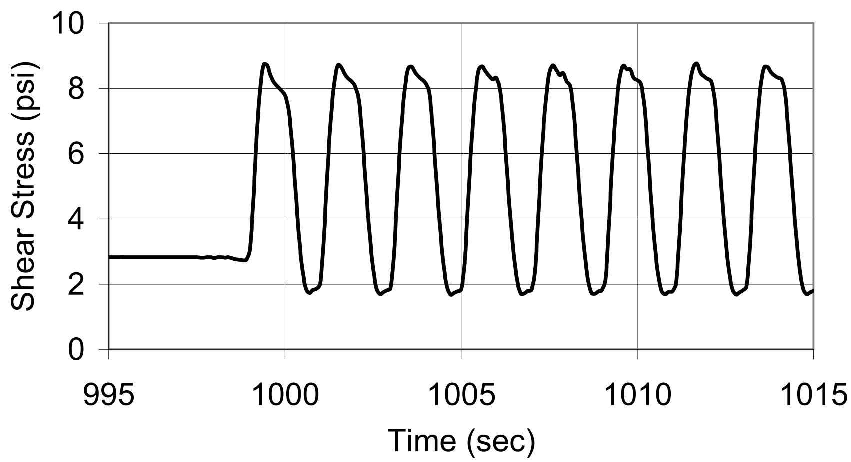


Figure 5-18. Shape of shear stress load pulses for test D2-090105-1.

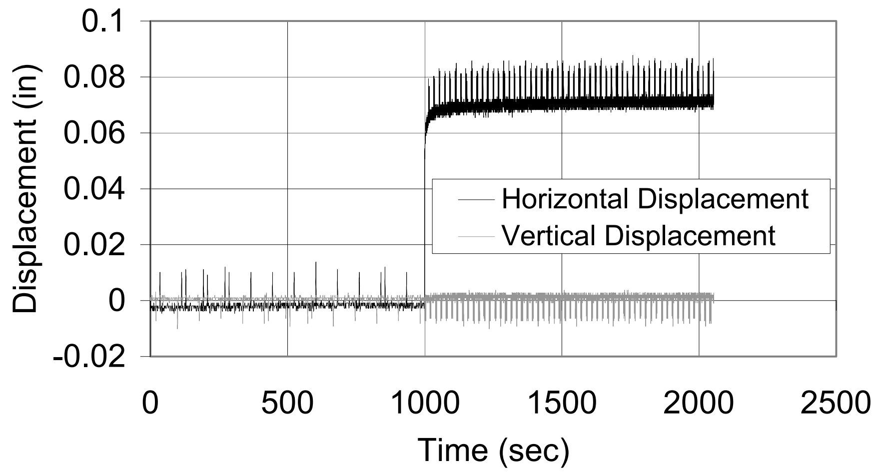


Figure 5-19. Measured horizontal and vertical displacement for test D2-090105-1.

As shown in Figures 5-17 and 5-18, the applied cyclic load was not always symmetric about the static load. This was due to limitations of the stress application system, which made it difficult to apply the desired loading to the specimen consistently. These device stress-control problems were not addressed because the solution was costly, and because the effect of the unsymmetric loading on the test results was believed to be second-order.

As shown in Figure 5-19, there was significant electrical noise in the recorded horizontal and vertical LVDT data. This noise was introduced by the data acquisition system, appearing as periodic spikes in the recorded displacement values. Because caution was taken when analyzing and interpreting test results, this electrical noise is believed to have no effect on the interpreted test results. The electrical noise problems with the data acquisition system were not addressed because the solution was costly and because it was still possible to interpret the tests results with good accuracy.

Data from the cyclic direct shear tests were analyzed by plotting the applied shear stress ratio vs. displacement. A plot of shear stress ratio vs. displacement for test D2-090105-1 is shown in Figure 5-20. Shear stress ratios in the cyclic direct shear device were calculated using the following equation:

$$\text{Shear Stress Ratio} = \frac{\text{Shear Stress on the Slip Plane}}{\text{Initial Effective Normal Stress on the Slip Plane}} \qquad (5\text{-}5)$$

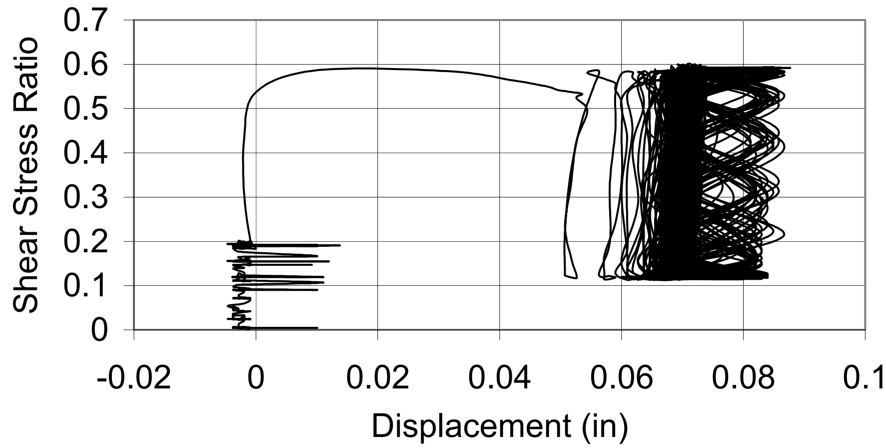


Figure 5-20. Shear stress ratio vs. displacement for test D2-090105-1.

As shown in Figure 5-20, most of the displacement that occurred during test D2-090105-1 happened in the first few pulses. As shear displacement occurred on the slickensided plane, the specimen exhibited displacement hardening, with smaller and smaller amounts of displacement observed for each consecutive pulse. (The apparent increases in horizontal displacement of approximately 0.012 inches during the first cycle and any pulses observed beyond 0.073 inches are due to electrical noise.) Table 5-5 lists the cyclic shear stress ratio and the cumulative displacements recorded as a function of the number of applied stress cycles for each test. Figure 5-21 is a plot of the values listed in Table 5-5, with a hatched zone drawn between the upper and lower bounds of the measured data. Figure 5-21 also plots the stress ratio vs. displacement from test D1-062704-1, a drained direct shear test.

Table 5-5: Applied Shear Stress Ratio and Resulting Displacement During Cyclic Loading

Test Number	τ_{peak}/σ'_{fc}	1 Cycle	2 Cycles	3 Cycles	5 Cycles	10 Cycles	100 Cycles	500 Cycles
D2-040805-1	0.52	0.0443	0.0489	0.0508	0.0526	0.0563	0.0646	0.0655
D2-042905-1	0.64	0.0378	0.0425	0.0452	0.0471	0.0489	0.0563	0.0591
D2-062105-1	0.30	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017	0.0017
D2-062705-1	0.31	0.0018	0.0018	0.0018	0.0018	0.0018	0.0018	0.0018
D2-062805-1	0.38	0.001	0.001	0.001	0.001	0.001	0.002	0.003
D2-090105-1	0.58	0.0507	0.0572	0.0609	0.0618	0.0655	0.0683	0.0701
D2-092705-1	0.63	0.0839	0.095	0.0996	0.1061	0.1116	0.1292	0.1347
D2-092905-1	0.66	0.1375	0.1625	0.1726	0.1818	0.1911	0.216	0.2243

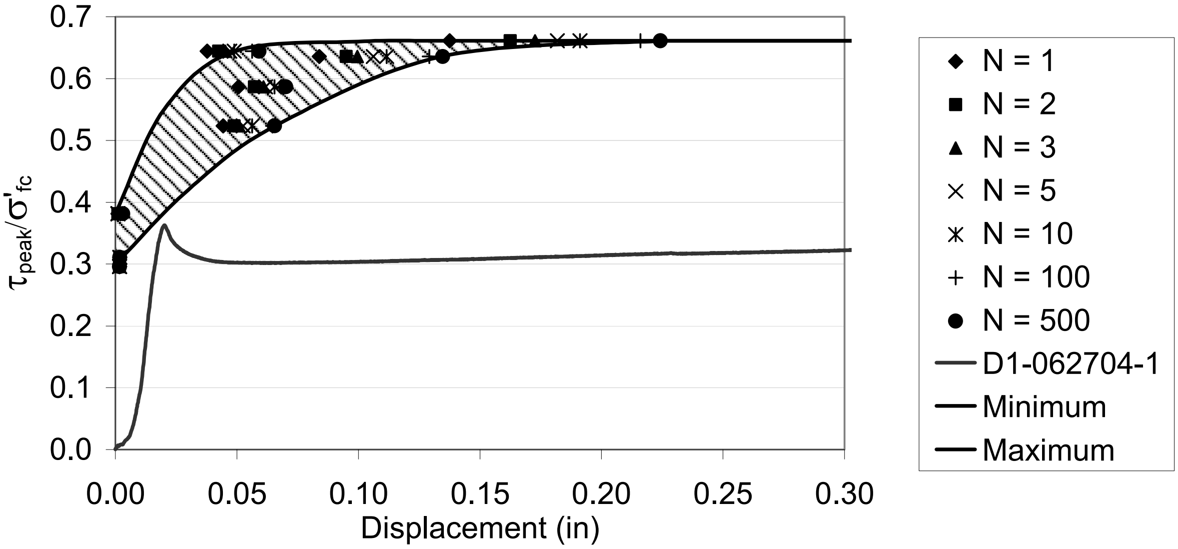


Figure 5-21. Approximate relationship between peak shear stress ratio and displacement for cyclic direct shear tests on Rancho Solano Clay #1.

From this data, it is clear that there is a relationship between the applied cyclic shear stress ratio and the resulting displacement during the test. When cyclic loads with a peak shear stress ratio less than 0.4 are applied, no displacement occurs. Some displacement occurs when cyclic loads with a peak shear stress ratio greater than 0.5 are applied and more significant displacements are observed at cyclic stress ratios above 0.6.

As shown in Figure 5-21, the relationship between peak shear stress ratio and displacement becomes asymptotic at a shear stress ratio of 0.66. At this value, large displacements are observed in the first few pulses, and displacement hardening is not effective at slowing the accumulation of displacement. From this data, a shear strength ratio of 0.66 was chosen as the cyclic strength for Rancho Solano Clay #1. This strength ratio corresponds to sinusoidal cyclic loading conditions imposed at a frequency of 0.5 Hz. This cyclic strength is 2.2 times higher than the drained shear strength mobilized along slickensided surfaces in Rancho Solano Clay #1 during slow loading. Consequently, it would be excessively conservative to use drained residual strengths for dynamic analyses of slickensided slopes in Rancho Solano Clay #1.

CHAPTER 6: CENTRIFUGE TESTING PROGRAM

The centrifuge tests described in this chapter were conducted to measure the slow residual strength and the cyclic shear strength along slickensided discontinuities in the Rancho Solano Clay and the San Francisco Bay Mud. Two centrifuge tests were performed at the University of California, Davis (UC Davis), using the 30-foot radius centrifuge located at the UC Davis Center for Geotechnical Modeling. This centrifuge is shown in Figure 6-1. Test CLM01 was performed using San Francisco Bay Mud obtained from Hamilton Air Force Base in California. Test CLM02 was performed using Rancho Solano Clay #2, which was obtained from the Rancho Solano residential development in Fairfield, California. These soils are described in more detail in Chapter 3.

Figure 6-1. 30-foot radius centrifuge at UC Davis.

The 30-foot radius centrifuge is currently part of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES) Program sponsored by the National Science Foundation. The centrifuge facilities and instrumentation systems used for the centrifuge tests are described in detail at the NEES Site Specifications Database website (http://www.nacse.org/neesSiteSpecs/do/siteSelection). Data reports and the raw data recorded during centrifuge tests CLM01 and CLM02 are available at the NEES Central website (https://central.nees.org/).

A number of significant problems were encountered during centrifuge test CLM01, which made interpretation of the data from this test nearly impossible. These problems included: significant soil erosion, which resulted in a large, unknown change in specimen area throughout the centrifuge test; ineffective soil polishing techniques, which resulted in a pre-sheared soil surface that was not at its residual condition; and physical and electronic problems with the embedded pore pressure transducers, which made it very difficult to evaluate the pore pressures that were measured during the test. The combined effect of these problems made it impossible to develop an accurate understanding of how the soil behaved during test CLM01. Consequently, the results from centrifuge test CLM01 are not presented in this report. For details regarding centrifuge test CLM01, please refer to the NEES Central website (https://central.nees.org/).

Despite the lack of useful data regarding soil behavior, the data from test CLM01 was useful in refining the design of the centrifuge model. Test CLM02 used a modified design that successfully addressed the problems encountered in test CLM01. Consequently, only the model construction methods and centrifuge test results from CLM02 are discussed in the following sections.

Overall Concept of Centrifuge Model Test CLM02

A sketch that captures the important elements of test CLM02 is shown in Figure 6-2. In the model, a heavily overconsolidated clay layer was confined between two rigid steel plates, which were fixed securely to the clay. The clay layer contained a preformed slickensided clay surface, along which shear displacement occurred during static and seismic loading. The clay/steel plate system was supported by an inclined base constructed of concrete. This system simulated the Newmark "sliding block" analogy commonly used in practice to predict displacements of slopes during earthquakes.

The system was instrumented to measure accelerations and displacements during seismic loading, as well as pore pressures that were generated within the model. Accelerometers were placed along the upper and lower steel plates, parallel and perpendicular to the direction of sliding. Relative displacements between the upper and the lower steel plates were measured using Linear Variable Differential Transformers (LVDTs) and Linear Potentiometers (LPs). Pore pressures were measured using pore pressure transducers (PPTs) embedded in the clay at locations that did not interfere with shearing along the slickensided plane.

Once the model was constructed and instrumented, it was spun-up in the centrifuge. The model was then saturated and allowed to come to pore pressure equilibrium. Once pore pressure equilibrium had been achieved, both fast and slow displacement-controlled loading tests were performed to measure shear resistance along the preformed slickensided surface. The hydraulic actuator and pulley system shown in Figure 6-2 were used to apply the loads. The static load on the specimen was then removed, and the specimen was subjected to a series of seismic loading events. Accelerations, pore pressures, and displacements were measured during the static and seismic loading events.

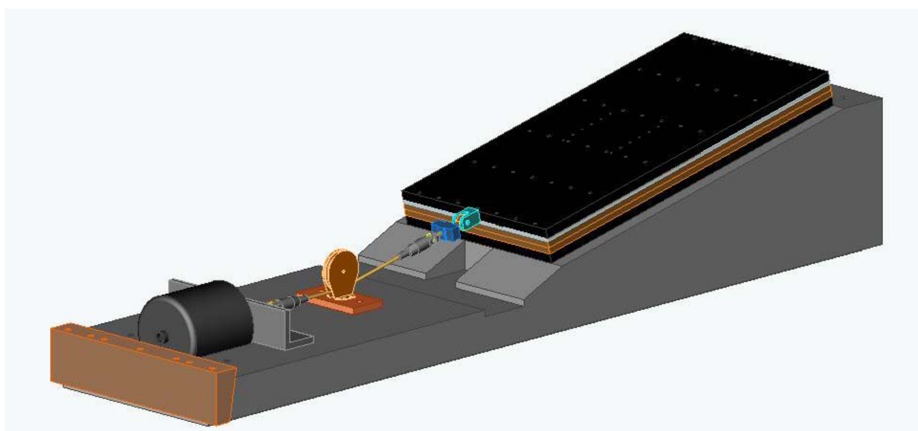


Figure 6-2. Centrifuge test specimen representing an element of soil on a slickensided rupture surface within a slope.

<u>Configuration of Centrifuge Model Test CLM02</u>

Test CLM02 was performed in a rigid-walled model container (RC 1), and consisted of two side-by-side "sliding block" clay models. The models were set at different slope inclinations, the flatter slope at an angle of 10.5 degrees and the steeper slope at an angle of 12 degrees, in order to produce two different sliding responses for each loading event. The slope angles used were selected to correspond to static factors of safety of 1.3 and 1.5, based on the residual friction angle that had been measured in the laboratory. The side-by-side layout used for test CLM02 is shown in Figure 6-3. Detailed shop drawings can be found in Appendix C.

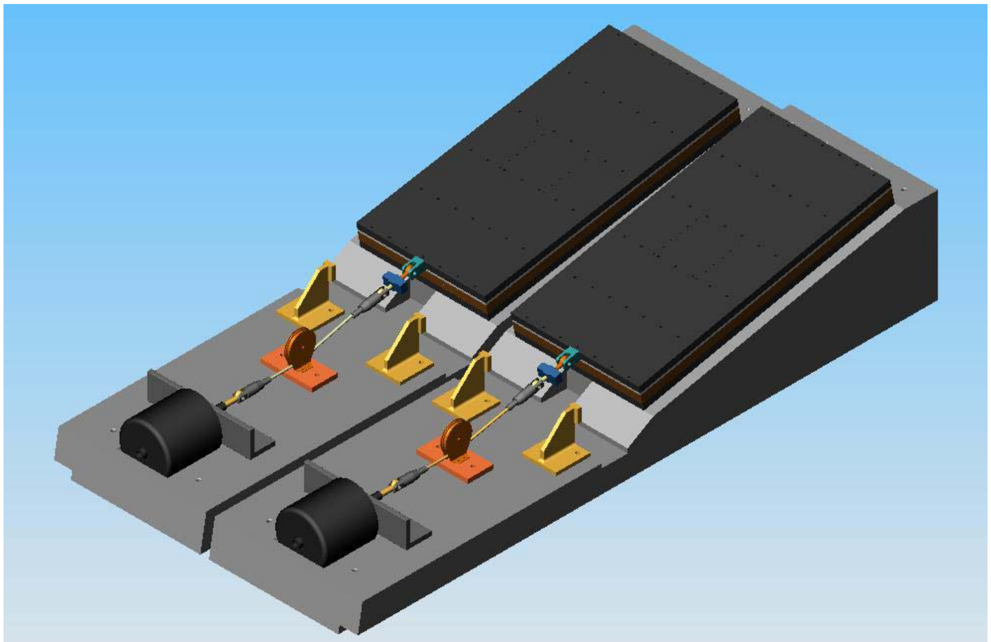


Figure 6-3. Model layout for centrifuge test CLM02.

The “sliding block” models shown in Figure 6-3 consisted of a 1 inch thick clay layer that was confined between two 3/4-inch thick steel plates. Deionized water was used as the pore fluid for the clay. The preformed slickensided surface was located at mid-height in the clay layer. The clay layer was singly-drained, with pore pressures being allowed to dissipate through a sheet of porous plastic between the upper steel plate and the clay. The layout is shown in Figures C-1 and C-2. A cross section that shows the clay “sliding block” system is shown in Figure C-3.

Scaling Laws

The scale factors shown in Table 6-1 can be used to convert the data from model units to prototype units. The scale factors given in Table 6-1 were calculated using the principles discussed in Chapter 2, and are based on an applied centrifugal acceleration of 45 g’s, which corresponds to a centrifuge rotational speed of 68.2 revolutions per minute.

Table 6-1: Scale Factors for Converting Model Data to Prototype Units (Kutter, 1992)

Quantity	Model Dimension / Prototype Dimension
Time (dynamic)	1/45
Displacement, Length	1/45
Acceleration, Gravity	45/1
Force	1/2025
Pressure, Stress	1/1
Time (diffusion)*	1/2025

*Note: The diffusion time scale factor depends on whether the diffusion coefficient (e.g. coeffiecient of consolidation) is scaled. For this test, since the same soil was used in the model as the prototype, use $1/N^2$.

At a centrifugal acceleration of 45 g's, the model corresponded to a 3.75 ft thick layer of clay, with a 2.8 ft thick steel plate bonded to its upper surface, with a 0.9 ft thick plastic drainage layer in between. The overlying steel plate and plastic drain together correspond to approximately 15 ft of soil overburden in the prototype condition.

As discussed in Chapter 2, it is not possible to separate the assumption of rate dependent mechanical properties from the derivation of the centrifuge scale factors. Consequently, the centrifuge data that is presented in this report is given in model units.

Model Construction

The Reinforced Concrete Base Support System

The first step in the model construction process was to construct the two reinforced concrete bases that supported the model. These served as the foundation for the lower steel plates, providing both a bearing surface and a reaction for the steel plates and the actuator during the static and seismic loading events. Figure 6-4 shows the concrete bases prior to their installation in the container.

The base of the container was lined with a thin layer of coarse sand (Monterey #3 sand), which was used to transmit energy during seismic shaking from the base of the container to the concrete supports. The concrete bases were then placed side-by-side in the container, and all open spaces were back-filled with Monterey #3 sand, which was vibrated into place. Figure 6-5 shows the reinforced concrete supports prior to back-filling with the Monterey sand.

Figure 6-4. Concrete bases used to support Rancho Solano Clay test specimens.

Figure 6-5. Two concrete bases side-by-side in the rigid container.

<u>Hydraulic Actuator Static Loading System</u>

In order to measure the static shear resistance along the preformed slickensided surface, a hydraulic actuator system was developed to apply a downslope static load to the test specimens. Each actuator in the hydraulic system was able to apply 2000 pounds of force at two different displacement rates: a fast rate of approximately 0.05 in/min and a slow rate of approximately 0.0005 in/min. Using the remotely-operated hydraulic system, the actuators could be independently advanced or retracted at either displacement rate from the centrifuge control room. Figure 6-6 shows the hydraulic actuators and their control systems mounted on the concrete bases.

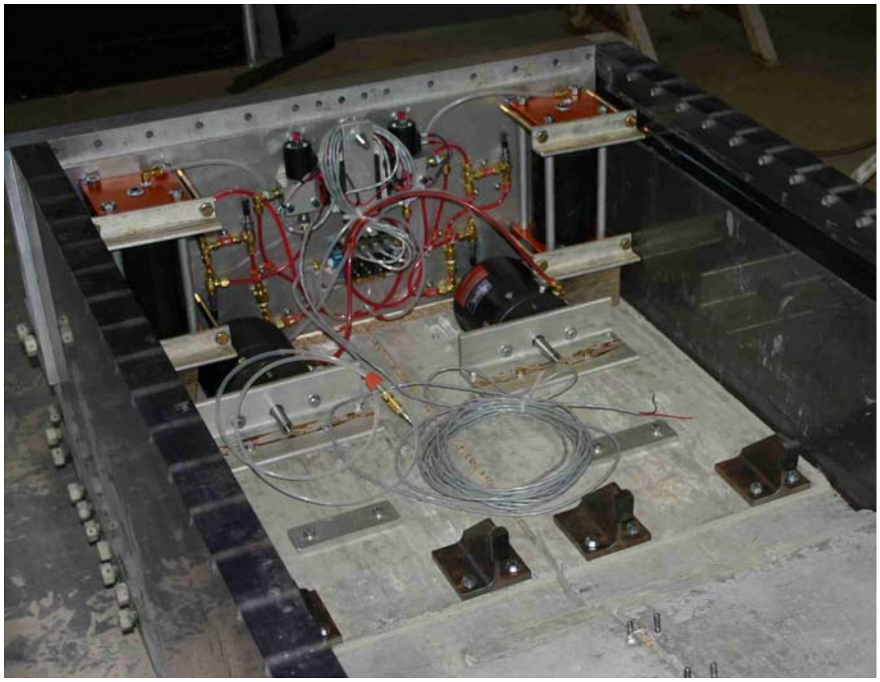


Figure 6-6. The hydraulic actuators.

Upper and Lower Steel Plates

At this point in the model construction process, the model container was ready to support the sliding block soil models. Each model consisted of a 1 inch thick clay layer confined between 3/4-inch thick steel plates. Detailed shop drawings and design specifications for the upper and lower steel plates are shown in Figures C-4 and C-5.

To ensure that sliding would occur within the clay layer, it was necessary to roughen the interface between the lower steel plate and the clay, and between the upper porous plastic and the clay. The lower steel surface was roughened by gluing sand grains to the steel plate. The porous plastic interface was roughened by machining a series of grooves into the plastic. Close-up views of the roughened surfaces are shown in Figure 6-7.

Soil Preparation

The soil used in the test was Rancho Solano Clay #2, which was obtained from Rancho Solano residential development in Fairfield, California. A description of the clay can be found in Chapter 3.

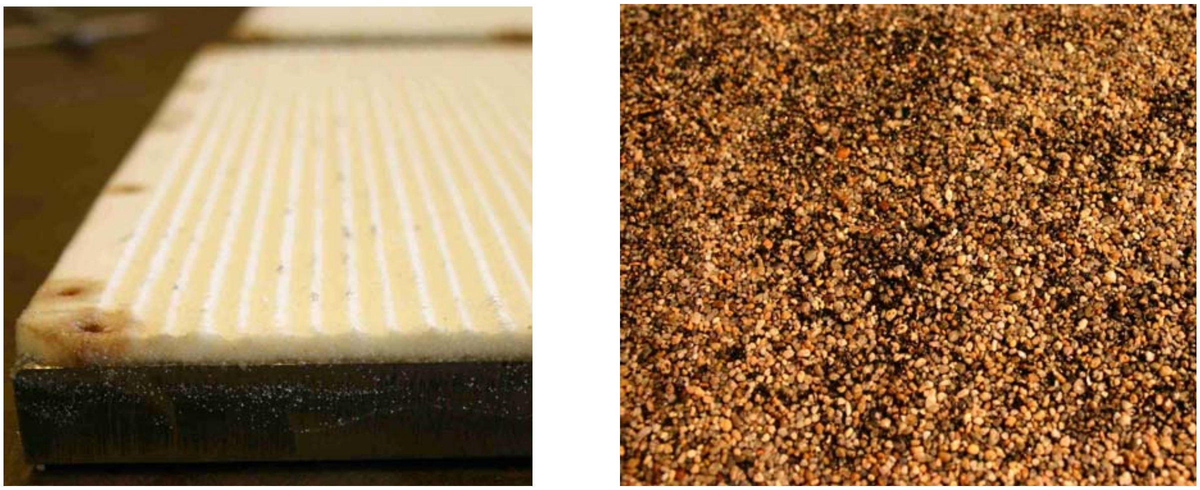


Figure 6-7. The roughened surfaces of the upper steel plate and the lower steel plate.

The first step in preparing the soil was to batch-mix it to ensure uniformity. This was accomplished by remolding the soil at approximately 1.5 times its Liquid Limit in a large mixer. This process yielded a homogenous clay slurry. The clay slurry was passed through a #40 sieve, to remove larger soil particles that could interfere with the preparation of the preformed slickensided failure plane. The soil was passed through the #40 sieve by hand-trowelling the soil on top of the sieve while applying vacuum pressure to the bottom of the sieve, as shown in Figure 6-8.

Figure 6-8. Pushing the soil through the #40 sieve.

Soil Consolidation and Slickenside Preparation

After batch mixing the soil and passing it through the #40 sieve, pore pressure transducers (PPTs) were placed along each of the steel plates, and the clay slurry was consolidated against the plates to create a stiff clay that could be polished to form slickensides. The installation and location of the PPTs is discussed in more detail in the "Instrumentation" section. Figures C-6 and C-7 show the consolidation molds that were used. Two consolidation molds were built, one for each of the slopes that were tested in the model. The layered system shown in Figure C-8 was used to reduce consolidation time. Figure 6-9 shows the clay models being consolidated side-by-side in the consolidation press.

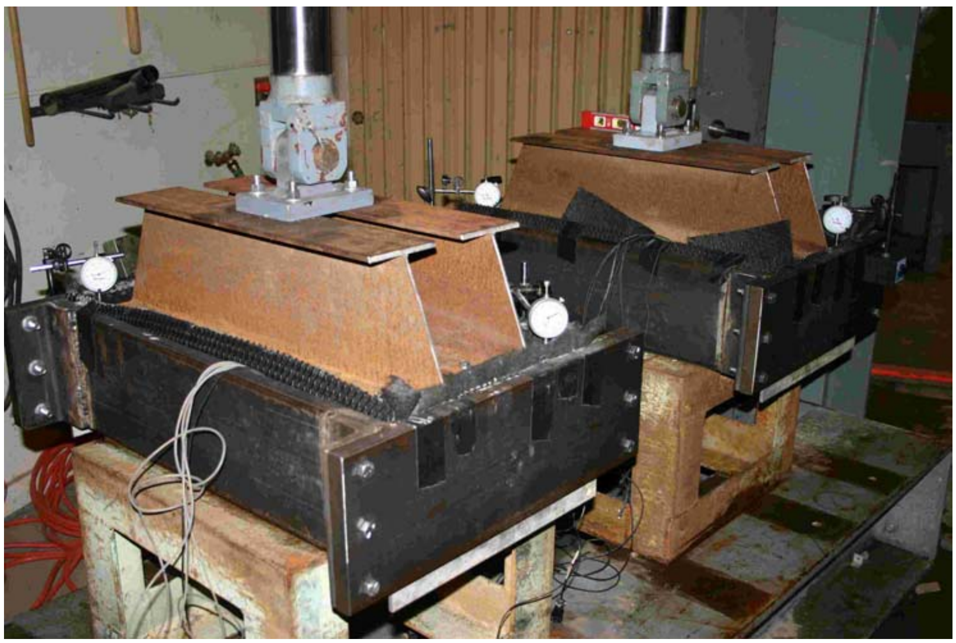


Figure 6-9. Two centrifuge test specimens being consolidated in the consolidation press.

During consolidation, a load-increment ratio of 1 was used for each load step, and each load was maintained until the soil was strong enough to handle the next consolidation load step without significant soil extrusion. The progress of consolidation was tracked using two dial gauges for each mold, as shown in Figure 6-9. Pore pressure transducers embedded in the consolidating clay slurry were used to track pore pressure dissipation during consolidation.

The final consolidation pressure for the soil specimens was 100 pounds per square inch (14,400 psf). This pressure was maintained until completion of primary consolidation, as determined from the dial gauges and pore pressure transducers. Upon completion of consolidation, the specimens were unloaded in stages over an approximately 15 minute period. The consolidation molds were disassembled, and the upper and lower plates were removed. Excess clay was trimmed from the edges of the steel plates using a wire-cutter, and the specimens were cut to the appropriate height using a stiff metal straight-edge.

Once each half of a centrifuge "sliding block" test specimen had been cut to the appropriate thickness, the clay was polished to form a slickensided surface. The soil was polished using a smooth Teflon polishing wheel that had been mounted on a horizontal milling machine. During polishing, the 1.5-inch diameter Teflon wheel was rotated at 15 revolutions per minute, while the specimen was fed under it at an approximate feed rate of 3 inches per minute. Figure 6-10 shows the smooth Teflon wheel that was used to polish each half of the two test specimens for each sliding block model. Two to four passes with a change-in-height setting of 0.005 inches for each pass were needed to fully polish the entire surface of a clay plate. Figure 6-11 shows the appearance of one of the slickensided failure surfaces after completion of the soil polishing process.

Figure 6-10. The soil polishing wheel.

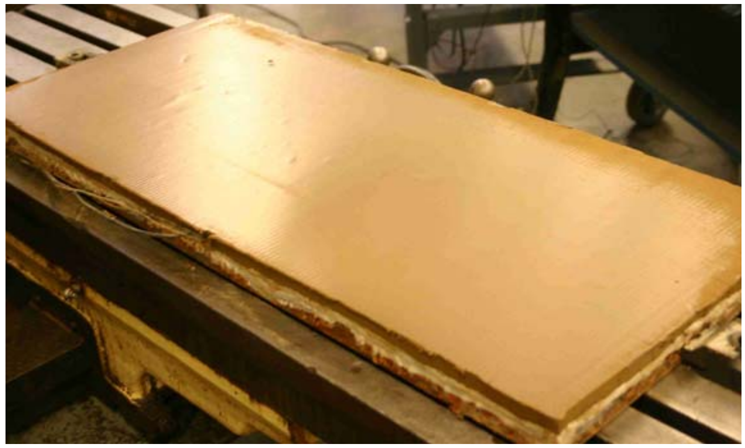


Figure 6-11. The slickensided failure plane for a centrifuge test specimen.

Once the upper and lower halves of each sliding block system had been polished, the two halves were sandwiched together to form the sliding block test specimens. Care was taken to ensure that the direction of slickenside polishing was the same as the direction of shear that the centrifuge test specimen would be subjected to during dynamic loading. Figure 6-12 shows a fully assembled "sliding block" test specimen ready for testing in the centrifuge.

Figure 6-12. A fully assembled "sliding block" centrifuge test specimen.

<u>Kaolinite Markers, Instruments, and Weir Systems for the Sliding Block Models</u>

After each sliding block had been fully assembled, a series of vertical kaolinite markers were installed into the soil at each of the 5/32-inch diameter hole locations shown in Figure C-4. The kaolinite markers were installed by first drilling a hole in the stiff clay soil using a drill bit. The hole was then injected with a kaolinite slurry from the bottom up, using a "tremie-like" approach, as shown in Figure 6-13. The purpose of these vertical kaolinite markers was to show shear localization in the Rancho Solano Clay after the test. Pictures of these markers can be found in the section entitled "Sliding Block Model Dissection".

Figure 6-13. Installing a kaolinite marker in a sliding block test specimen.

After the kaolinite markers had been installed, accelerometers and displacement transducers were mounted on the specimens. For more details on accelerometer and displacement transducer locations, see the "Instrumentation" section.

The next step in the model construction process was to install the weir system used to prevent drying of the clay model during the test. The weir system consisted of aluminum angles bonded to the upper steel plate with quick-set epoxy. These can be seen in Figure 6-14. A series of holes was drilled behind each aluminum angle, as shown in Figure C-4. During the test, deionized water was applied at the upper end of the steel plate, and cascaded down the system of weirs to the lower end of the steel plate. The water then flowed into a channel in the concrete base, where it was guided to a drain at the bottom of the container. The holes drilled behind each weir allowed water to flow into the porous plastic layer, and through the porous plastic to the top of the clay specimen. Leakage of water along the boundary of the porous plastic was prevented by sealing the edges with an acrylic epoxy.

Model Construction

Once the kaolinite markers, instrumentation, and weir system for the test specimens had been installed, the assemblies were bolted inside the container. The cables for the accelerometers, displacement transducers, and pore pressure transducers were zip-tied into place, to prevent cable tangling or breakage during the test. Figure 6-14 shows the side-by-side sliding block models in place in the container prior to spin-up in the centrifuge.

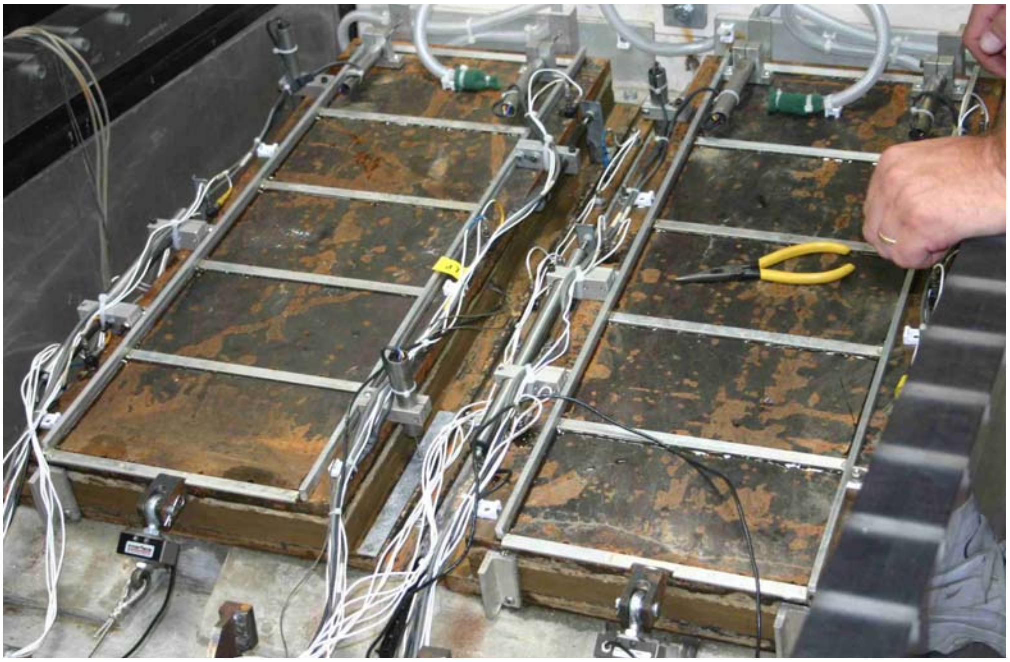


Figure 6-14. Side-by-side sliding block models in the rigid container.

Instrumentation

The sliding block models were instrumented with accelerometers, displacement transducers, pore pressure transducers, and load cells to measure the behavior of the slopes during the static and seismic loading events. Table 6-2 lists the instruments that were used. The approximate instrument sizes and dimensions for the accelerometers, displacement transducers and pore pressure transducers are shown in Figure C-9. The instrument layout that was used for each sliding block model is shown in Figure C-10. The locations of the instruments are shown in Figure C-11.

Accelerometers were placed along both sides of the steel plates to measure the acceleration applied parallel to and perpendicular to the direction of sliding, as shown in Figures C-10 and C-11. Accelerations were measured along the edges of the lower and the upper steel plates.

Displacement transducers were placed along the upper steel plate to measure relative displacement between the upper sliding mass and the fixed base. Displacements perpendicular to the slickensided shearing plane were also measured. Two types of displacement transducers were used to measure displacement parallel to the direction of sliding: Linear Variable Differential Transformers (LVDTs) and Linear Potentiometers (LPs). Displacements perpendicular to the direction of sliding were measured using LVDTs.

Saturated pore pressure transducers were placed at the locations shown in Figure C-10 prior to the placement of the clay slurry. Modeling clay was used to hold the PPTs and their electronic cables in place while the clay slurry was trowelled into place around the instruments. The clay was then consolidated to the desired stiffness around the PPTs.

Load cells were used to measure the force applied by the hydraulic actuator and pulley system during the static loading tests. The load cells were located where the cables connected to the upper steel plates.

Instrument calibrations were performed before the test and verified by comparison to the manufacturer's calibrations. The sign conventions for the calibration factors were established as follows:

- Parallel acceleration is positive in a downslope direction.
- Perpendicular acceleration is positive away from the concrete base.
- Parallel sliding is positive downslope.
- Perpendicular movement of the upper steel plate away from the lower steel plate is positive.
- Compression is positive for pore pressure.
- Shear loads in the downslope direction are positive.

The offset factors shown in Table 6-2 reflect the desired "zero" values. No offsets were applied to acceleration. The offsets for the displacement transducers (LVDTs and LPs) were applied such that the initial voltage reading for each transducer corresponded to an initial relative displacement of zero prior to spin-up. The offsets for the pore pressure transducers (PPTs) were applied so that a reading of zero voltage would correspond to zero pore pressure. The offsets for the load cells (LCs) were applied so that a reading of zero voltage would correspond to zero applied load.

Table 6-2: Instruments Used in Centrifuge Test CLM02

Inst #	Inst Name	Inst Type	Serial #	Model	Inst Range	Amplifier Channel #	Amplifier Gain	Calibration - Model		Offset	
1	A19	ACC	21067	12	100 g	DC48	1	-18.4843	g/V	0	V
2	A10	ACC	3202	10.5	50 g	DC49	1	9.7087	g/V	0	V
3	A11	ACC	21048	10.5	100 g	DC50	1	-19.4932	g/V	0	V
4	A22	ACC	3963	12	50 g	DC51	1	9.5785	g/V	0	V
5	A23	ACC	21318	12	100 g	DC52	1	-21.5517	g/V	0	V
6	A24	ACC	3964	12	50 g	DC53	1	9.5238	g/V	0	V
7	A25	ACC	21319	12	100 g	DC54	1	-19.1571	g/V	0	V
8	A26	ACC	4523	12	50 g	DC55	1	9.5329	g/V	0	V
9	A27	ACC	21320	12	100 g	DC56	1	-20.0803	g/V	0	V
10	A28	ACC	4596	12	50 g	DC57	1	9.4967	g/V	0	V
11	A29	ACC	21321	12	100 g	DC58	1	-19.6464	g/V	0	V
12	A30	ACC	5267	12	50 g	DC59	1	9.6339	g/V	0	V
13	A31	ACC	21322	12	100 g	DC60	1	-19.9601	g/V	0	V
14	A32	ACC	5269	12	50 g	DC61	1	9.6432	g/V	0	V
15	A33	ACC	21323	12	100 g	DC62	1	-21.2766	g/V	0	V
16	A34	ACC	5276	12	50 g	DC63	1	9.5602	g/V	0	V
17	P1	PPT	7985-100	10.5	100 psi	XDCR0	250	-4.6593	psi/V	0.007	V
18	P2	PPT	11146-100	10.5	100 psi	XDCR1	100	5.9380	psi/V	-0.008	V
19	P3	PPT	7811-100	10.5	100 psi	XDCR2	250	4.5360	psi/V	-0.012	V
20	P4	PPT	11151-100	10.5	100 psi	XDCR4	100	6.1059	psi/V	0.081	V
21	P5	PPT	7722-100	10.5	100 psi	XDCR5	250	4.6839	psi/V	0.502	V
22	P6	PPT	11152-100	10.5	100 psi	XDCR6	100	6.0122	psi/V	0.132	V
23	P7	PPT	11149-100	10.5	100 psi	XDCR7	100	6.2719	psi/V	-0.175	V
24	P8	PPT	10315-100	10.5	100 psi	XDCR8	250	4.9444	psi/V	0.136	V
25	P9	PPT	11150-100	12	100 psi	XDCR9	100	6.0772	psi/V	0.073	V
26	P10	PPT	10323-50	12	50 psi	XDCR10	100	5.2278	psi/V	-0.023	V
27	P11	PPT	10041-100	12	100 psi	XDCR11	250	4.7105	psi/V	-0.115	V
28	P12	PPT	11148-100	12	100 psi	XDCR12	100	6.0378	psi/V	-0.021	V
29	P13	PPT	10321-50	12	50 psi	XDCR15	100	5.3916	psi/V	0.021	V
30	P14	PPT	11147-100	12	100 psi	XDCR16	100	6.2419	psi/V	-0.03	V
31	P15	PPT	11143-50	12	50 psi	XDCR17	50	5.5427	psi/V	0.022	V
32	P16	PPT	11141-50	12	50 psi	XDCR18	50	5.4212	psi/V	-0.085	V
33	P17	PPT	11154-200	-	200 psi	XDCR19	250	4.9935	psi/V	-0.054	V
34	L1	LC	181325	10.5	2000 lb	XDCR20	250	200.8	lbf/V	0.055	V
35	L2	LC	181363	12	2000 lb	XDCR21	250	202.4	lbf/V	0.059	V
36	A13	ACC	21043	10.5	100 g	DC32	1	-18.2482	g/V	0	V
37	A14	ACC	3948	10.5	50 g	DC33	1	9.4518	g/V	0	V
38	A5	ACC	5604	10.5	100 g	DC34	1	-18.7266	g/V	0	V
39	A6	ACC	3164	10.5	50 g	DC35	1	9.3809	g/V	0	V
40	A7	ACC	21059	10.5	100 g	DC36	1	-18.3150	g/V	0	V
41	A8	ACC	3166	10.5	50 g	DC37	1	9.3897	g/V	0	V
42	A9	ACC	21046	10.5	100 g	DC38	1	-18.7266	g/V	0	V
43	A20	ACC	3962	12	50 g	DC39	1	9.3897	g/V	0	V
44	A21	ACC	21070	12	100 g	DC40	1	-21.6450	g/V	0	V
45	A12	ACC	3204	10.5	50 g	DC41	1	9.2851	g/V	0	V
46	A3	ACC	5602	10.5	100 g	DC42	1	-18.5874	g/V	0	V
47	A4	ACC	3157	10.5	50 g	DC43	1	9.1075	g/V	0	V
48	A15	ACC	21060	10.5	100 g	DC44	1	-19.0840	g/V	0	V
49	A16	ACC	3949	10.5	50 g	DC45	1	9.4162	g/V	0	V
50	A17	ACC	21061	10.5	100 g	DC46	1	-20.2020	g/V	0	V
51	A18	ACC	3955	10.5	50 g	DC47	1	9.3809	g/V	0	V
52	D1	LVDT	A017-01	10.5	4"	PBP64	N/A (1)	0.5668	in/V	-3.678	V
53	D2	LP	416	10.5	4"	PBP65	N/A (1)	0.3997	in/V	-4.888	V
54	D3	LVDT	469053	10.5	2"	PBP66	N/A (1)	0.2280	in/V	-4.027	V
55	D4	LVDT	455851	10.5	2"	PBP67	N/A (1)	0.2369	in/V	-4.093	V
56	D5	LVDT	469051	10.5	2"	PBP68	N/A (1)	-0.2342	in/V	-0.186	V
57	D6	LVDT	418434	10.5	1.5"	PBP69	N/A (1)	-0.1673	in/V	0.022	V
58	D7	LVDT	A017-02	12	4"	PBP70	N/A (1)	0.5706	in/V	-3.411	V
59	D8	LP	423	12	4"	PBP71	N/A (1)	0.4002	in/V	-4.795	V
60	D9	LVDT	455850	12	2"	PBP72	N/A (1)	0.2365	in/V	-3.555	V
61	D10	LVDT	419741	12	2"	PBP73	N/A (1)	0.2292	in/V	-3.689	V
62	D11	LVDT	434653	12	1"	PBP74	N/A (1)	-0.1385	in/V	0.078	V
63	D12	LVDT	434655	12	1"	PBP75	N/A (1)	-0.1397	in/V	-0.010	V
64	A1	ACC	5598	-	100 g	PBP76	1	18.7266	g/V	0	V
65	A2	ACC	5599	-	100 g	PBP77	1	19.0476	g/V	0	V

Test Procedure

After the model was constructed, the cover was placed on the container, and the container was placed on the centrifuge arm. The centrifuge test was started by gradually increasing the rotation rate. This "spin-up" process was performed gradually, with continuous pore pressure monitoring, to ensure that the induced pore pressures did not trigger a static slope failure. This spin-up process was continued until the centrifuge had reached a rotation rate of 68.2 RPM, which corresponded to a centrifugal acceleration of 45.0 g.

Static Loading and Seismic Loading Events

Once the target centrifugal acceleration had been reached, water was applied to the slopes, and 2.5 hours was allowed for the model to come to equilibrium. The model was then subjected to a series of "static loading" and "seismic loading" events, which were intended to move the upper steel plates downslope. The sequence of static loading and seismic loading events is shown in Table 6-3.

Table 6-3: Centrifuge Test Loading Events

Event ID	Name of Motion	Time	Freq. (Hz)	# Cycles	Amp. Factor	Peak to Peak Base Accel. (g)	Cent. Accel. (g)	Ratio H/V Accel
Spin-Up	0 g to 45 g	10:33 – 12:20						
CLM02_S1	Static Pull #1 (both slopes)	13:59 – 17:04						
CLM02_01	Shake #1	17:20	55	5	.24	6.0	45.3	.05
CLM02_02	Shake #2	17:43	55	5	.97	30.2	45.0	.32
CLM02_03	Shake #3	18:25	55	20	3.00	48.8	45.0	.51
CLM02_S2	Static Pull #2 (both slopes)	20:20 – 22:32						
Spin-Down	45 g to 0 g	22:35 – 23:15						

During the static loading events, the upper steel plates were moved downslope by applying a displacement-controlled load. For Static Pull #1, the load was applied in two stages: fast loading followed by slow loading. The fast loading rate was approximately 0.05 in/min. The fast loading stage was maintained until a constant shearing resistance had been reached. At that point, the rate of displacement was reduced to approximately 0.0005 in/min. The slow loading rate was maintained until a constant shearing resistance had been measured. For Static Pull #2, the load was applied in three stages: a fast loading stage, a slow loading stage, and another fast loading stage, using the same loading rates (0.05 in/min and 0.0005 in/min).

During the seismic loading events, either five or twenty horizontal acceleration pulses of approximately the same amplitude were applied, with a frequency of 55 Hz. At the 45 g centrifugal acceleration of the model, the 55 Hz shaking corresponded to 1.2 Hz shaking at prototype scale.

During both the static and seismic loading events, all of the instrument channels listed in Table 6-2 were read at various intervals of time. "Slow Data" was acquired by sampling all instrument channels at a frequency that varied from 1/2 to 1 Hz throughout the entire centrifuge test. Additional "Fast Data" was acquired by sampling all instrument channels at a frequency of 4096 Hz during the seismic loading events.

Static Slope Behavior

Representative downslope displacements were calculated by averaging the relative displacements measured using the relative displacement instruments. Representative measurements of pore pressure during shear were developed by averaging the measured pore pressures for instruments located above and below the slickensided shearing plane. Figures 6-15 and 6-16 show the applied loads, the resulting displacements, and the measured pore pressures for the 10.5° slope and the 12° slope during Static Pull #1. Figures 6-17 and 6-18 show the loads, displacements, and pore pressures during Static Pull #2. The perpendicular displacement response is not shown in Figures 6-15 through 6-18, because no measurable displacement occurred during static loading.

During Static Pull #2, both of the steel loading cables broke, rapidly unloading the slopes. For the 10.5° slope, the cable broke during the first rapid loading. For the 12° slope, the cable broke during the second rapid loading. In neither case was the full strength of the soil mobilized prior to failure of the cable.

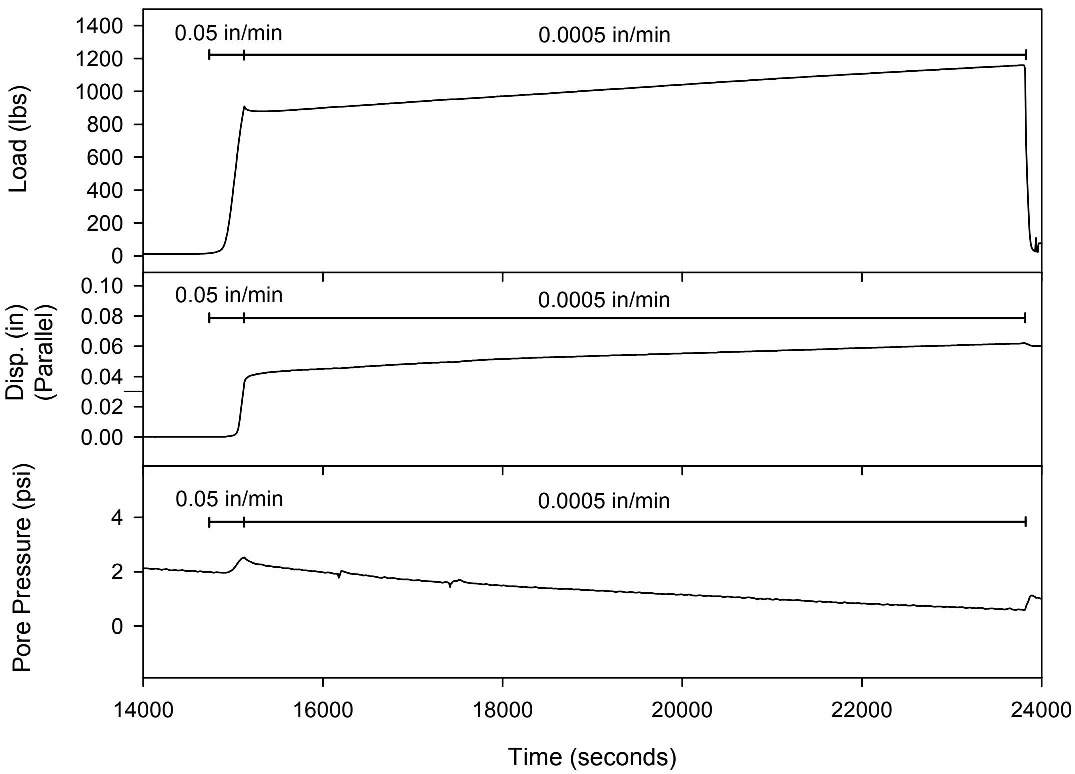


Figure 6-15. Shear behavior of the 10.5° sliding block model during Static Pull #1.

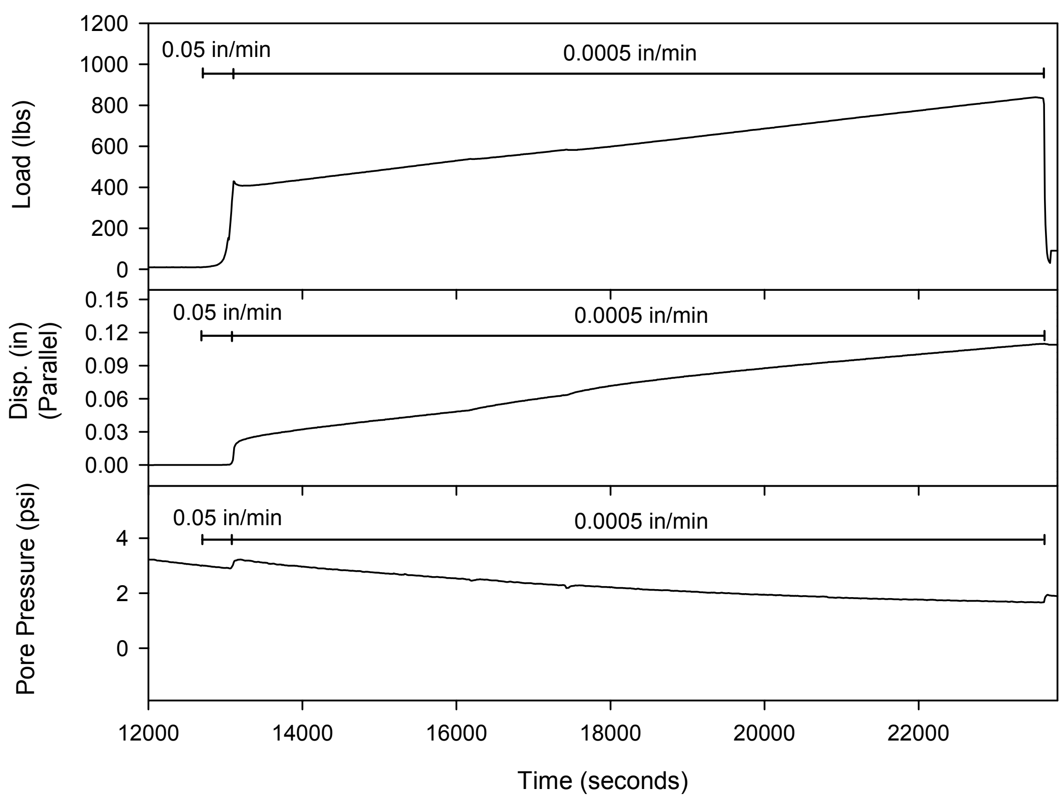


Figure 6-16. Shear behavior of the 12° sliding block model during Static Pull #1.

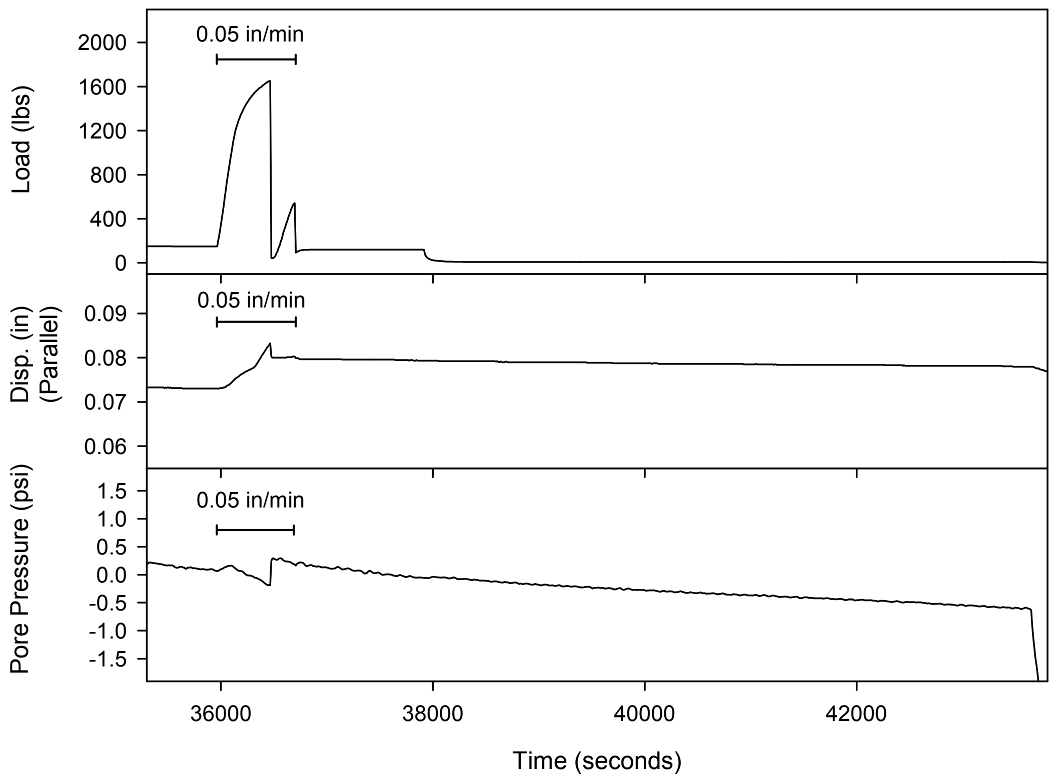


Figure 6-17. Shear behavior of the 10.5° sliding block model during Static Pull #2.

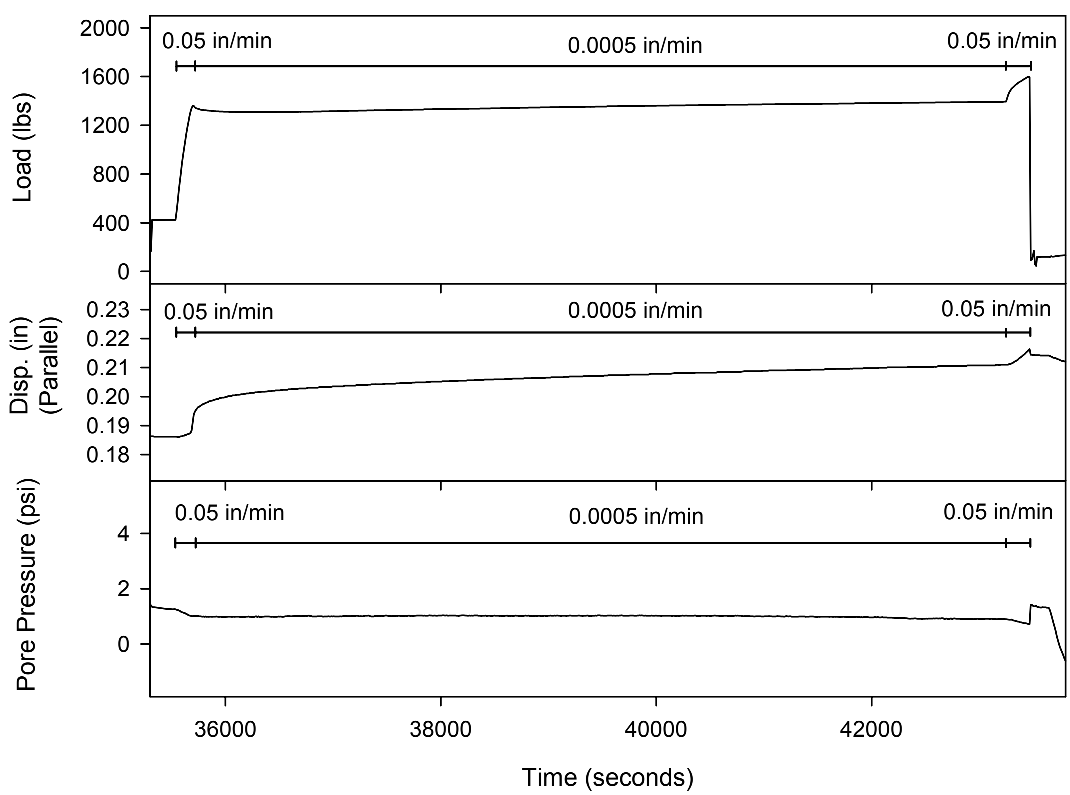


Figure 6-18. Shear behavior of the 12° sliding block model during Static Pull #2.

Seismic Slope Behavior

During the seismic loading events, shaking-induced “input” accelerations were measured at four different locations along the base of the sliding block models. The resulting “response” accelerations were measured at four different locations along the tops of the models. At each sampling location, accelerations were measured parallel and perpendicular to the downslope direction of sliding. The overall accelerations were calculated by averaging the measured values. For example, the parallel base acceleration for the 10.5 degree slope was calculated by averaging the response of instruments A3, A5, A7, and A9. The overall acceleration responses are shown in Figures 6-19 through 6-24.

Relative displacements were calculated by averaging the measured displacements. The downslope displacements are shown in Figures 6-19 through 6-24. No measurable perpendicualar displacements occurred during any of the shaking events.

Pore Pressure Response

The overall pore pressure response at the slickensided shearing plane was estimated by averaging the pore pressure responses above and below the slickensided plane. The pore pressure response for the 10.5° slope is shown in Figure 6-25. The pore pressure response for the 12° slope is shown in Figure 6-26.

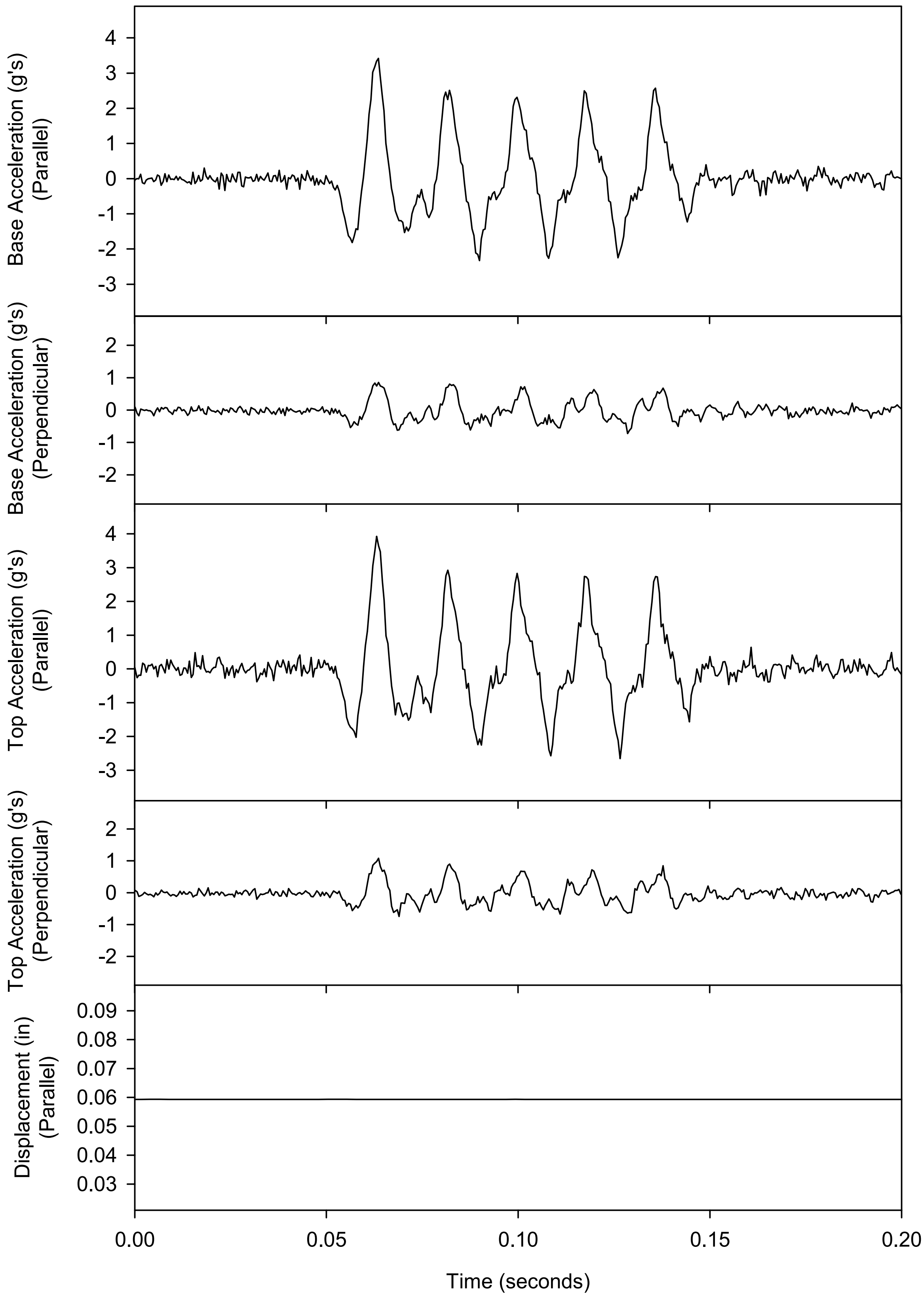


Figure 6-19. Recorded slope behavior for the 10.5° slope during Shake 1.

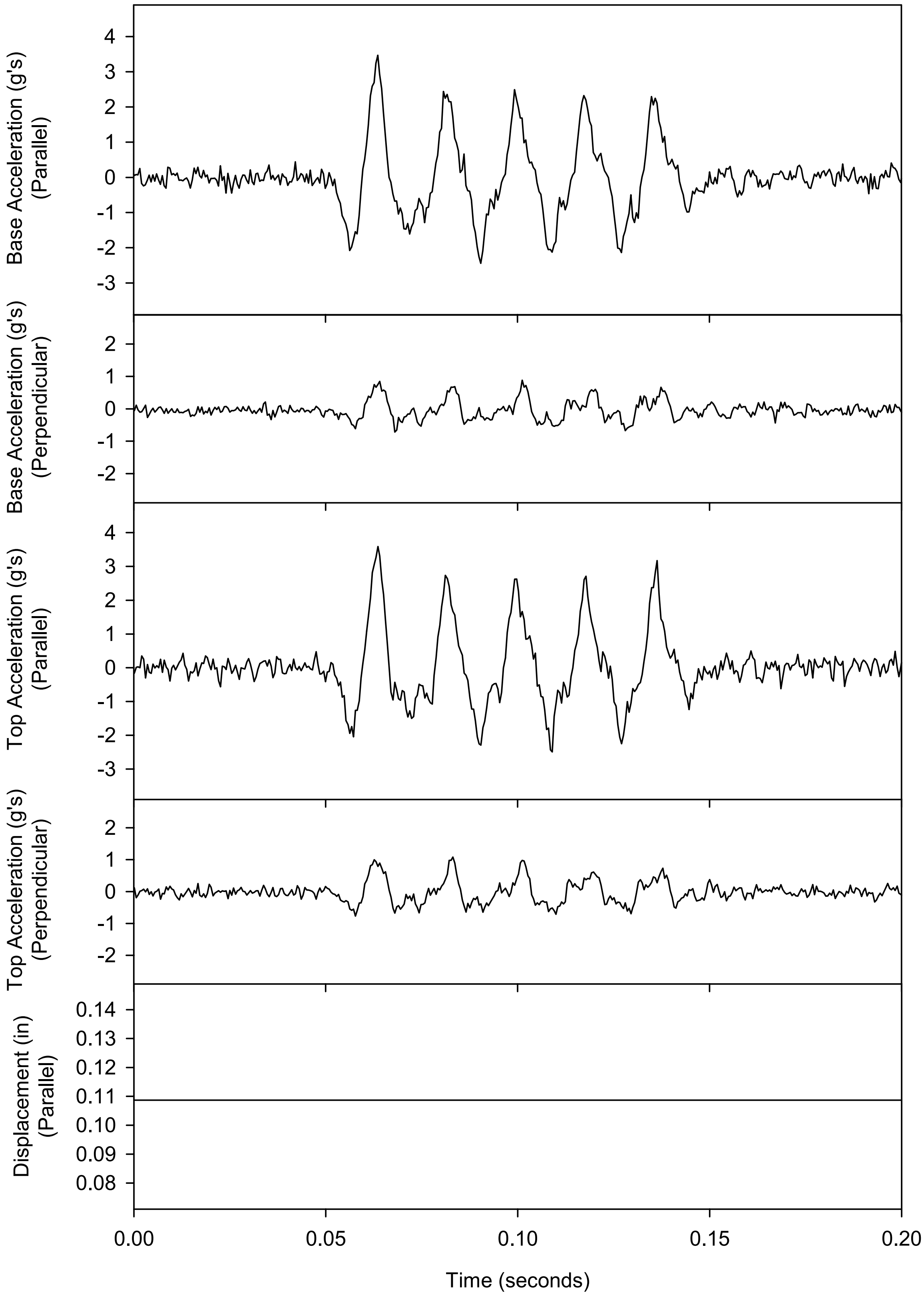


Figure 6-20. Recorded slope behavior for the 12° slope during Shake 1.

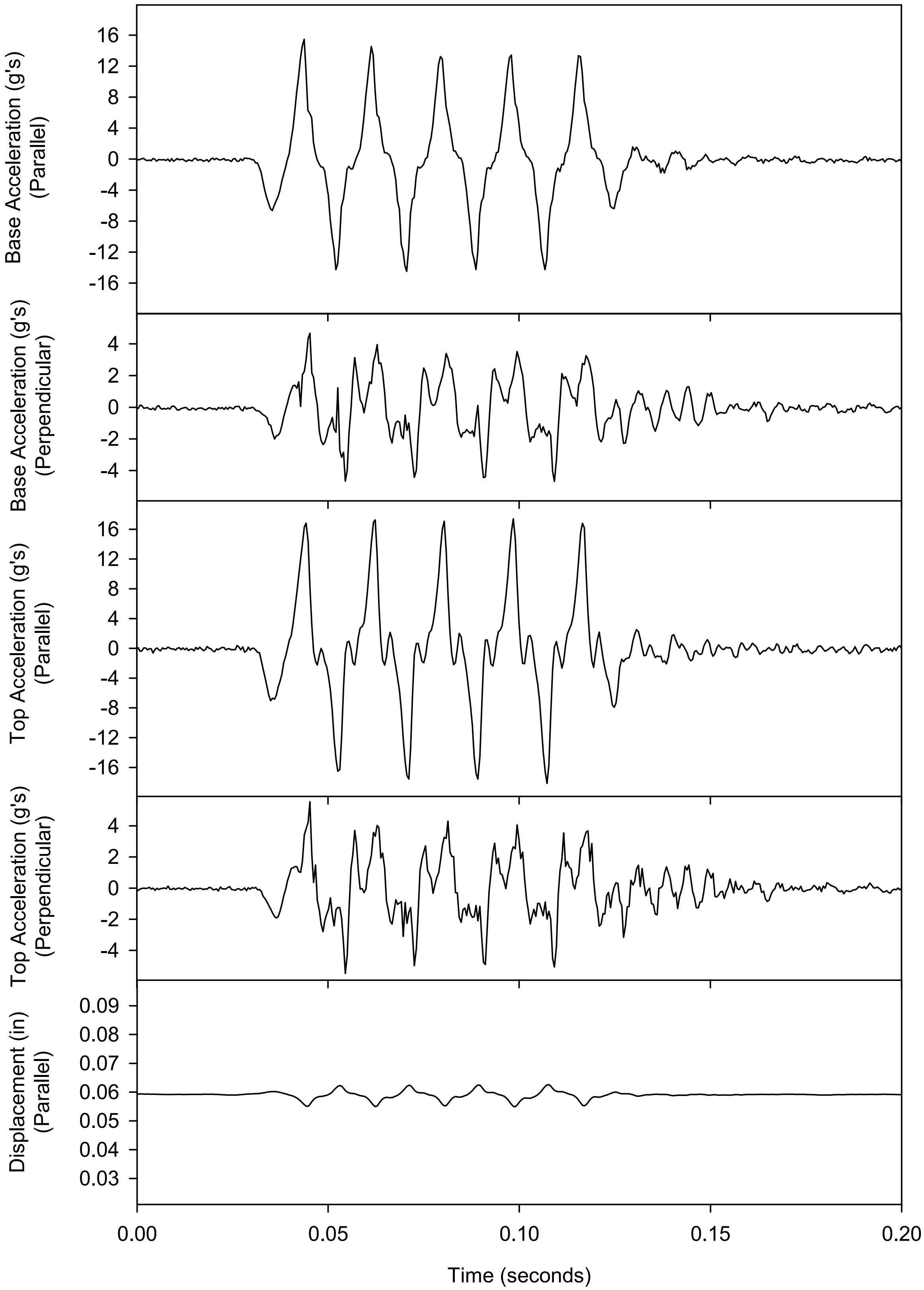


Figure 6-21. Recorded slope behavior for the 10.5° slope during Shake 2.

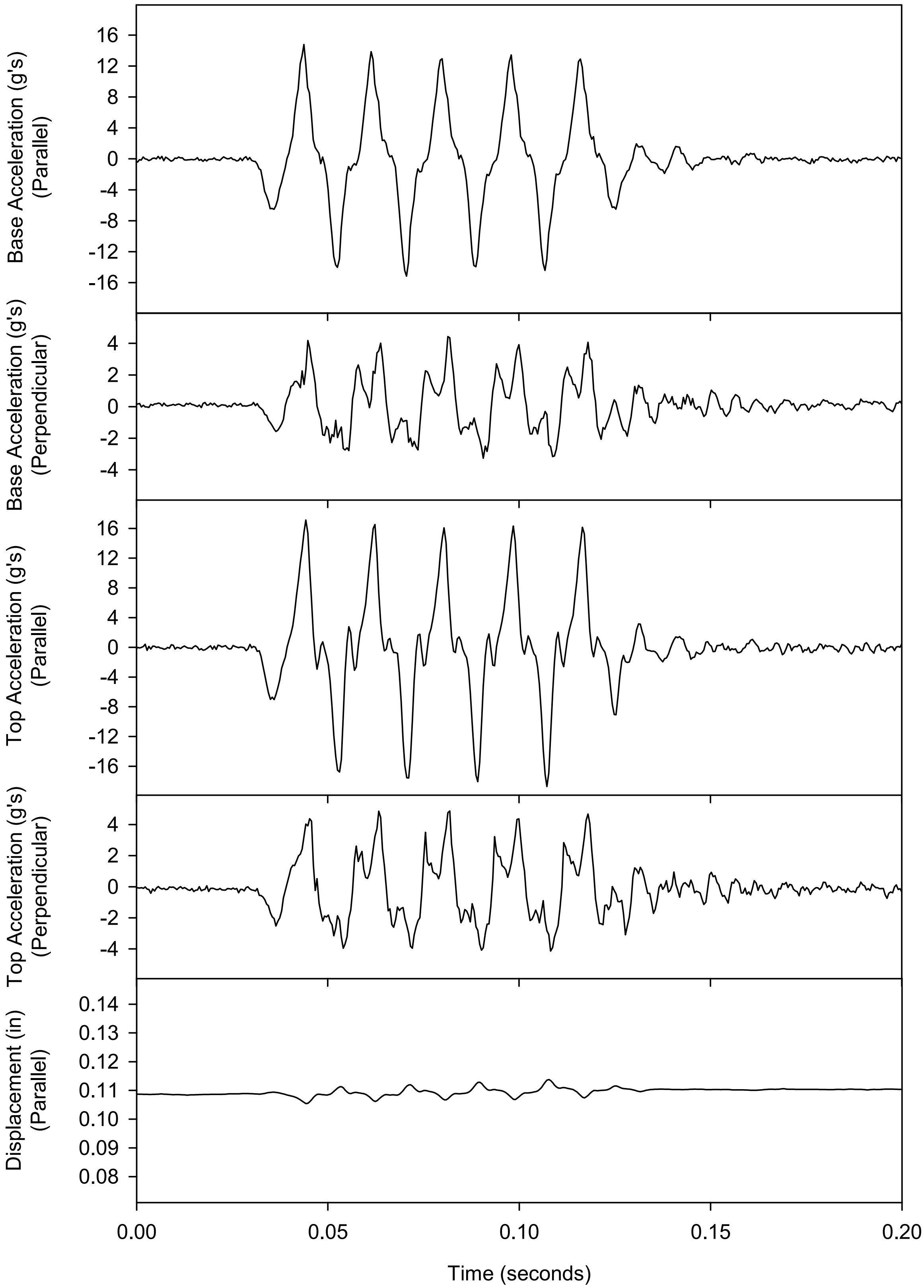


Figure 6-22. Recorded slope behavior for the 12° slope during Shake 2.

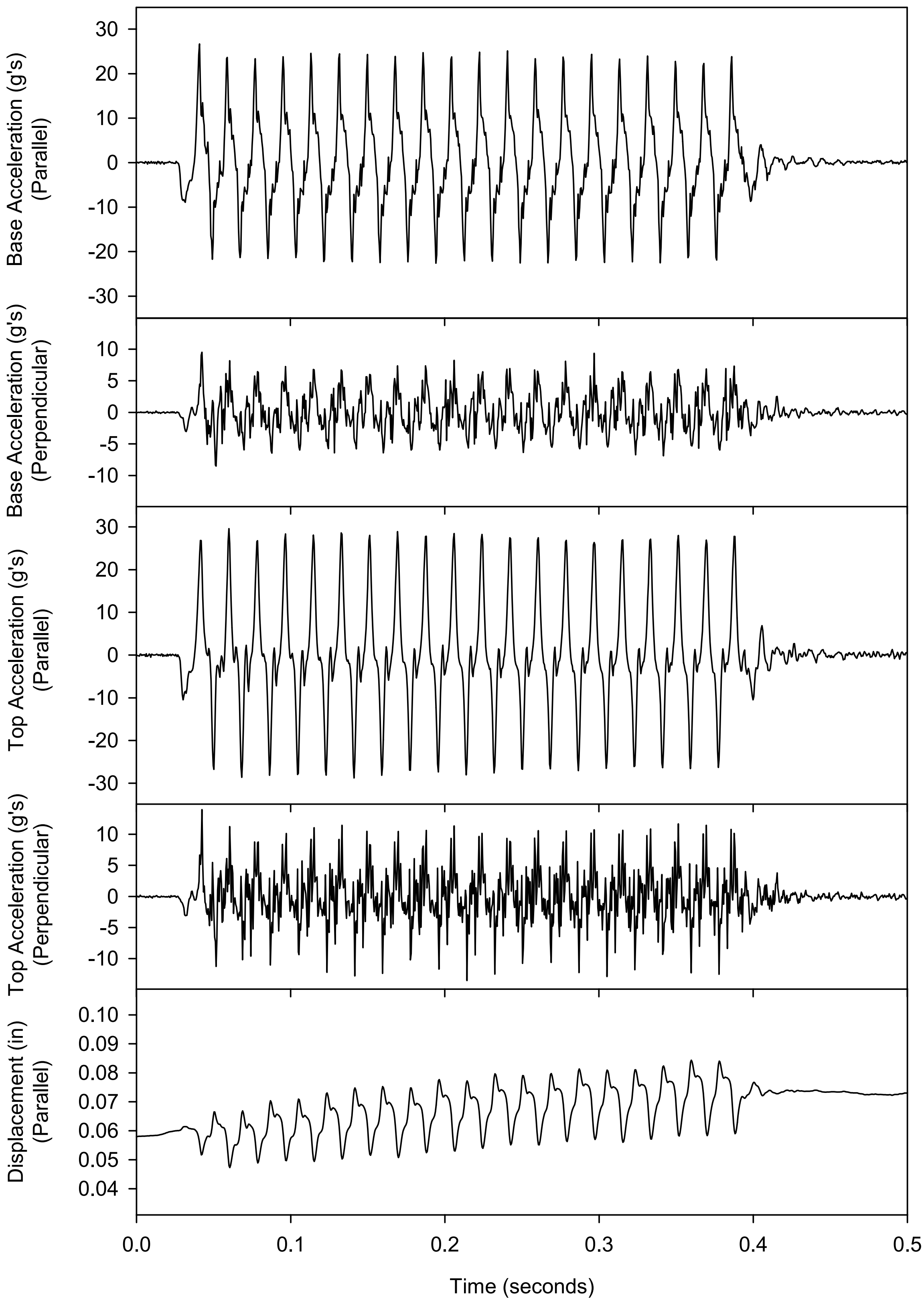


Figure 6-23. Recorded slope behavior for the 10.5° slope during Shake 3.

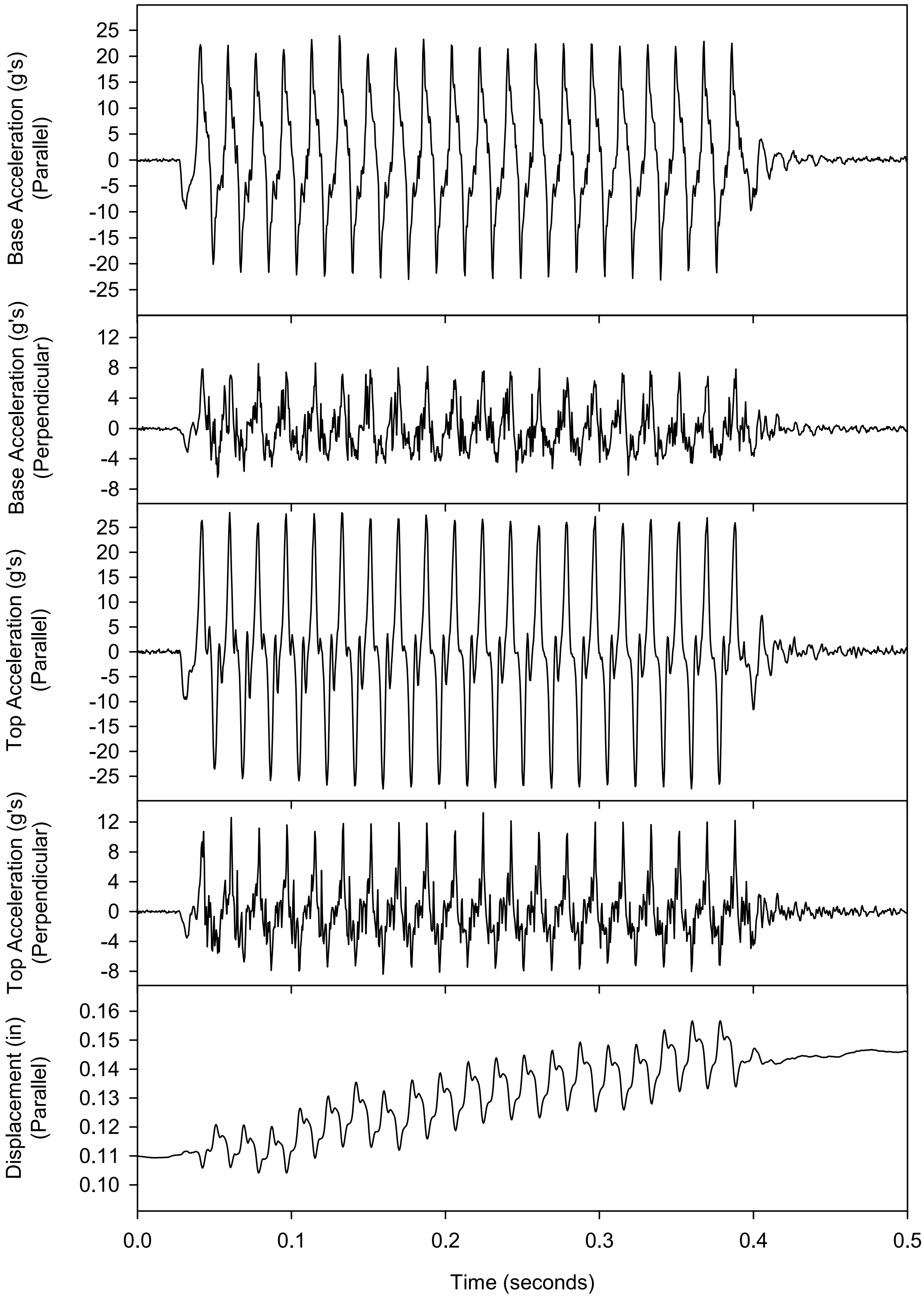


Figure 6-24. Recorded slope behavior for the 12° slope during Shake 3.

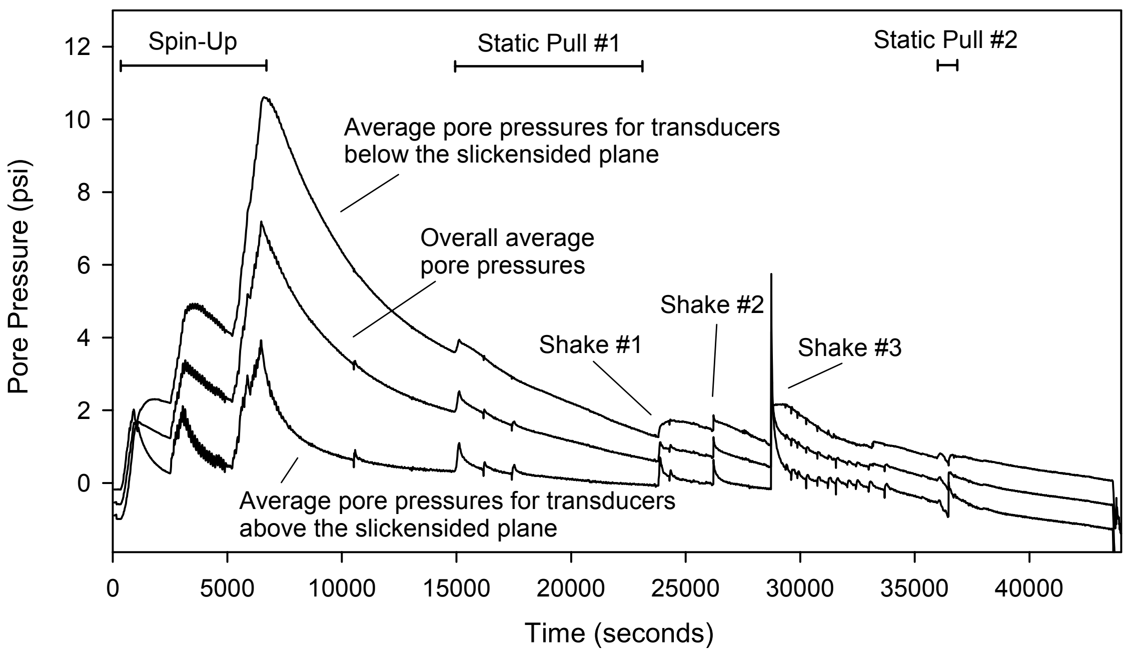


Figure 6-25. Overall pore pressure response of the 10.5° sliding block model.

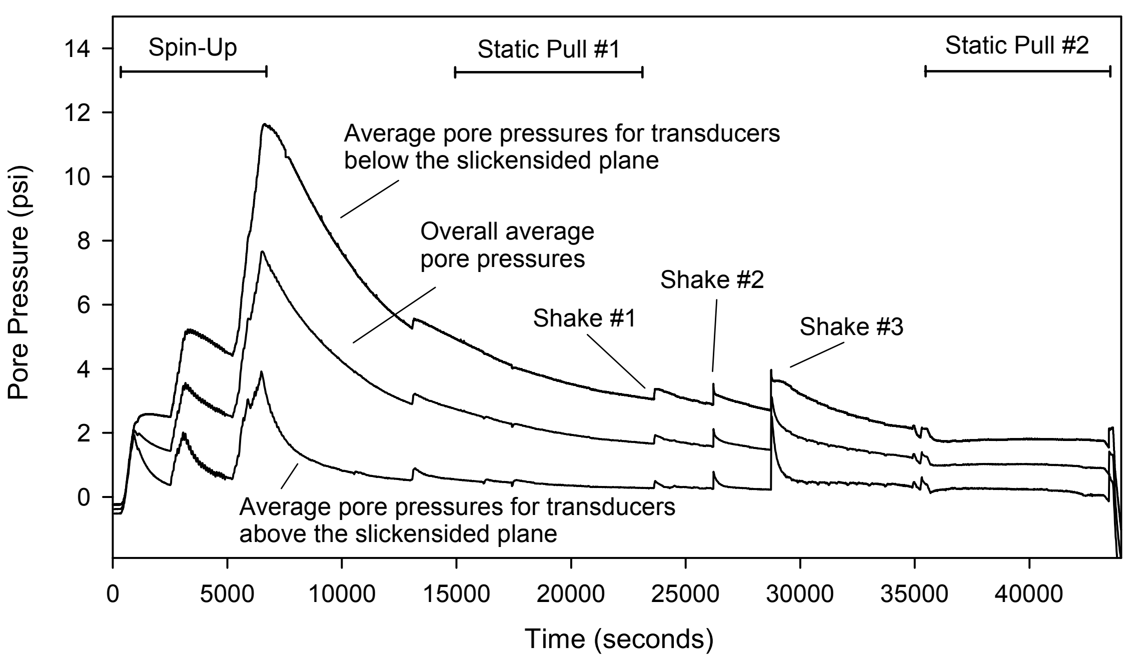


Figure 6-26. Overall pore pressure response of the 12° sliding block model.

Sliding Block Model Dissection

Upon completion of the centrifuge test, the sliding block models were removed from the container and dissected to examine the mechanism of shearing within the clay. The appearances of the preformed slickensided failure plane for the 10.5 degree slope and the 12 degree slope sliding block models are shown in Figure 6-27 and Figure 6-28. The appearances of the vertical kaolinite markers for the 10.5 degree slope and the 12 degree slope sliding block models are shown in Figure 6-29 and Figure 6-30.

Figure 6-27: Slickensided failure plane for the 10.5º slope sliding block model.

Figure 6-28: Slickensided failure plane for the 12º slope sliding block model.

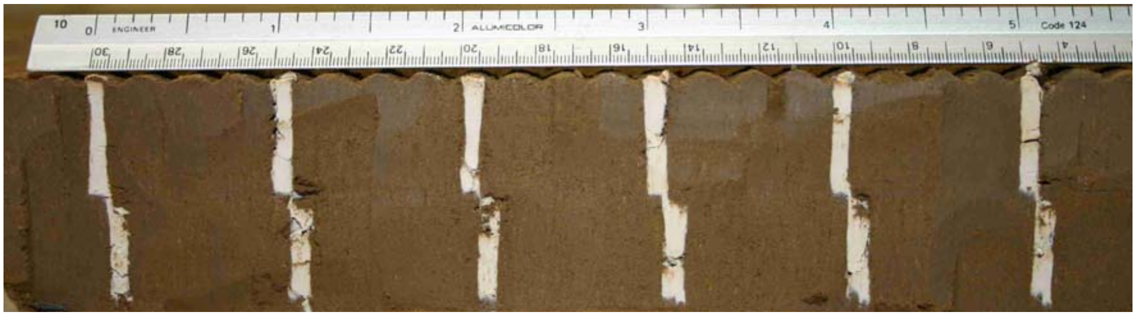


Figure 6-29: Excavated kaolinite columns from the 10.5º slope.

Figure 6-30: Excavated kaolinite columns from the 12º slope.

Analysis of Stress-Displacement Behavior During Shaking

The shear stresses, normal stresses, and displacements along the preformed slickensided plane can be calculated using the recorded accelerations and displacements. The resulting stress-displacement relationships provide valuable insight into the cyclic-loading strength behavior of the slickensided interface. These calculations require an understanding of how scaling laws and signal processing techniques are used to analyze data that is recorded in the centrifuge.

Signal Processing

Signal processing and numerical integration of the data were necessary to convert the recorded dynamic accelerations and displacements into accurate values of relative displacement between the upper and lower steel plates. The signal processing techniques that were used are similar to those used in site response studies (e.g., Zeghal et al. 1995) and soil-pile-interaction studies (e.g., Wilson et al. 2000) for calculating displacements during dynamic loading.

During test CLM02, displacements were measured using three different types of instruments: linear potentiometers (LP's), linear variable differential transformers (LVDTs), and accelerometers. The LPs and LVDTs provided direct measurements of displacement, while the accelerometers provided an indirect measurement of displacement. Displacement measurements were obtained from recorded acceleration values by double-integrating the relative acceleration values between two adjacent accelerometers. The relative strengths and weaknesses of the three methods are given in Table 6-4.

Table 6-4: Strengths and Weaknesses of Three Methods for Evaluating Displacement

Method	Strengths	Weaknesses
Use of LPs to measure displacement	– Accurate over a wider frequency range than LVDTs. – Greater range of displacement than LVDTs.	– Significant amounts of electrical noise. – Not as accurate as LVDTs for small displacements.
Use of LVDTs to measure displacement	– More accurate for small displacements.	– Cannot capture high frequency displacement accurately. – Suffer from phase lag at high frequencies.
Double-integration of measured accelerations to compute displacement	– Have high resolution when used to capture high-frequency displacement behavior.	– Cannot measure permanent displacements. – Cannot capture low frequency displacement accurately.

The most accurate measurements of dynamic displacement can be obtained using signal processing techniques to combine the strengths of different types of instruments in a fashion that minimizes their respective weaknesses. One of the most useful findings from centrifuge test CLM01 was that signal processing could be used to combine the low frequency displacement response from the LVDTs with the high frequency displacement response from the accelerometers. The resulting displacement measurement agreed well with the displacement measured by the LPs, but gave higher resolution and less noise than the LP displacement measurements. Figure 6-31 shows a comparison between the recorded LP data and the combined LVDT and accelerometer data for test CLM01.

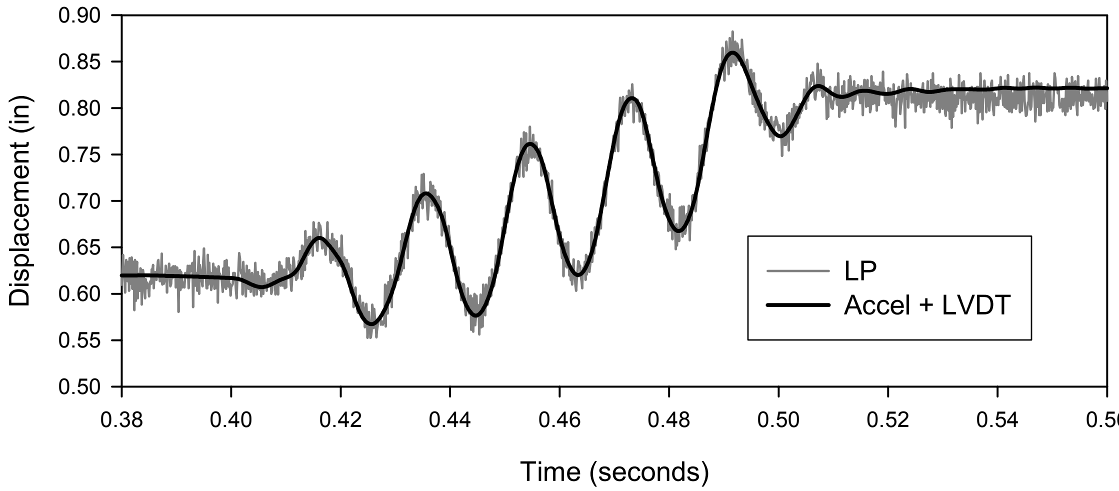


Figure 6-31: Comparison between LP data and combined accelerometer and LVDT data.

During test CLM02, a different mounting technique was used for some of the displacement transducer flags. ("Flags" are aluminum angle sections attached to the lower steel plate that provide a fixed reference point for measuring the downslope displacement. They are visible in Figure 6-14). The flags used in test CLM02 were not sufficiently rigid, and flexion of these flags during shaking resulted in unreliable LP measurements of displacement during test CLM02. Consequently, reliable measurements of displacement for test CLM02 could only be achieved using signal processing techniques to combine the recorded accelerometer and displacement transducer data. The dynamic displacements shown in Figures 6-19 to 6-24 were calculated by combining the accelerometer and LVDT data using the following signal processing approach:

1) A low pass Butterworth filter was applied to the average LVDT measurements to minimize high frequency noise. The magnitude of the frequency response of an *n*th order low pass filter can be defined mathematically as:

$$G_n(\omega)_{\text{Low Pass}} = \left|H_n(j\omega)\right| = \frac{1}{\sqrt{1+\left(\dfrac{\omega}{\omega_c}\right)^{2n}}}, \qquad (6\text{-}1)$$

where: $G_{\text{Low Pass}}$ = gain of the low pass filter,
H = the transfer function,
j = the imaginary number,

n = the order of the filter,
ω = the angular frequency of the signal, and
ω_c = the corner frequency (also known as the cutoff frequency).

For Shake 1, it was not necessary to filter the data, because no response was observed in any of the displacement transducers. For Shake 2, only a small dynamic response was observed, and a fourth order filter with a corner frequency of 30 Hz was used for both slopes. For Shake 3, which exhibited larger displacements, a fourth order filter with a corner frequency of 7.4 Hz was used for both slopes. The lower corner frequency was used to reduce the LVDT contribution to the displacement time history, which minimized the pronounced phase lag that was produced during the larger shaking event.

2) Recorded accelerations were converted to displacements by doubly integrating the average relative accelerations between the upper and lower steel plates for each slope.

3) A high pass filter was applied to the displacement time history that had been generated from the recorded acceleration values in Step 2. This high pass filter can be defined mathematically as:

$$G_n(\omega)_{\text{High Pass}} = 1 - G_n(\omega)_{\text{Low Pass}}, \tag{6-2}$$

where: $G_{\text{High Pass}}$ = gain of the high pass filter, and
$G_{\text{Low Pass}}$ = gain of the low pass filter that was applied to the LVDTs.

The form of the high pass filter was chosen such that the sum of the low pass and the high pass filters was always unity. This ensured that the total dynamic displacement would have continuous contributions from both the LVDT and accelerometer displacement data.

4) The filtered LVDT displacement time history was added to the filtered accelerometer displacement time history to yield the total dynamic displacement time history.

Calculated Stress-Slip Behavior

For the static loading phases, displacement data was recorded slowly over time using both LPs and LVDTs, which supplemented each other. No signal processing was needed for interpretation of these results. The applied shear stress and effective normal stress on the slickensided plane during the static loading phases were calculated using the following equations:

$$\text{Shear Stress} = \frac{W \cdot \sin(\beta) + T}{A}, \tag{6-3}$$

$$\text{Normal Stress} = \frac{W \cdot \cos(\beta) - U}{A}, \tag{6-4}$$

where:
W = weight of the sliding block,
β = slope angle,
T = pulling force in the load cell,
U = pore pressure force acting on the slickensided plane, and
A = area of the slickensided plane.

For the seismic loading phases, it was necessary to use signal processing techniques to develop an accurate understanding of the displacement behavior. The applied shear stress and effective normal stress on the slickensided plane during the seismic loading phases were calculated using the following equations:

$$\text{Shear Stress} = \frac{W \cdot \sin(\beta) - M \cdot a_{para}}{A}, \tag{6-5}$$

$$\text{Normal Stress} = \frac{W \cdot \cos(\beta) - U_o}{A}, \tag{6-6}$$

where:
W = weight of the sliding block,
M = mass of the sliding block,
β = slope angle,
a_{para} = acceleration applied parallel to the orientation of the slickensided plane (sign convention is positive upward, and toward the low end of the slope),

U_o = intial pore pressure force acting on the slickensided plane, and
A = area of the slickensided plane.

Note that the equation for normal stress uses an approach that is similar to the total stress approach used for consolidated-undrained triaxial testing, in which the initial effective consolidation stress is used to characterize the normal stress throughout the test. This approach is necessary for the seismic loading events because the dynamic load is applied rapidly, and resulting changes in pore pressure cannot equalize during the short loading period. Consequently, the effective normal stress on the slip surface during shaking is unknown.

In order to present the static and seismic load-displacement results on the same graph, the results have to be interpreted in terms of the shear stress ratio, defined as:

$$\text{Shear Stress Ratio} = \frac{\text{Shear Stress on the Slip Plane}}{\text{Initial Effective Normal Stress on the Slip Plane}} \qquad (6\text{-}7)$$

The variation of measured displacement with shear stress ratio for static and seismic loading of the 10.5° slope is shown in Figure 6-32. A similar plot for the 12° slope is shown in Figure 6-33.

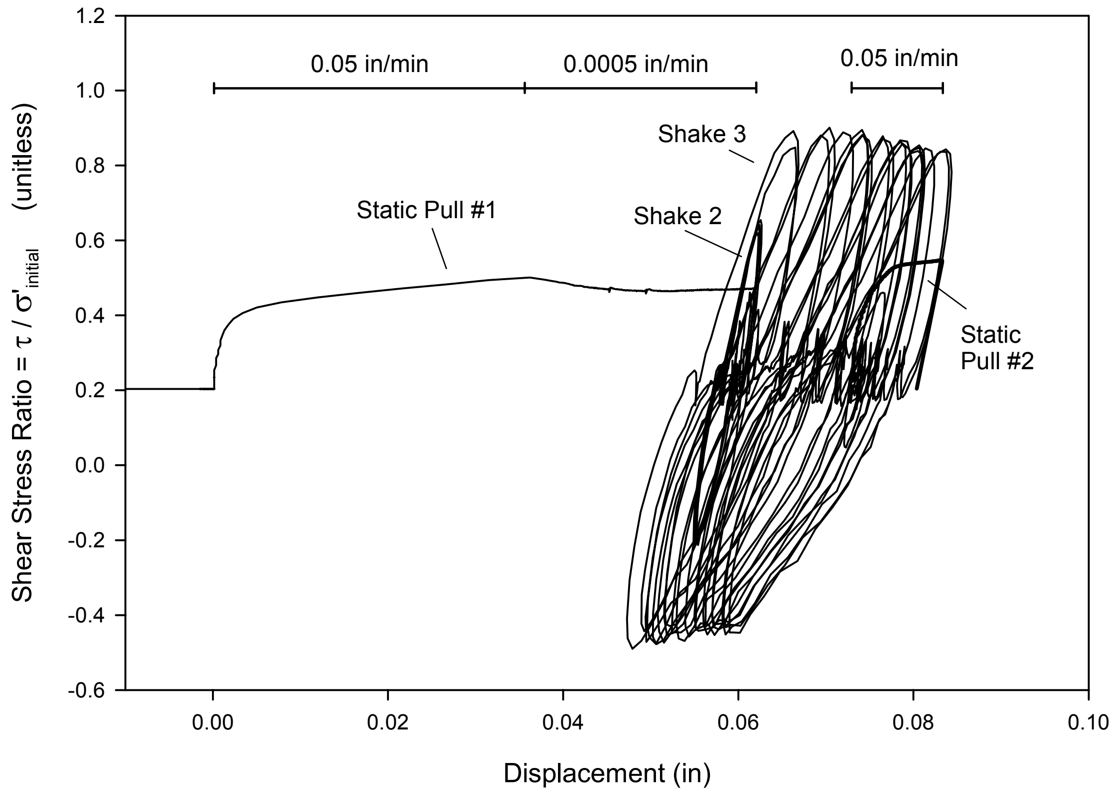


Figure 6-32: Shear stress ratio vs. displacement for the 10.5° slope.

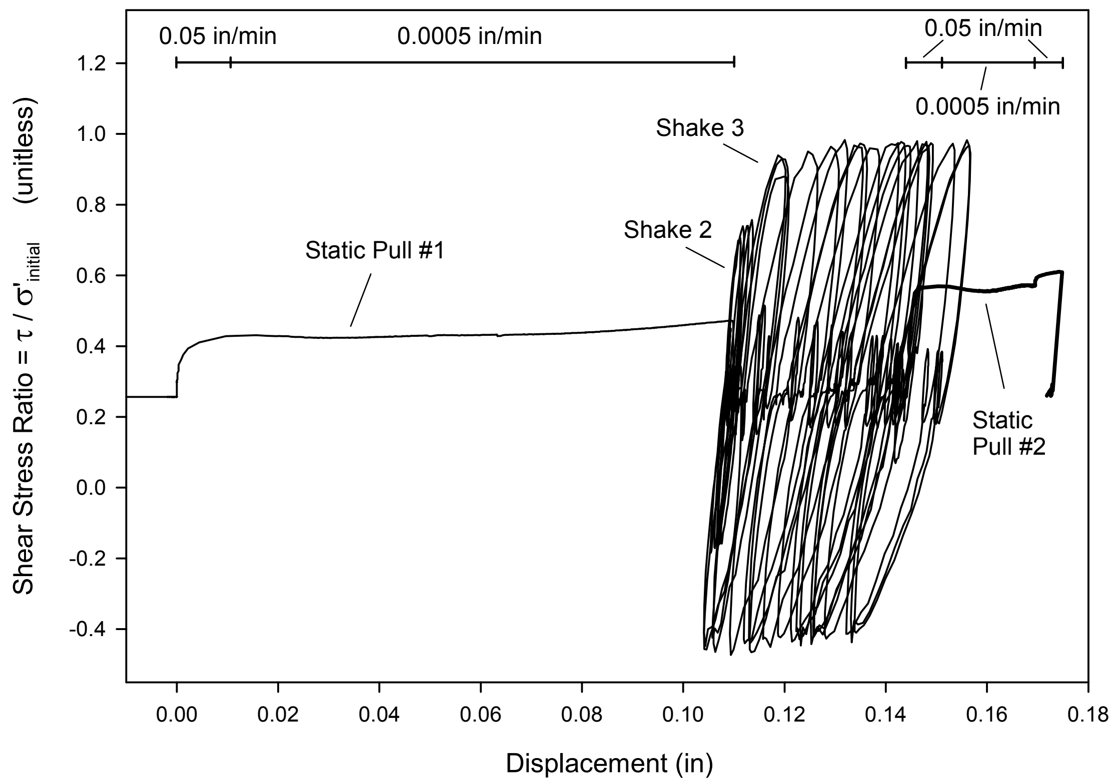


Figure 6-33: Shear stress ratio vs. displacement for the 12° slope.

Discussion of Centrifuge Test Results

Previous studies on the earthquake behavior of slickensides have focused primarily on the rapid, one-directional shear response of slickensided surfaces (Skempton, 1985; Lemos et al., 1985; Tika et al., 1996; Tika and Hutchinson, 1999; and Vesseley and Cornforth, 1998). One-directional loading tests are useful for studying the shear behavior of slickensides under rapid loading, but they do not model earthquake loading conditions, because they do not involve cyclic loading. The centrifuge test program described in this chapter provides a procedure for investigating behavior during rapid cyclic loading, and is a better method of investigating the behavior of slickensided slip surfaces under earthquake loading.

During Static Pull #1, a slow steady-state shearing condition was achieved for both slopes, which corresponded to a shear stress ratio $\tau / \sigma'_{initial} = 0.46$ for the 10.5° sliding block model and $\tau / \sigma'_{initial} = 0.42$ for the 12° sliding block model.

During Shake #1, no movement was observed for either slope, which was not surprising because the applied stress was transient in nature and induced a shear stress ratio of only 0.34.

During Shake #2, five pulses of shear stress were applied at a frequency of 55 Hz. For the 10.5° model, the maximum value of applied shear stress ratio was 0.65 (40% higher than the maximum static shear stress ratio). For the 12° model, the applied shear stress ratio was 0.73 (70% higher than the maximum static shear stress ratio).

Hysteretic stress-displacement behavior was observed in both slopes during Shake #2, with only small amounts of permanent displacement remaining after shaking.

During Shake #3, twenty pulses of shear stress were applied at a frequency of 55 Hz. For the 10.5° slope, the maximum value of applied shear stress ratio was 0.88 (90% higher than the maximum static shear stress ratio). For the 12° slope, the applied shear stress ratio was 0.94 (120% higher than the maximum static shear stress ratio).

Hysteretic stress-displacement behavior was observed during Shake #3, with a final relative displacement of 0.014" for the 10.5° model and 0.037" for the 12° model.

After shaking, both soil slopes were subjected to Static Pull #2. The load cable for the 10.5° slope broke during initial load application, which indicated that the post-shaking shear stress ratio for the 10.5° slope was at least 0.54 (at an approximate loading rate of 0.05"/min). A steady-state slow shearing condition was achieved for the 12° model, which indicated that the post-shaking shear stress ratio was 0.55 (at an approximate loading rate of 0.0005"/min).

The following conclusions were drawn from the observed stress-displacement behavior, the observed pore pressure behavior, and the data obtained during dissection of the model slopes:

- The polishing process used to prepare the slickensided surfaces was successful. When the models were dissected, the slickensided surface still had the shiny, reflective look associated with slickensides. When sheared slowly prior to shaking, the slickensided surfaces reached maximum shear stress ratios in the range of 0.42 to 0.46 at very small displacements. These shear stress ratios agree with those measured in the Bromhead ring shear device, providing validation for the polishing process that was used to prepare the centrifuge specimens. The measured shear resistance is greater than the residual strengths measured in the direct shear device for specimens prepared using a similar Teflon “dry polish” approach.

- The shear resistance mobilized during dynamic loading was significantly larger than the drained residual strength of the soil. A fundamental and complete understanding of the magnitude of this strength gain cannot be obtained from the centrifuge data alone, because it is not possible to isolate the effect of cyclic loading from the effect of the rate at which the cyclic loading occurred.

- Both the static and dynamic shear displacements were concentrated along the pre-formed slickensided plane. Even during dynamic loading, the pre-formed slickensided surface was much weaker than the surrounding heavily-overconsolidated clay soil.

- Dynamic loading caused a positive pore pressure response in the soil surrounding the slickensided plane. From the recorded pore pressure data, it was not clear whether this pore pressure increase was caused by shearing along the slickensided plane, by stress mobilization in the soil surrounding the slickensided plane, or by some sort of boundary effect at the soil/steel plate interface.

- A catastrophic sliding failure did not occur, even during a very large shaking event.

- The post-shaking shear resistance was higher than the resistance prior to shaking.

- No displacements of the slopes occurred after shaking stopped.

CHAPTER 7: NEWMARK ANALYSES

The Newmark (1965) analyses described in this chapter were performed to compute dynamic displacements for the significant shaking events in centrifuge test CLM02. Comparison of the computed displacements with the displacements measured during the test provides a means of evaluating the cyclic shear resistance of the Rancho Solano #2 clay that was tested in the centrifuge model. This was done by repeated analyses, varying the clay strength until the calculated displacements matched the measured values.

Newmark's method is ideally suited for analyses of displacements for test CLM02, because test CLM02 exhibited sliding block behavior along a well defined shear interface. Newmark's inherent assumption of rigid-block shaking behavior is satisfied, because the clay layer was sandwiched between rigid steel plates, and the thickness of the clay layer was very small relative to its length. Consequently, later refinements to Newmark's method such as those proposed by Makdisi and Seed (1978) and Rathje and Bray (1999) are not needed for the analysis.

Applied Base Motions and Resulting Displacements for Test CLM02

Base acceleration time histories for each of the shaking events in test CLM02 were measured using accelerometers mounted parallel and perpendicular to the inclined steel base plates. The horizontal accelerations needed for the Newmark analyses were calculated by summing the horizontal components of the measured accelerations. The equation used to calculate the horizontal base acceleration time histories for each of the shaking events in test CLM02 is:

$$a_{\text{base horizontal}} = a_{\text{base parallel}} \cdot \cos(\beta) + a_{\text{base perpendicular}} \cdot \sin(\beta), \qquad (7\text{-}1)$$

where: $a_{\text{base horizontal}}$ = calculated horizontal input acceleration at the base of the sliding block model,

$a_{\text{base parallel}}$ = base acceleration measured parallel to the direction of sliding,

$a_{\text{base perpendicular}}$ = base acceleration measured perpendicular to the direction of sliding, and

β = slope angle.

The values of $a_{base\ horizontal}$ calculated using data from the 10.5º slope and the 12º slope, which should be equal because both slopes were subjected to the same shaking, agreed within about 10 %.

In order to use the calculated horizontal base acceleration time histories to compute displacements using Newmark's method, it was necessary to invert the calculated acceleration records using D'Alembert's principle (which allows a dynamic system to be analyzed as an equivalent static system subjected to an inertial force and an inertial torque). The peak horizontal accelerations from the inverted base acceleration records are given in Table 7-1. Table 7-1 also lists the number of applied stress cycles and the shaking-induced displacement recorded for each of the slopes in test CLM02.

Table 7-1: Applied Base Motions and Resulting Displacements for Test CLM02

Event	Peak Horizontal Accel., Downslope, 10.5º Slope [1/(N·g)]	Peak Horizontal Accel., Upslope, 10.5º Slope [1/(N·g)]	Peak Horizontal Accel., Downslope, 12º Slope [1/(N·g)]	Peak Horizontal Accel., Upslope, 12º Slope [1/(N·g)]	No. of Applied Stress Cycles	Disp. for 10.5º Slope	Disp. for 12º Slope
Shake 1	0.05	-0.08	0.06	-0.08	5	0"	0"
Shake 2	0.32	-0.35	0.35	-0.33	5	0"	0.0015"
Shake 3	0.51	-0.60	0.51	-0.54	20	0.014"	0.037"

As shown in Table 7-1, no displacement was observed for either slope as a result of Shake 1, or for the 10.5º slope in Shake 2. Only a very small amount of displacement was observed for the 12º slope during Shake 2. Consequently, the Newmark analyses described in the following sections were only performed for the Shake 3 time histories.

Calculating Yield Acceleration from Undrained Shear Strength

Using Newmark's approach, the yield acceleration is defined as the smallest value of horizontal inertial acceleration that would induce slope failure. The yield acceleration is calculated by determining the horizontal acceleration that corresponds to a factor of safety of

1.0 against sliding, and is assumed to remain constant during shaking. The derivation of the equation for the downslope-directed horizontal yield acceleration using Newmark's approach is shown in Figure 7-1. The equation is:

$$a_{y\ ds} = \frac{S_u A - mgN \cdot \sin(\beta)}{mN \cdot \cos(\beta)}, \tag{7-2}$$

where: $a_{y\ ds}$ = downslope-directed horizontal yield acceleration,
S_u = undrained shear strength,
A = area of the slickensided plane,
m = mass of the sliding block,
g = acceleration of gravity,
N = centrifuge scale factor, and
β = slope angle.

For flat slopes with low friction angles, there is the possibility of upslope slip during shaking. At Newmark's recommendation, this effect is often neglected in practice, since it is conservative to assume that upslope slip will not occur. However, when comparing predicted displacements with measured displacements, this effect should be included for the sake of accuracy. The derivation of the equation for the upslope-directed horizontal yield acceleration using Newmark's approach is shown in Figure 7-2. The equation for the upslope yield acceleration is:

$$a_{y\ us} = \frac{S_u A + mgN \cdot \sin(\beta)}{mN \cdot \cos(\beta)}, \tag{7-3}$$

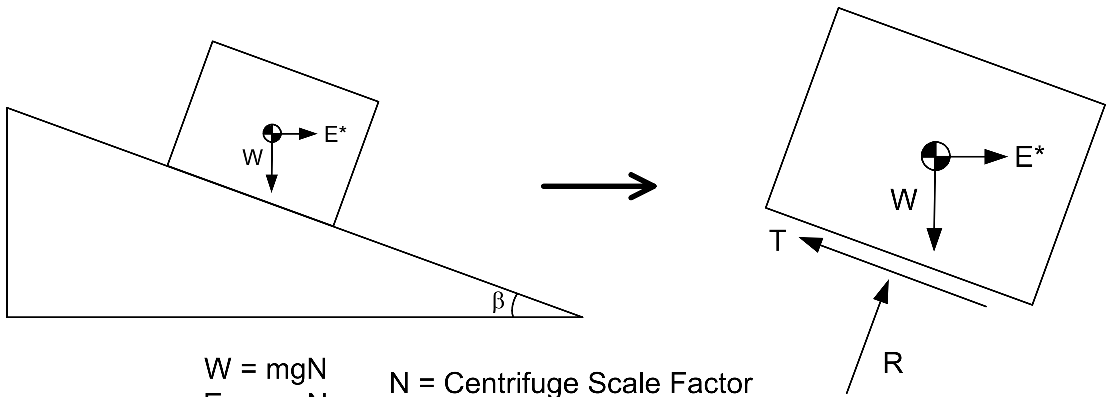
where: $a_{y\ us}$ = upslope-directed horizontal yield acceleration,
S_u = undrained shear strength,
A = area of the slickensided plane,
m = mass of the sliding block,
g = acceleration of gravity,
N = centrifuge scale factor, and
β = slope angle.

For test CLM02, m, g, N, and β are known for both slopes. This allows Equations 7-2 and 7-3 to be used to calculate the downslope and upslope yield accelerations as a function of the undrained shear strength. The resulting equations for yield acceleration can then be used to predict Newmark displacements as a function of undrained shear strength.

Newmark Displacement Analyses

Using Newmark's method, earthquake-induced slope displacements were calculated by double integration of the portion of the acceleration record that is larger than the yield acceleration (Newmark, 1965). Newmark displacement analyses were performed using a numerical integration routine written within the MathCAD computer platform. The numerical integration routine includes a provision for upslope sliding, in the event that the applied dynamic acceleration exceeds the upslope yield acceleration.

Figure 7-3 shows displacements predicted for the 10.5º slope for Shake 3, as a function of undrained shear strength. Figure 7-4 shows displacements predicted for the 12º slope for Shake 3, as a function of undrained shear strength. Figures 7-3 and 7-4 also show the relative displacements measured after Shake 3 for each of the slopes.



$W = mgN$
$E = ma_yN$

N = Centrifuge Scale Factor

a_y = Horizontal Yield Acceleration

*Note: For infinite slope mechanisms, it is more convenient to use components of acceleration that are parallel and perpendicular to the slope

Finding the value of yield acceleration that will cause downslope slip of the block:

$$F = 1 = \frac{S_u A}{T}$$

S_u = Undrained Shear Strength

A = Area of the Slickensided Plane

Apply equilibrium parallel to the sliding plane:

$$\Sigma F_{para} = 0 = mgN \sin(\beta) + ma_yN \cos(\beta) - T$$

$$T = mgN \sin(\beta) + ma_yN \cos(\beta)$$

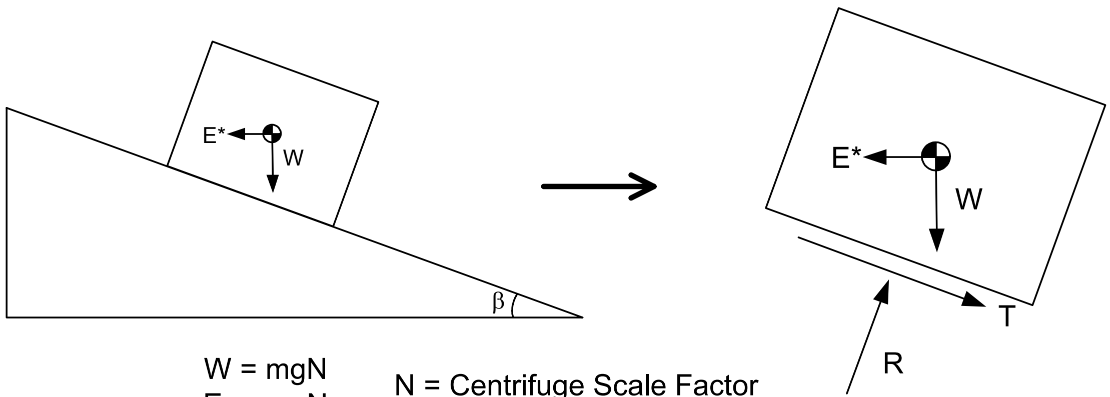
Solving for a_y:

$$S_u A = T$$

$$S_u A = mgN \sin(\beta) + ma_yN \cos(\beta)$$

$$a_y = \frac{S_u A - mgN \sin(\beta)}{mN \cos(\beta)}$$

Figure 7-1. Calculating the yield acceleration that causes downslope slip of the block.



$W = mgN$
$E = ma_yN$

N = Centrifuge Scale Factor

a_y = Horizontal Yield Acceleration

*Note: For infinite slope mechanisms, it is more convenient to use components of acceleration that are parallel and perpendicular to the slope

Finding the value of yield acceleration that will cause upslope slip of the block:

$$F = 1 = \frac{S_u A}{T}$$

S_u = Undrained Shear Strength

A = Area of the Slickensided Plane

Apply equilibrium parallel to the sliding plane:

$$\Sigma F_{para} = 0 = T + mgN \sin(\beta) - ma_yN \cos(\beta)$$

$$T = ma_yN \cos(\beta) - mgN \sin(\beta)$$

Solving for a_y:

$$S_u A = T$$

$$S_u A = ma_yN \cos(\beta) - mgN \sin(\beta)$$

$$a_y = \frac{S_u A + mgN \sin(\beta)}{mN \cos(\beta)}$$

Figure 7-2. Calculating the yield acceleration that causes upslope slip of the block.

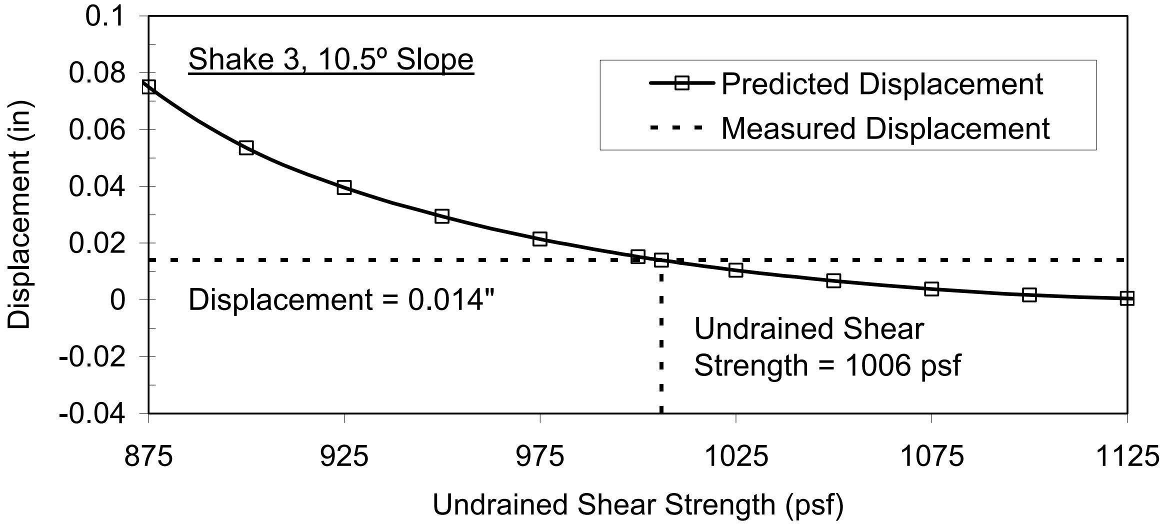


Figure 7-3. Newmark displacements calculated for the 10.5º slope as a function of undrained shear strength.

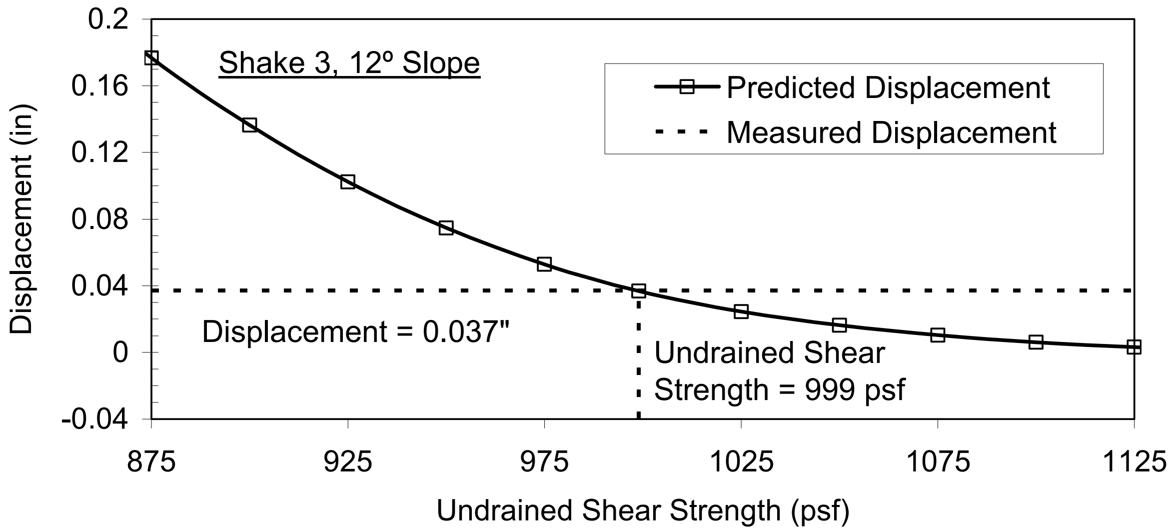


Figure 7-4. Newmark displacements calculated for the 12º slope as a function of undrained shear strength.

As shown in Figures 7-3 and 7-4, an undrained shear strength of approximately 1,000 psf produces good agreement between predicted and measured displacements for both slopes. Figure 7-5 shows the base acceleration, relative velocity and relative displacement predicted by Newmark's method for the 10.5º slope during Shake 3, assuming an undrained shear strength of 1,006 psf. Figure 7-6 shows the base acceleration, relative velocity and relative displacement predicted by Newmark's method for the 12º slope during Shake 3, assuming an undrained shear strength of 999 psf.

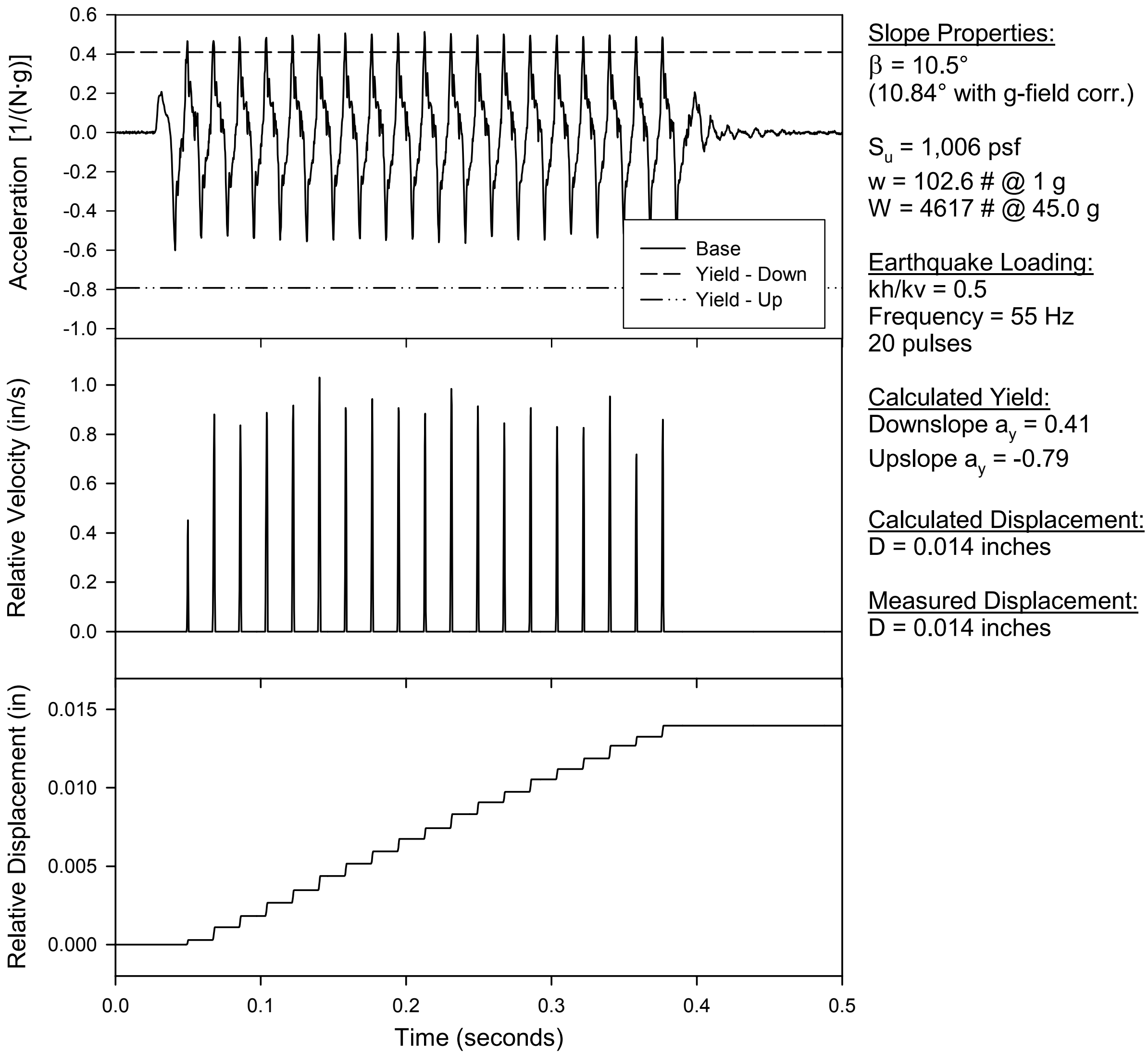


Figure 7-5. Newmark analysis of 10.5º slope for Shake 3.

Note that the details of the displacement responses predicted by Newmark's method (shown in Figures 7-5 and 7-6) are significantly different than the displacement responses that were observed during Shake 3 (shown in Figures 6-23 and 6-24). This difference is due to the assumption of rigid-plastic sliding behavior that is inherent to Newmark's method. Because Newmark's method assumes rigid-plastic sliding behavior, it cannot be used to capture the elastic, hysteretic load-displacement response that was observed during Shake 3 (shown in Figures 6-32 and 6-33), although it does match the post-shaking irrecoverable displacements (Pradel et al., 2005).

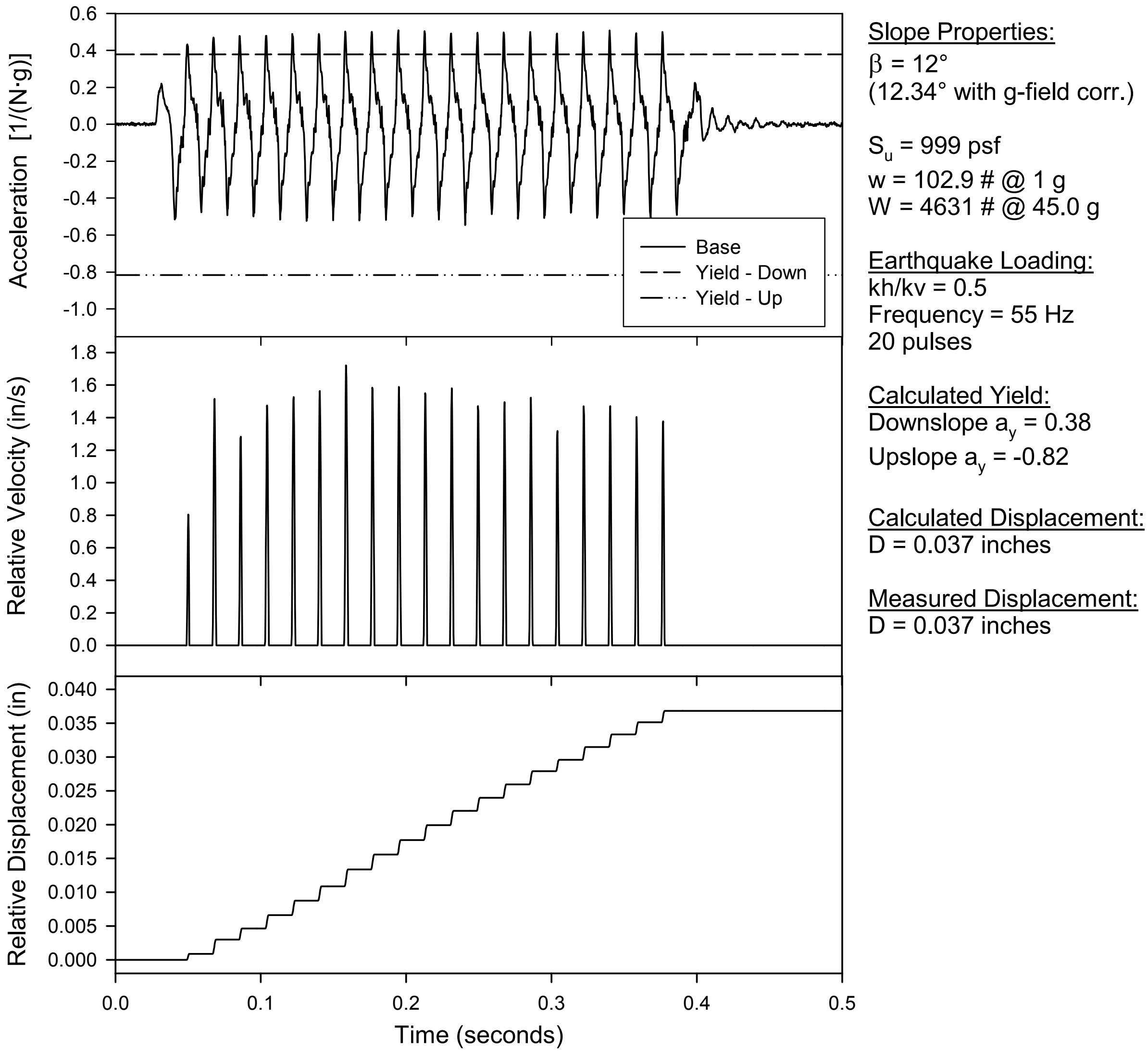


Figure 7-6. Newmark analysis of 12º slope for Shake 3.

Based on test CLM02, an undrained shear strength of 1,000 psf is appropriate for Rancho Solano Clay #2, in order to match slope displacements calculated using Newmark's method with the measured displacements. The calculated undrained strengths can also be expressed using cyclic shear strength ratios, as follows:

$$S_c = \frac{S_u}{\sigma'} = \frac{S_u}{\frac{mgN \cdot \cos(\beta)}{A} - U_o}, \tag{7-4}$$

where: S_c = cyclic shear strength ratio,
S_u = undrained shear strength,
σ' = initial effective normal stress on the slip plane,
m = mass of the sliding block,
g = acceleration of gravity,
N = centrifuge scale factor,
β = slope angle,
A = area of the slickensided plane, and
U_o = intial pore pressure acting on the slickensided plane.

Table 7-2 lists the values that were used to calculate the cyclic shear strength ratios for each slope, and the resulting cyclic strength ratios. A cyclic shear strength ratio between 0.63 and 0.69 is appropriate for Rancho Solano Clay #2.

Table 7-2: Calculated Cyclic Shear Strength Ratios

Slope Being Analyzed	S_u (psf)	mg (lb)	N (unitless)	β (degrees)	A (ft^2)	U_o (psf)	S_c (unitless)
10.5° Slope	1,006	102.6	45	10.84	2.71	79.2	0.63
12° Slope	999	102.9	45	12.34	2.71	216	0.69

<u>Using Simplified Displacement-Based Approaches to Back-Calculate Strength</u>

In engineering practice, simplified displacement-based screening approaches are often used to estimate upper bound earthquake-induced slope displacements. Two commonly used displacement-based screening approaches have been proposed by Newmark (1965) and Hynes-Griffin and Franklin (1984). Both of these screening approaches are semi-empirical, and were developed by performing Newmark analyses for various slope conditions, using a range of earthquake input motions.

Newmark (1965)

Newmark (1965) developed an upper bound estimate for earthquake-induced slope displacements using an empirical curve that bounded the results from his analyses of four earthquake time histories. The proposed upper-bound curve is given by the formula:

$$u = \frac{V^2}{2gN}\left[1 - \frac{N}{A}\right]\left[\frac{A}{N}\right], \qquad (7\text{-}5)$$

where: u = maximum displacement (inches),
V = peak velocity (inches per second),
g = acceleration of gravity (inches per second squared),
N = horizontal acceleration that reduces factor of safety to 1.0 (fraction of g), and
A = peak horizontal acceleration (same units as N).

Cyclic shear strength ratios for Shake 3 were back-calculated for both centrifuge model slopes using Equation 7-5. The results from these simplified Newmark (1965) analyses are given in Table 7-3. Prototype displacements were used to back-calculate the cyclic strength ratios. Peak velocities were determined by integrating the horizontal input acceleration time histories.

Table 7-3: Cyclic Shear Strength Ratios Back-Calculated Using Equation 7-5

Slope Being Analyzed	Prototype Displacement (in)	Peak Horizontal Accel., Downslope, (1/g)	Peak Velocity (in/s)	Calculated N (1/g)	Strength Ratio Based on Equation 7-5 S_c	Strength Ratio From Detailed Numerical Analyses S_c	Diff-erence (%)
10.5º Slope	0.014 x 45 = 0.63	0.51	20.7	0.362	0.63	0.63	0
12º Slope	0.037 x 45 = 1.67	0.51	21.1	0.281	0.62	0.69	11

As shown in Table 7-3, the cyclic strength ratios back-calculated using Newmark's simplified method (Eq. 7-5) agree quite clearly with the strength ratios that were calculated using the detailed Newmark numerical integration approach. The cyclic strength ratios back-calculated using the simplified approach are equal to or smaller than those calculated from the numerical integration, which is contrary to what would be expected, given the upper-bound nature of Newmark's simplified approach, but this difference is not great.

Hynes-Griffin and Franklin (1984)

Hynes-Griffin and Franklin (1984) developed an upper bound estimate for earthquake-induced slope displacements based on the results of Newmark analyses of 348 earthquake motions and six synthetic acceleration time histories. The upper bound curve that envelops all 354 results is shown in Figure 7-7.

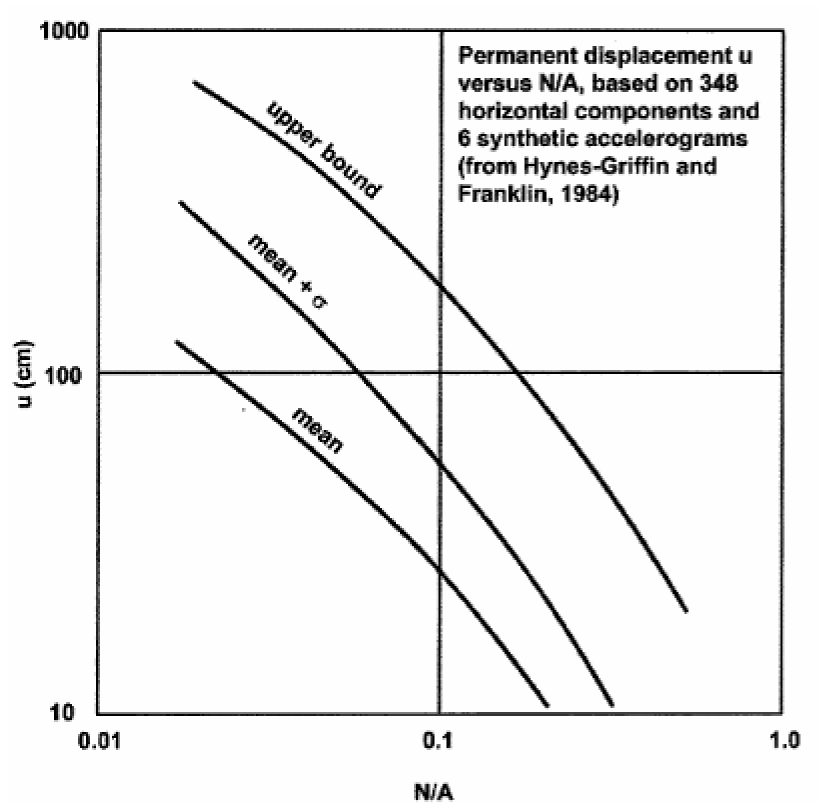


Figure 7-7. Earthquake-induced displacement vs. N/A (Hynes-Grifin and Franklin, 1984).

The prototype displacements that were observed for the 10.5° and the 12° slopes were 1.6 cm and 4.2 cm, respectively. These displacements are outside the range of the chart provided by Hynes-Griffin and Franklin. An empirical equation proposed by Duncan and

Brandon (2005) was used to extrapolate Hynes-Griffin and Franklin's upper bound curve and calculate displacements as a function of N. Their proposed equation is:

$$u = 7\left[\frac{N}{A}\right]^{-1.5}, \tag{7-6}$$

where: u = maximum displacement (cm),
N = horizontal acceleration that reduces factor of safety to 1.0, and
A = peak horizontal acceleration (same units as N).

Cyclic shear strength ratios for Shake 3 were back-calculated for both centrifuge model slopes using Equation 7-6. The results from these simplified Hynes-Griffin and Franklin (1984) analyses are given in Table 7-4. Prototype displacements were used to back-calculate the cyclic strength ratios.

Table 7-4: Cyclic Shear Strength Ratios Back-Calculated Using Equation 7-6

Slope Being Analyzed	Prototype Displacement (cm)	Peak Horizontal Accel., Downslope, (1/g)	Calculated N (1/g)	Strength Ratio Based on Equation 7-5 S_c	Strength Ratio From Detailed Numerical Analyses S_c	Difference (%)
10.5º Slope	0.0356 x 45 = 1.6	0.51	1.36	2.24	0.63	256
12º Slope	0.0940 x 45 = 4.2	0.51	0.72	1.32	0.69	91

As can be seen in Table 7-4, the cyclic strength ratios back-calculated using Hynes-Griffin and Franklin's simplified method result in much higher strength ratios than were calculated using Newmark's numerical integration approach. The difference between these two approaches is due to the fact that Hynes-Griffin and Franklin's method does not involve the actual peak velocity for the particular acceleration time history, but instead implicitly uses high velocities corresponding to the extremes of the 354 cases they considered.

CHAPTER 8: CYCLIC SHEAR STRENGTHS OF SLICKENSIDED SURFACES

The results from the laboratory tests conducted on Rancho Solano Clay #1 and the centrifuge tests conducted on Rancho Solano Clay #2 described in previous chapters provide insight into the cyclic strength that can be mobilized along slickensided surfaces.

Results from Laboratory Test Program on Rancho Solano Clay #1

Chapters 4 and 5 outline the results from the ring shear and direct shear tests that were conducted at Virginia Tech on Rancho Solano Clay #1, Rancho Solano Clay #2, and San Francisco Bay Mud test specimens. Of the three soils tested in the direct shear device, only the slickensided surfaces prepared for Rancho Solano Clay #1 gave drained residual strengths that agreed well with those measured in the Bromhead ring shear device. Consequently, rapid loading and cyclic loading direct shear tests were only performed on Rancho Solano Clay #1.

The loading rates that were used for the direct shear tests on Rancho Solano Clay #1 were as follows:

- The displacement rate for the drained direct shear tests was 0.000123 inches/minute.
- The displacement rate for the fast direct shear tests was 0.048 inches/minute.
- The loading frequency for the cyclic direct shear tests was 0.5 cycles/second.

In order to compare the strengths measured in the monotonic strain-controlled tests with the strengths measured in the cyclic loading tests, it is convenient to express the monotonic displacement rates as stress cycle loading rates. To use this approach, it is necessary to assume that failure occurs in one cycle of monotonic loading. The equivalent loading frequency for the monotonic direct shear tests can then be calculated as follows:

$$\text{Equivalent Frequency} = \frac{\text{\# of Cycles to Failure}}{\text{Time to Failure}} = \frac{1}{\text{Time to Failure}} \qquad (8\text{-}1)$$

For the four slow direct shear tests that were performed at 14.5 psi on Rancho Solano Clay #1, the average time to failure was 638 minutes. Using Equation 8-1, an equivalent loading frequency of 2.6 x 10^{-5} Hz can be used to represent the slow direct shear tests. For the two fast direct shear tests that were performed at 14.5 psi on Rancho Solano Clay #1, the average time to failure was 0.64 minutes. Using Equation 8-1, an equivalent loading frequency of 2.6 x 10^{-2} Hz can be used to represent the fast direct shear tests. An equivalent loading frequency of 8.4 x 10^{-6} Hz was calculated for the Bromhead ring shear tests, for which the average time to failure was 1996 minutes.

Table 8-1 lists the strength ratios that were measured for Rancho Solano Clay #1 during the ring shear and direct shear testing programs. Figure 8-1 shows a plot of the measured strength ratios as a function of equivalent loading frequency.

Table 8-1: Strength Ratios Measured for Rancho Solano Clay #1

Type of Test Performed	No. of Tests Performed @ 14.5 psi	Normal Stress (psi)	Equivalent Loading Frequency (Hz)	Average Measured Strength Ratio
Bromhead Ring Shear Tests	4	14.6	8.4 x 10^{-6}	0.31
Slow Direct Shear Tests	4	14.5	2.6 x 10^{-5}	0.32
Fast Direct Shear Tests	2	14.5	2.6 x 10^{-2}	0.31
Cyclic Direct Shear Tests	8	14.9	0.5	0.66

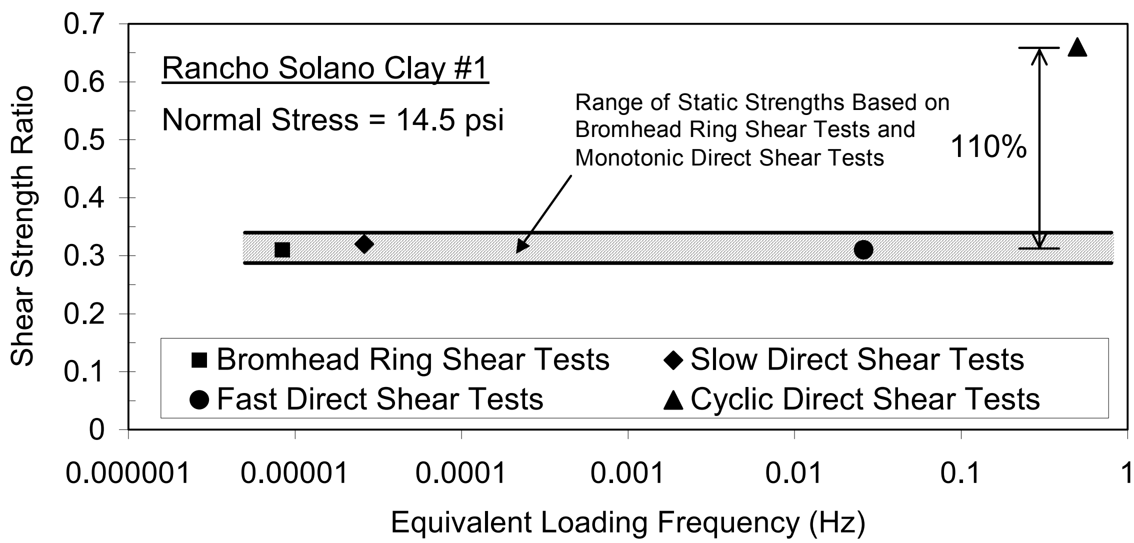


Figure 8-1. Shear strength ratio vs. equivalent loading frequency for Rancho Solano Clay #1.

As shown in Table 8-1 and Figure 8-1, the shear strength ratios measured in the Bromhead ring shear device and in the slow and fast monotonic direct shear tests were essentially the same, indicating that the rate of loading had no effect on the measured strength within the range of loading rates covered in these tests. The cyclic strengths measured in the cyclic direct shear tests were 110% higher than the static strengths measured in the ring shear and direct shear tests, indicating that either the faster rate of loading or the cyclic nature of the loading had a very significant effect on the shearing resistance.

<u>Results from Centrifuge Test Program on Rancho Solano Clay #2</u>

Chapter 6 discusses the results from the centrifuge tests that were conducted at UC Davis on two Rancho Solano Clay #2 test specimens. Chapter 7 describes the Newmark analyses that were performed to determine representative cyclic strengths for Rancho Solano Clay #2 in the centrifuge tests. As discussed in Chapter 6, both slow, static loading and rapid, cyclic loading were performed for the Rancho Solano Clay #2 test specimens during the centrifuge test.

The loading rates used during the centrifuge test are as follows:

- The displacement rate for the slow, static loading events was 0.0005 inches/minute.
- The loading frequency for the rapid, cyclic loading events was 55 cycles/second.

In order to compare the strengths measured in the slow, static loading events with the strengths measured in the rapid, cyclic loading events, equivalent loading frequencies for the static loading events were calculated using Equation 8-1.

For Static Pull #1, the time to failure for the 10.5º slope was 67 minutes, which leads to an equivalent loading frequency of 2.5×10^{-4} Hz. The time to failure for the 12º slope was 18 minutes, which leads to an equivalent loading frequency of 9.3×10^{-4} Hz.

For Static Pull #2, the time to failure for the 12º slope was 7 minutes, which leads to an equivalent loading frequency of 2.4×10^{-3} Hz.

Table 8-2 lists the strength ratios measured for Rancho Solano Clay #2 during centrifuge test CLM02. Figure 8-2 shows a plot of the measured strength ratios as a function of the equivalent loading frequency.

Table 8-2: Strength Ratios Measured for Rancho Solano Clay #2

	No. of Tests Performed Near 10.1 psi Normal Stress	Equivalent Loading Frequency (Hz)	Average Measured Strength Ratio
Bromhead Ring Shear Tests (Interpolated between 7.5 psi & 14.6 psi)	12	8.4×10^{-6}	0.42
Static Pull #1 – 10.5º Slope	1	2.5×10^{-4}	0.46
Static Pull #1 – 12º Slope	1	9.3×10^{-4}	0.42
Shake 3 – 10.5º Slope (from Newmark analysis)	1	55	0.63
Shake 3 – 12º Slope (from Newmark analysis)	1	55	0.69
Static Pull #2 – 12º Slope (after shaking)	1	2.4×10^{-3}	0.55

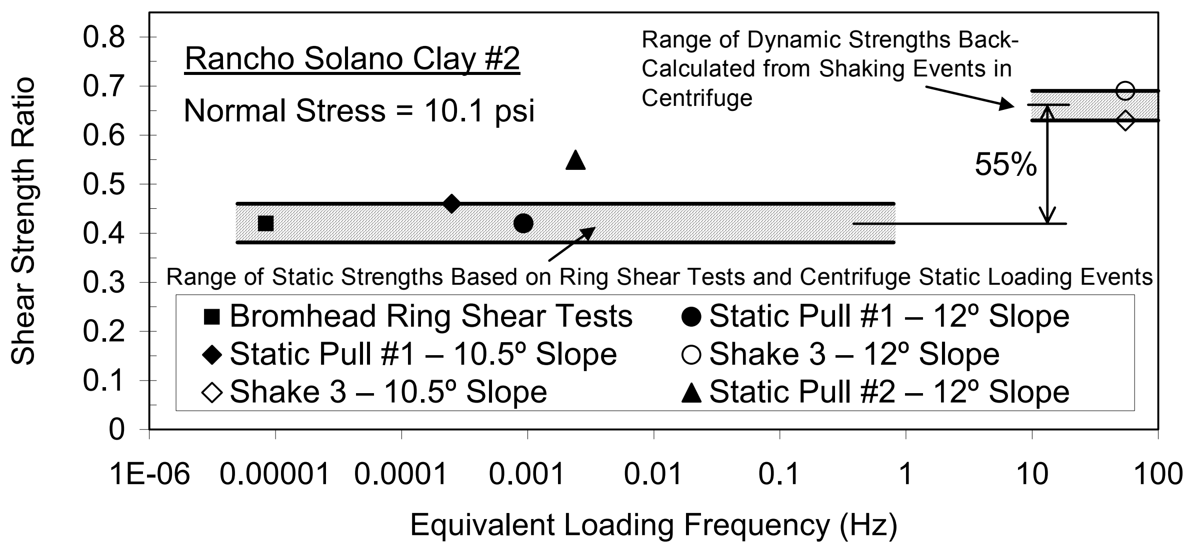


Figure 8-2. Shear strength ratio vs. equivalent loading frequency for Rancho Solano Clay #2.

As shown in Table 8-2 and Figure 8-2, the shear strength ratios measured in the Bromhead ring shear device agreed quite well with the strength ratios measured for both slopes during Static Pull #1. This agreement provides validation for the polishing process that was used to prepare the centrifuge specimens. The cyclic shear resistances back-calculated from centrifuge test CLM02 were 55% higher than the static strengths measured in the Bromhead ring shear tests and the centrifuge static loading events that were performed before the cyclic loading.

Static Pull #2 was performed after Shake 3, and the measured strength ratio for that loading was 30% higher than the strength ratio that was measured during Static Pull #1. This data indicates that the post-shaking shear strength along the slickensided surface was higher than the shear strength before shaking, indicating that the clay had been stiffened and strengthened by the cyclic loading. This increase in strength may be due to disordering of the slickensided shear plane during cyclic loading, which would be consistent with the mechanism proposed by Skempton (1985), Lemos et al. (1985), and Tika et al. (1996) for the significant strength gains observed during rapid monotonic loading on slickensided shear surfaces.

Implications for Design Practice

Both the laboratory tests conducted on Rancho Solano Clay #1 and the centrifuge tests conducted on Rancho Solano Clay #2 show that the cyclic shear resistance that can be mobilized along slickensided surfaces is significantly higher than the drained shear resistance that is available under static loading conditions. For cyclic loading at frequencies of 0.5 Hz and 55 Hz, the measured cyclic strengths for Rancho Solano Clay #1 and Rancho Solano Clay #2 were 110% and 55% higher, respectively, than the corresponding static drained residual strengths for these soils. This agrees with the results of cyclic ring shear tests performed on 16 different soils by Yoshimine et al. (1999), who reported cyclic strengths 20% to 100% larger than the slow residual strengths, for tests conducted at cyclic load frequencies of 0.5 Hz, 1.0 Hz, and with actual earthquake time histories.

As noted by Blake et al. (2002), the current state of practice is to use drained residual shear strengths when performing dynamic analyses of slopes that contain slickensided slip surfaces. Because the actual dynamic shearing resistance may be significantly larger than the drained residual shear strength, this design approach can result in overly conservative slope designs.

Taken together, the laboratory tests outlined in this report and the dynamic ring shear tests performed by Yoshimine et al. (1999) provide justification for using cyclic strengths that are larger than the drained residual shear strength when performing seismic slope stability analyses of slopes that contain slickensided slip surfaces. To reduce the conservatism that is built into the current state of practice, it seems logical that dynamic slope stability analyses be performed using a cyclic shear resistance that is at least 20% larger than the drained residual shear strength.

The centrifuge tests performed on Rancho Solano Clay #2 also showed that the post-shaking shear strength along the slickensided surface is higher than the shear strength before shaking. This data indicates that post-earthquake stability of slickensided clay slopes should not be an issue of significant concern. This is consistent with what has been observed in the field by other researchers (Pradel et al., 2005).

Further research to develop a better understanding of the factors that influence the magnitude of cyclic shear resistance could be of great value. Of particular utility could be research that would identify a relationship between index properties, clay mineralogy, and the cyclic shear resistance.

CHAPTER 9: SUMMARY AND CONCLUSIONS

The objective of the study outlined in this report was to investigate, through laboratory strength tests and centrifuge model tests, the shearing resistance that can be mobilized on slickensided rupture surfaces in clay slopes during earthquakes. Test results show that the cyclic shear resistance that can be mobilized along slickensided surfaces is higher than the drained shear resistance that is applicable for static loading conditions. These test results, coupled with a review of existing literature, provide justification for using cyclic strengths that are at least 20% larger than the drained residual shear strength for analyses of seismic stability of slickensided clay slopes. This represents a departure from the current state of practice, which is to use the drained residual shear strength as a "first-order approximation of the residual strength friction angle under undrained and rapid loading conditions" (Blake et al., 2002).

A summary of the work accomplished in this study, the conclusions drawn from the tests that were performed, and recommendations for further research are provided in the following sections.

Summary of Work Accomplished

The work accomplished in this study is as follows:

1.) A literature review was performed to summarize the results of previous research on the shear behavior of slickensided soils. Additional literature related to centrifuge model testing and seismic slope stability analysis methods was also reviewed, because of its relevance to the research program described in this report.

2.) Three natural clay soils (Rancho Solano Clay #1, Rancho Solano Clay #2, and San Francisco Bay Mud) were obtained, classified, mixed and remolded to ensure uniformity, and consolidated so they could be used in the testing program described in this report.

3.) Slow strain-controlled ring shear tests were performed on Rancho Solano Clay #1, Rancho Solano Clay #2, and San Francisco Bay Mud test specimens to measure the drained residual strength along slickensided discontinuities. Fast strain-controlled ring shear tests were performed on Rancho Solano Clay #1 test specimens to measure the fast residual strength.

4.) Slow strain-controlled direct shear tests were performed on Rancho Solano Clay #1, Rancho Solano Clay #2, and San Francisco Bay Mud test specimens to measure the drained residual strength along artificially prepared slickensided surfaces. Fast strain-controlled direct shear tests were performed on slickensided Rancho Solano Clay #1 test specimens to measure the fast residual strength. Cyclic stress-controlled direct shear tests were performed on slickensided Rancho Solano Clay #1 test specimens to measure the cyclic shear resistance.

5.) Slow strain-controlled triaxial tests were performed on Rancho Solano Clay #1 test specimens to measure the drained residual strength along artificially prepared slickensides.

6.) Centrifuge tests were performed on Rancho Solano Clay #2 sliding block models to measure the static and dynamic shear resistance along slickensided surfaces.

7.) Newmark analyses were performed to back-calculate the dynamic shear resistance for shaking events in centrifuge test CLM02. Comparison of the computed and measured displacements provided a means of evaluating the cyclic shear resistance of the Rancho Solano #2 clay.

8.) A comparison of the measured static and cyclic strengths was performed to determine the shear strength that should be used in seismic stability analyses of slopes that contain slickensided surfaces.

Conclusions

Ring Shear Testing Program:

The conclusions reached as a result of the ring shear tests that were conducted are as follows:

1.) In the Bromhead ring shear device, significant amounts of friction are developed at large shear displacements, due to the extrusion and entrapment of clay particles between the top platen and the side walls of the specimen container. Minimizing the effect of wall friction is essential for accurate measurements of the drained residual strength. Wall friction in the standard Bromhead ring shear device can be reduced by modifying the ASTM test procedure to reduce top platen settlement into the specimen container. The recommended modifications include: preparing ring shear specimens at the plastic limit instead of the liquid limit, avoiding the use of a rapid preshearing stage, and performing only one shear test per specimen (no multistage testing).

2.) In the Bromhead ring shear device, the magnitude of wall friction that is developed can be reduced to an insignificant level by beveling the inside and outside walls of the top platen. This allows the top platen to settle into the specimen container without clay particle entrapment, resulting in significantly more accurate measurements of drained residual shear strength.

3.) Provided that the effect of wall friction is addressed, the Bromhead ring shear device is an excellent tool for the measurement of drained residual shear strengths. Very little scatter was observed in the measured strength data, and good agreement was achieved between the strengths measured in the two different ring shear devices. The resulting drained residual strength envelopes for the three soils that were tested are curved, which agrees well with test data collected by other researchers.

4.) The Bromhead ring shear device is not a useful tool for measuring fast residual strengths. During the fast shear tests, a cyclic increase and decrease in measured shear resistance was observed, which made it impractical to select a fast residual shear resistance for the soil. This cyclic increase and decrease in shear stress is believed to be a machine effect rather than a soil behavior phenomenon, and is likely caused by wobbling of the top platen during shear. The device might be made useful for measuring fast residual strengths by modifying the design.

Direct Shear Testing Program:

The conclusions reached as a result of the direct shear tests that were conducted are as follows:

1.) It is much more difficult than expected at the beginning of this research program to prepare slickensided surfaces that exhibit drained residual strength behavior. For Rancho Solano Clay #2 and San Francisco Bay Mud, neither wet nor dry polishing techniques gave direct shear test results that agreed with the residual strengths measured in the Bromhead ring shear device. This result is unsatisfactory, and further research is necessary to identify why the direct shear test results deviated so significantly from the ring shear test results. Until the reason for this deviation is more clearly identified, the use of artificially prepared slickensides is not recommended for use in geotechnical engineering practice.

2.) For some soils, it is possible to prepare slickensided surfaces that behave as would be expected, based on results of Bromhead ring shear tests. Drained direct shear tests performed on wet polished Rancho Solano Clay #1 test specimens gave residual strengths that agreed well with those measured in the Bromhead ring shear device. This provides experimental validation for the use of the wet polishing method with Rancho Solano Clay #1.

3.) An increase in monotonic shear rate from 0.0001 in/min to 0.05 in/min does not produce significant changes in the shear strength measured along preformed slickensided surfaces. In this range of loading rates, noise in the data obscures any actual changes in strength that may occur.

4.) The cyclic shear resistance that can be mobilized along slickensided surfaces is significantly higher than the drained shear resistance that is available under static loading conditions. Stress-controlled cyclic direct shear tests gave cyclic strengths that were 110% higher than the measured static shear strengths for Rancho Solano Clay #1.

Triaxial Testing Program:

The conclusions reached as a result of the triaxial shear tests that were conducted are as follows:

It is more difficult to use triaxial tests than direct shear tests to measure the shear strength along preformed slickensided discontinuities. Significant obstacles encountered during the triaxial testing program include:

- Difficulties with the effect of end platen restraint on specimens that fail along a well-defined failure plane,
- Uncertainties involving the appropriate area correction and membrane correction to use when reducing the triaxial data, and
- Long test times for consolidated-drained triaxial tests, which has tied up equipment and made it difficult to run the desired number of triaxial tests in a timely fashion.

Because of the difficulties encountered, the triaxial test is not recommended for future testing of this type. Due to the uncertainty surrounding the triaxial test results at this time, useful conclusions cannot be drawn from the triaxial test data regarding the residual strength behavior of pre-formed slickensided surfaces.

Centrifuge Testing Program:

The conclusions reached as a result of the centrifuge tests that were conducted are as follows:

1.) The polishing process used to prepare the slickensided surfaces in the centrifuge test on Rancho Solano Clay #2 was successful. When the models were dissected, the slickensided surfaces still had the shiny look associated with slickensides. When sheared slowly prior to shaking, the shear stress ratios measured for the slickensided surfaces agreed with those measured in the Bromhead ring shear device, providing validation for the polishing process that was used to prepare the centrifuge specimens.

2.) The shear resistance mobilized during cyclic loading was significantly larger than the drained residual strength of the soil. A fundamental and complete understanding of the magnitude of this strength gain cannot be obtained from the centrifuge data alone, because it is not possible to isolate the effect of cyclic loading from the effect of the rate at which the cyclic loading occurred.

3.) Both the static and dynamic shear displacements were concentrated along the pre-formed slickensided plane. Even during cyclic loading, the pre-formed slickensided surface had much lower shearing resistance than the surrounding heavily-overconsolidated clay soil.

4.) Cyclic loading caused a positive pore pressure response in the soil surrounding the slickensided plane. From the recorded pore pressure data, it was not clear whether this pore pressure increase was caused by shearing along the slickensided plane, by stress mobilization in the soil surrounding the slickensided plane, or by some sort of boundary effect at the soil/steel plate interface.

5.) A catastrophic sliding failure did not occur, even during a very large shaking event.

6.) The post-shaking shear resistance was higher than the resistance prior to shaking.

7.) No displacements of the slopes occurred after shaking stopped.

<u>Newmark Analyses</u>

The conclusions reached as a result of the Newmark analyses that were performed are as follows:

1.) Newmark's method can be used to back calculate cyclic strengths. To use this approach, the earthquake time history and the earthquake-induced slope displacement must be known. In this approach, a range of assumed shear strengths are used to calculate a range of yield accelerations and displacements for the slope. Comparison of the calculated displacements with the observed slope displacement provides a means for evaluating the cyclic shear resistance.

2.) The cyclic shear resistance that can be mobilized along slickensided surfaces is significantly higher than the drained shear resistance applicable to static loading conditions. Cyclic strengths back-calculated from the significant shaking events in centrifuge test CLM02 were 55% higher than the measured static shear strengths.

3.) Newmark's simplified method (Newmark, 1965) can be used to back-calculate cyclic strengths. The strength ratios back-calculated using Newmark's simplified method agree well with the strength ratios that were calculated using the detailed Newmark numerical integration approach.

4.) Hynes-Griffin and Franklin's method (Hynes-Griffin and Franklin, 1984) is simpler than Newmark's simplified method and does not require knowledge of peak velocity, but it results in back-calculated dynamic strengths that are very high. The cyclic strength ratios back-calculated using Hynes-Griffin and Franklin's simplified method are much higher than those calculated using the detailed Newmark numerical integration approach or the simplified Newmark approach. The difference between these two approaches is due to the fact that Hynes-Griffin and Franklin's method does not involve the actual peak velocity for a particular acceleration time history, but instead implicitly uses high velocities corresponding to the extremes of the 354 cases they considered.

The Dynamic Behavior of Slickensided Surfaces:

The primary conclusion reached during the studies performed in this report is:

The cyclic shear resistance that can be mobilized along slickensided surfaces is significantly higher than the drained shear resistance that is applicable to static loading conditions.

This conclusion is supported by the work that has been performed by other researchers, most notably Yoshimine et al. (1999). It represents a departure from the current state of practice, which is to use the drained residual shear strength as a "first-order approximation of the residual strength friction angle under undrained and rapid loading conditions" (Blake et al., 2002). To reduce the conservatism that is built into the current state of practice, it seems logical that seismic slope stability analyses be performed using cyclic strengths at least 20% larger than the drained residual shear strength.

Recommendations for Further Research

Based on the findings of this experimental study, the following recommendations are made for areas of further research:

1.) Further research is needed to identify why the drained direct shear test results deviated so significantly from the Bromhead ring shear results (for two of the soils that were tested). From the widely varying strengths that were observed, it is clear that some sort of fundamental change in the shearing interface occurred when different polishing procedures were used to prepare the direct shear specimens. Microscopic studies of the shear interface after polishing might provide insight into how different polishing procedures affect the nature of the shear interface.

2.) Further research is needed to develop a cost-effective method for directly measuring the dynamic shear resistance along slickensided surfaces, for use in geotechnical engineering practice. Of particular value would be development of laboratory test equipment that could be used to measure the cyclic shear resistance along slickensided surfaces, for use in seismic slope stability analyses. Development of a simple ring shear device that can apply both static and cyclic loading would be extremely useful.

3.) Further research is needed to explore the effects of clay fraction and clay mineralogy on the cyclic shear strength of slickensided surfaces. Strong correlations between clay fraction and clay mineralogy exist for the drained residual strength of clayey soils, so it seems likely that some sort of parallel relationship would exist for the cyclic strength. A review of the currently available cyclic strength data for slickensided surfaces shows no discernable trends or correlations with clay fraction or clay mineralogy. If a relationship between clay fraction, clay mineralogy, and cyclic shear strength can be established, its discovery would be extremely beneficial to practicing engineers.

4.) Further research is needed to broaden the existing cyclic strength data set for slickensided clayey soils. As more data is gathered, identification of trends between clay fraction, clay mineralogy, and cyclic shear strength may become more clear.

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APPENDIX A

RING SHEAR DATA

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	5/20/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	5/27/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	25.8	(g)
Wt. of Dry Soil + Tare	20.9	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	13.9	(g)
Wt. of Dry Soil	9.0	(g)
Water Content, w%	54.4	(%)
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	Yes	0.584	in/min
Multistage Loading or New Specimen for Each Point?		Multistage Loading	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
110.7	115	0.80	0.06				
304.3	214	1.48	0.11				
497.9	313	2.17	0.16				
1002.1	570	3.96	0.29				
2001.8	1082	7.51	0.54				
4001.0	2104	14.61	1.05				
8000.5	4149	28.81	2.07	1.68	0.00238	1.71	0.00235

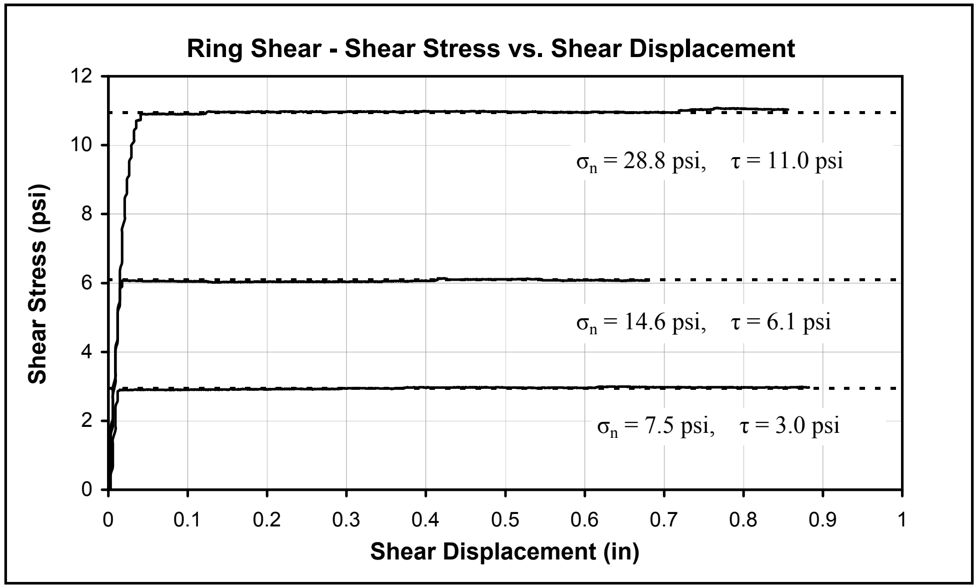
Minimum calc. shear rate =	0.00235	in/min.
Estim. failure displacement =	0.2	in

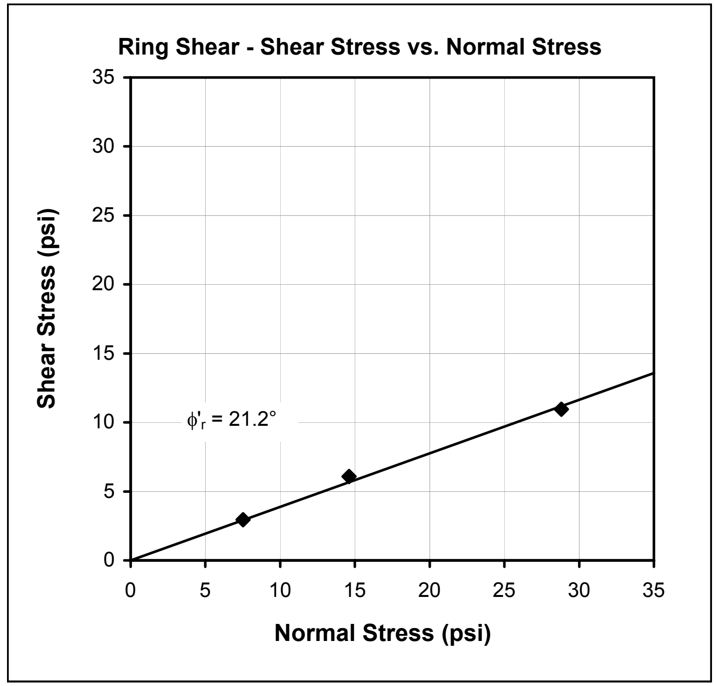
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

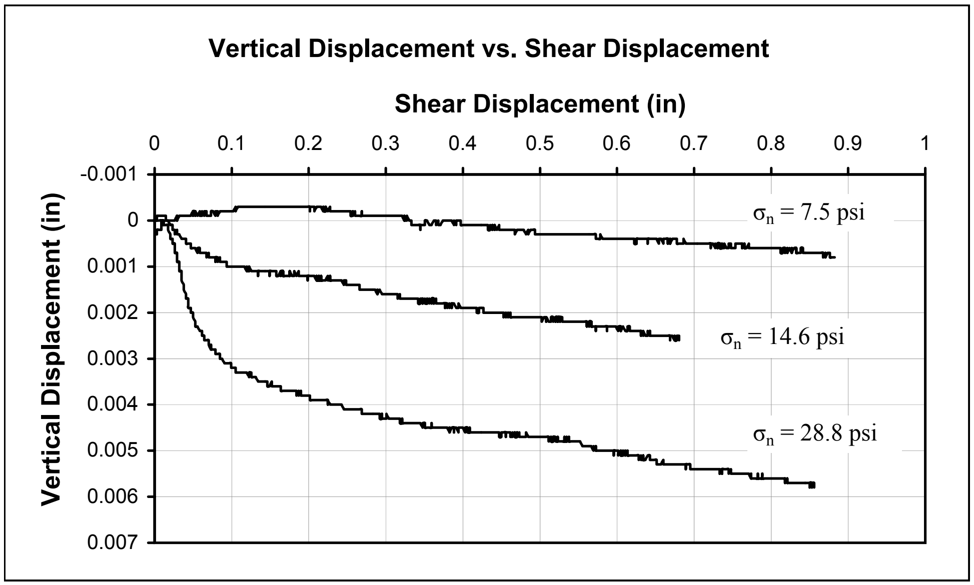
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-052003-1	2001.8	1082	7.51	0.54	425	2.95	0.21	0.0008
R1-052003-2	4001.0	2104	14.61	1.05	877	6.09	0.44	0.0026
R1-052003-3	8000.5	4149	28.81	2.07	1577	10.95	0.79	0.0057

Notes: Specimen was remolded at 1.2 times the LL
No special technique was used to minimize wall friction
This specimen was not pushed through the #40 sieve

ϕ'_r = 21.2 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Michael Wanger
Data Reduced By:	Chris Meehan
Test Started On:	5/20/03
Test Finished On:	5/27/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/3/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	6/10/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	38.0	(g)
Wt. of Dry Soil + Tare	29.0	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	26.1	(g)
Wt. of Dry Soil	17.1	(g)
Water Content, w%	52.6	(%)
Wet Weight of Entire Specimen	35.3	(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	Yes	0.584	in/min
Multistage Loading or New Specimen for Each Point?		Multistage Loading	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
110.7	115	0.80	0.06				
304.3	214	1.48	0.11				
497.9	313	2.17	0.16				
1002.1	570	3.96	0.29				
2001.8	1082	7.51	0.54				
4001.0	2104	14.61	1.05				
8000.5	4149	28.81	2.07	1.5	0.00267	0.8	0.00494

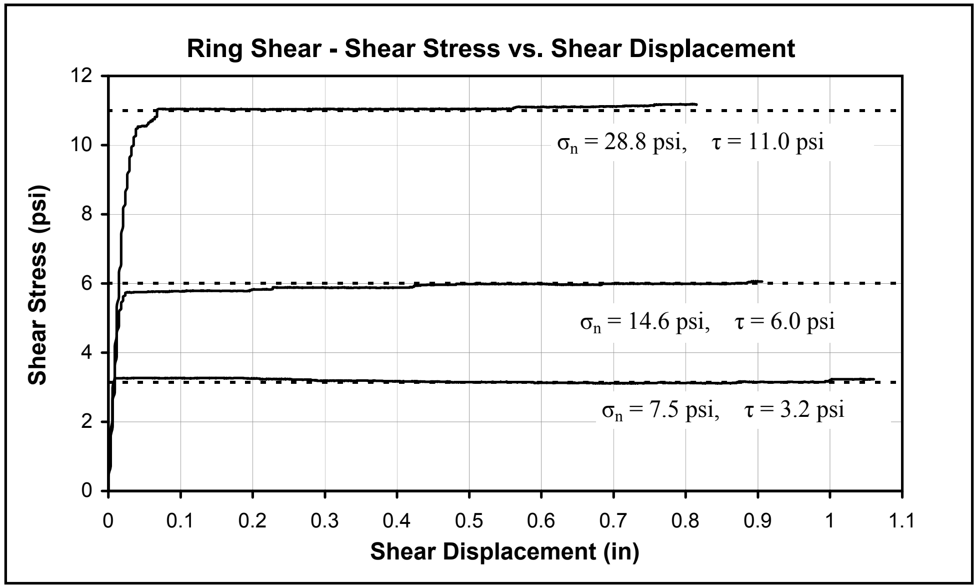
Minimum calc. shear rate =	0.00267	in/min.
Estim. failure displacement =	0.2	in

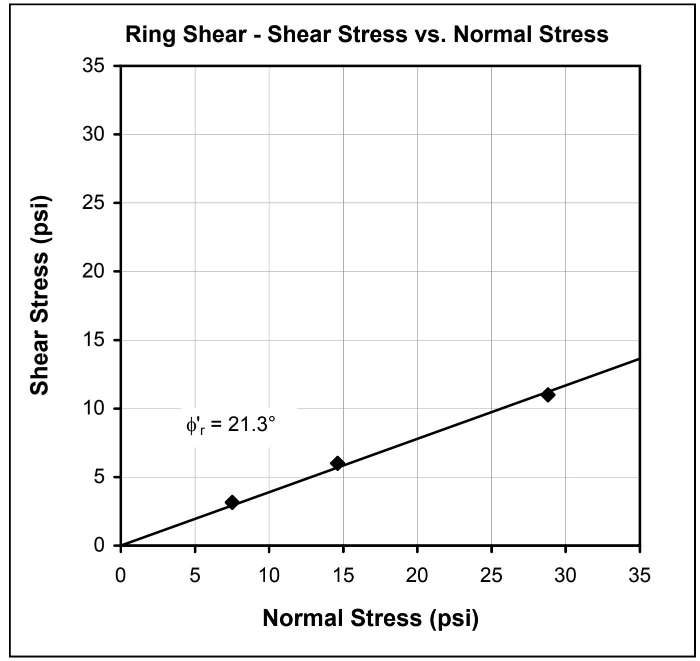
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

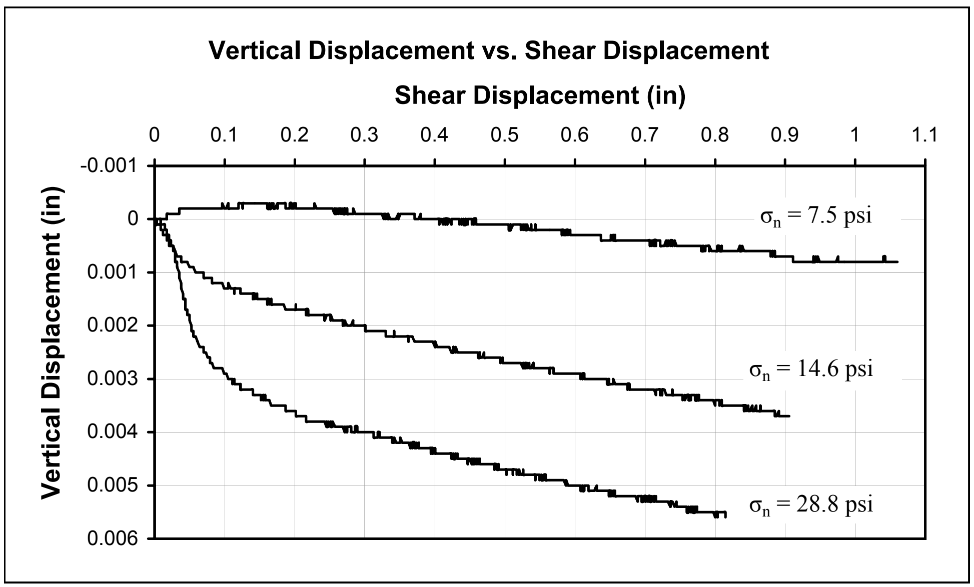
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-060303-1	2001.8	1082	7.51	0.54	454	3.15	0.23	0.0008
R1-060303-2	4001.0	2104	14.61	1.05	864	6.00	0.43	0.0037
R1-060303-3	8000.5	4149	28.81	2.07	1584	11.00	0.79	0.0056

Notes: Specimen was remolded at 1.2 times the LL
No special technique was used to minimize wall friction
This specimen was pushed through the #40 sieve

ϕ'_r = 21.3 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Mike Wanger
Data Reduced By:	Chris Meehan
Test Started On:	6/3/03
Test Finished On:	6/10/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/10/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	6/17/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	32.4	(g)
Wt. of Dry Soil + Tare	25.1	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	20.5	(g)
Wt. of Dry Soil	13.2	(g)
Water Content, w%	55.3	(%)
Wet Weight of Entire Specimen	37.0	(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	Yes	0.584	in/min
Multistage Loading or New Specimen for Each Point?		Multistage Loading	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
110.7	115	0.80	0.06				
304.3	214	1.48	0.11				
497.9	313	2.17	0.16				
1002.1	570	3.96	0.29				
2001.8	1082	7.51	0.54				
4001.0	2104	14.61	1.05				
8000.5	4149	28.81	2.07	2.0	0.00200	1.2	0.00331

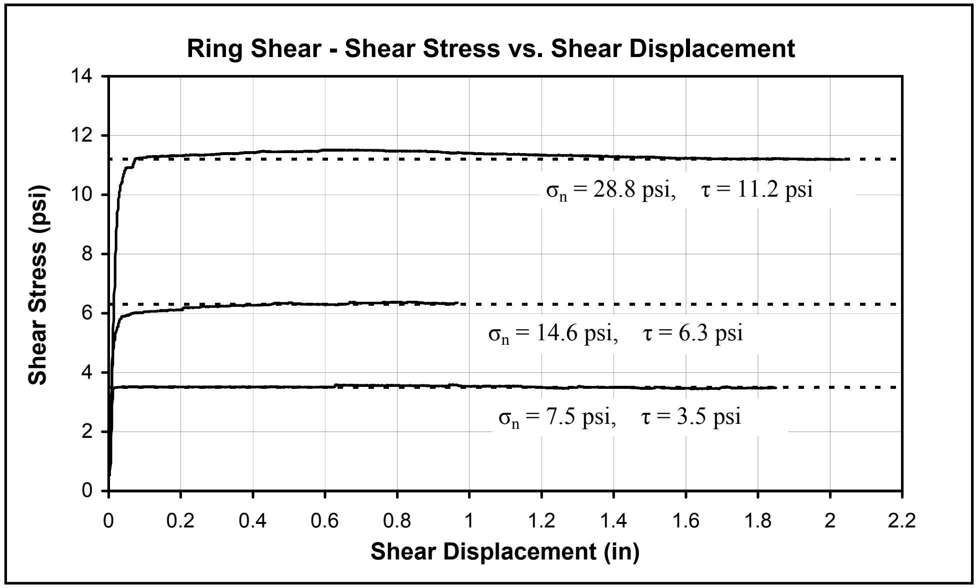
Minimum calc. shear rate =	0.00200	in/min.
Estim. failure displacement =	0.2	in

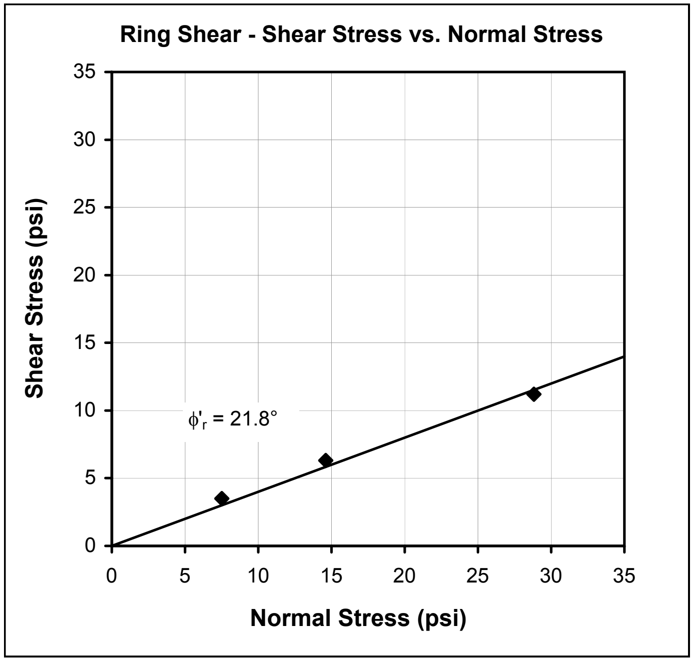
Test performed at shear rate =	0.000709	in/min.
Test performed at shear rate =	0.024	deg/min.

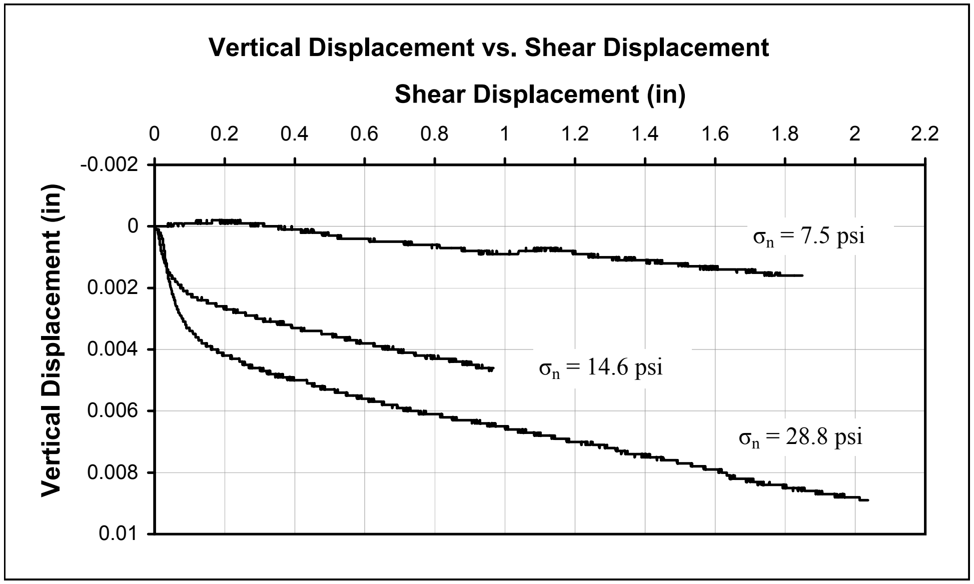
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-061003-1	2001.8	1082	7.51	0.54	504	3.50	0.25	0.0016
R1-061003-2	4001.0	2104	14.61	1.05	907	6.30	0.45	0.0046
R1-061003-3	8000.5	4149	28.81	2.07	1613	11.20	0.81	0.0089

Notes: Specimen was remolded at 1.2 times the LL
No special technique was used to minimize wall friction
This specimen was pushed through the #40 sieve

ϕ'_r = 21.8 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Michael J Wanger
Data Reduced By:	Chris Meehan
Test Started On:	6/10/03
Test Finished On:	6/17/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/19/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	6/26/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	31.5	(g)
Wt. of Dry Soil + Tare	24.5	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	19.6	(g)
Wt. of Dry Soil	12.6	(g)
Water Content, w%	55.6	(%)
Wet Weight of Entire Specimen	37.8	(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared Yes	0.584	in/min
Multistage Loading or New Specimen for Each Point?	Multistage Loading	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
110.7	115	0.80	0.06				
304.3	214	1.48	0.11				
497.9	313	2.17	0.16				
1002.1	570	3.96	0.29				
2001.8	1082	7.51	0.54				
4001.0	2104	14.61	1.05				
8000.5	4149	28.81	2.07	3.3	0.0012	2.5	0.0016

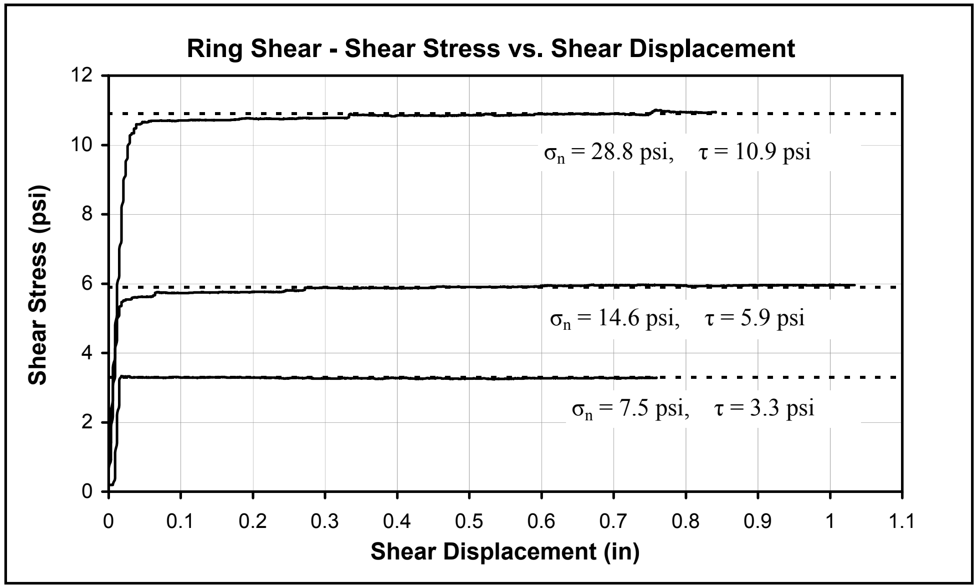
Minimum calc. shear rate =	0.0012	in/min.
Estim. failure displacement =	0.2	in

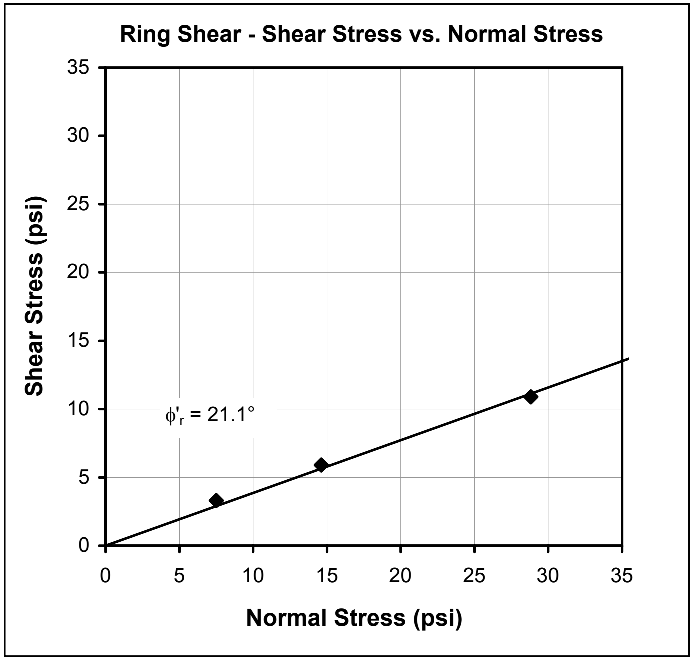
Test performed at shear rate =	0.000709	in/min.
Test performed at shear rate =	0.024	deg/min.

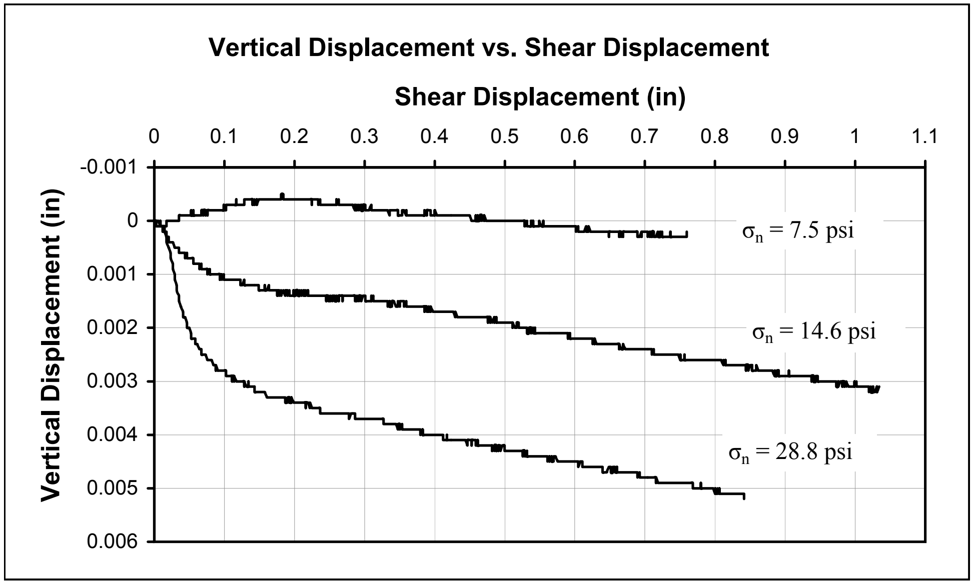
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-061903-1	2001.8	1082	7.51	0.54	475	3.30	0.24	0.0003
R1-061903-2	4001.0	2104	14.61	1.05	850	5.90	0.42	0.0032
R1-061903-3	8000.5	4149	28.81	2.07	1570	10.90	0.78	0.0052

Notes: Specimen was remolded at 1.2 times the LL
No special technique was used to minimize wall friction
This specimen was pushed through the #40 sieve

$\phi'_r =$ 21.1 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Michael J Wanger
Data Reduced By:	Chris Meehan
Test Started On:	6/19/03
Test Finished On:	6/26/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	7/15/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	7/18/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	36.5	(g)
Wt. of Dry Soil + Tare	31.1	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	24.6	(g)
Wt. of Dry Soil	19.2	(g)
Water Content, w%	28.1	(%)
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1002.1	570	3.96	0.29	Shear rate based on data from earlier tests.			
2001.8	1082	7.51	0.54				
4001	2104	14.61	1.05				
8000	4149	28.81	2.07				
16000	8239	57.22	4.12				
28000	14375	99.83	7.19				

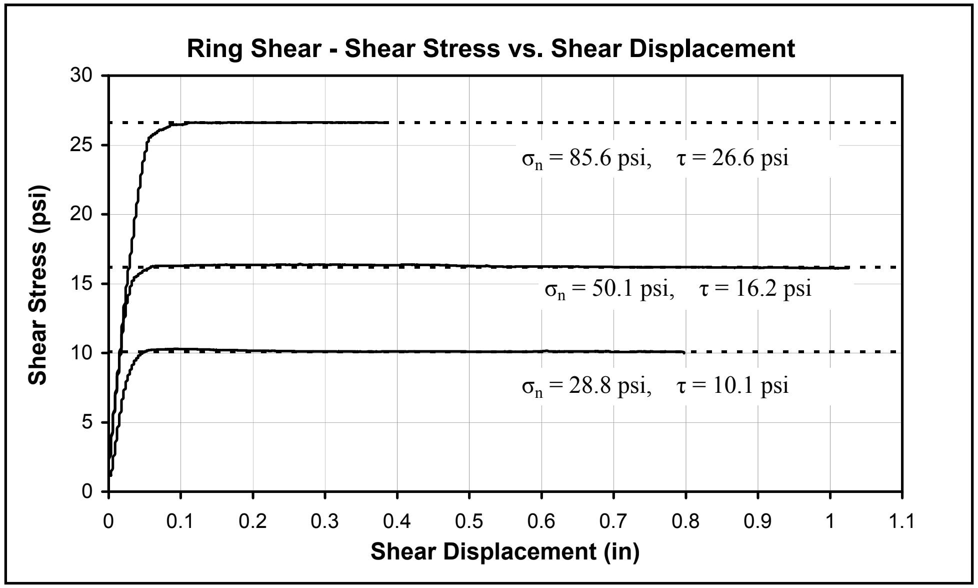
Minimum calc. shear rate =		in/min.
Estim. failure displacement =	0.2	in

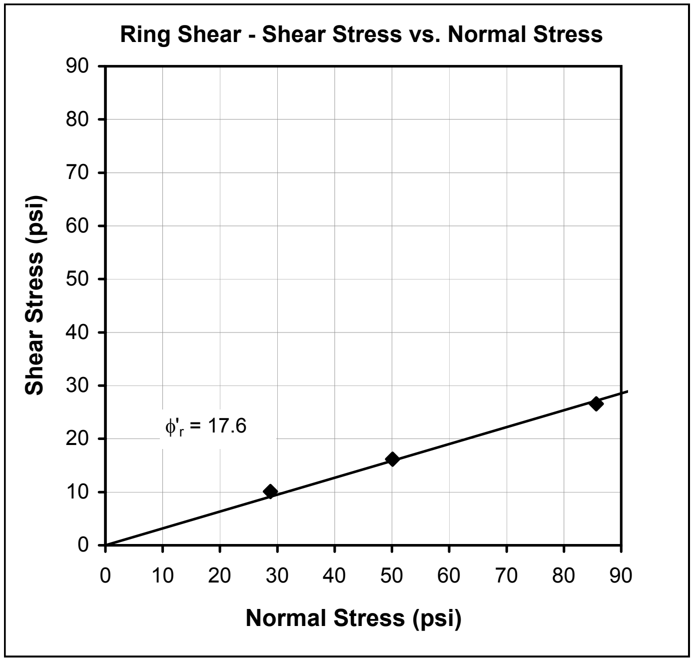
Test performed at shear rate =	0.000709	in/min.
Test performed at shear rate =	0.024	deg/min.

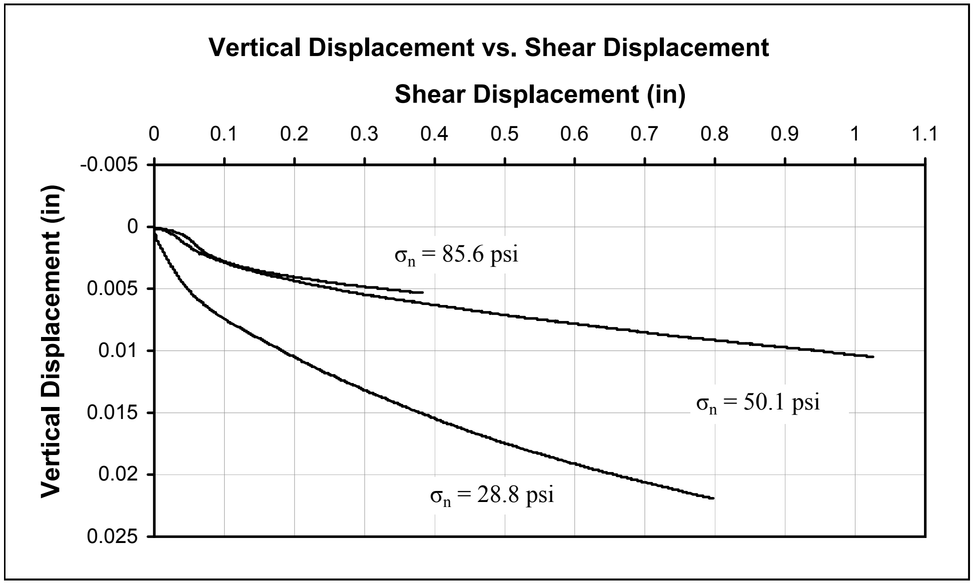
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-071503-1	8000	4149	28.81	2.07	1454	10.10	0.73	0.0219
R2-071503-2	14000	7216	50.11	3.61	2333	16.20	1.17	0.0105
R2-071503-3	24000	12330	85.62	6.16	3830	26.60	1.92	0.0053

Notes: Specimen was remolded at LL, and pushed through the #40 sieve
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 17.6 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Mike Wanger
Data Reduced By:	Chris Meehan
Test Started On:	7/15/03
Test Finished On:	7/18/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	7/19/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	7/24/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	39.7	(g)
Wt. of Dry Soil + Tare	33.7	(g)
Wt. of Tare	11.9	(g)
Wt. of Moist Soil	27.8	(g)
Wt. of Dry Soil	21.8	(g)
Water Content, w%	27.5	(%)
Wet Weight of Entire Specimen	26.6	(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
8000	4149	28.81	2.07				
14000	7216	50.11	3.61				
24000	12330	85.62	6.16				

Shear rate based on data from earlier tests.

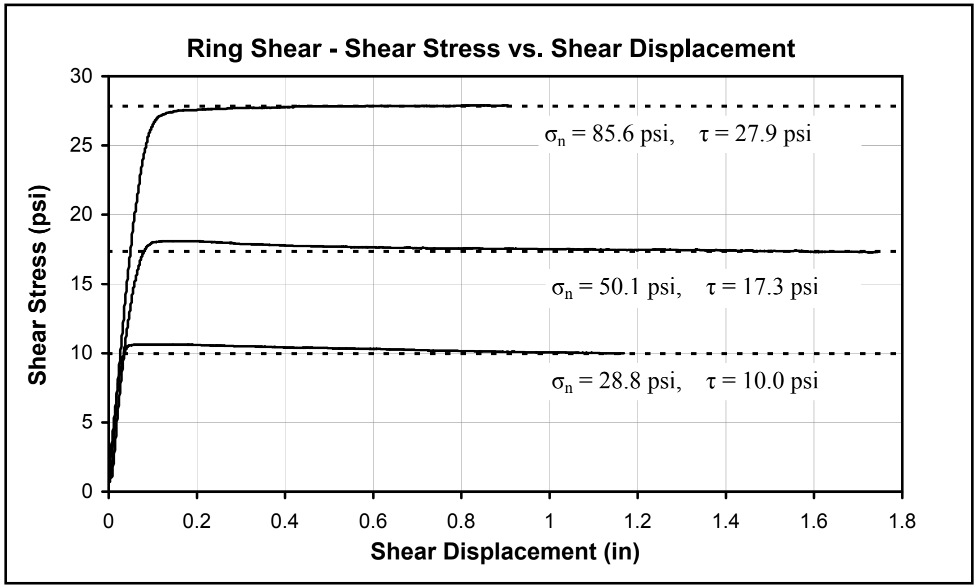
Minimum calc. shear rate =		in/min.
Estim. failure displacement =	0.2	in

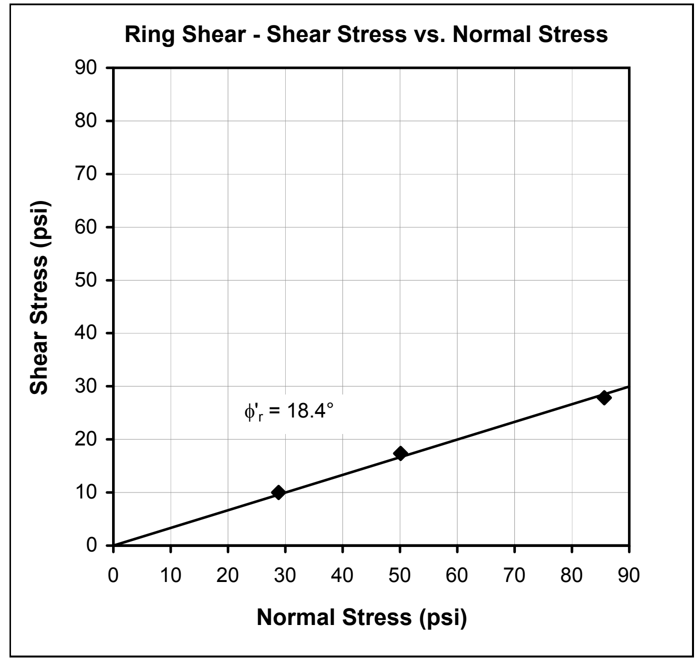
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

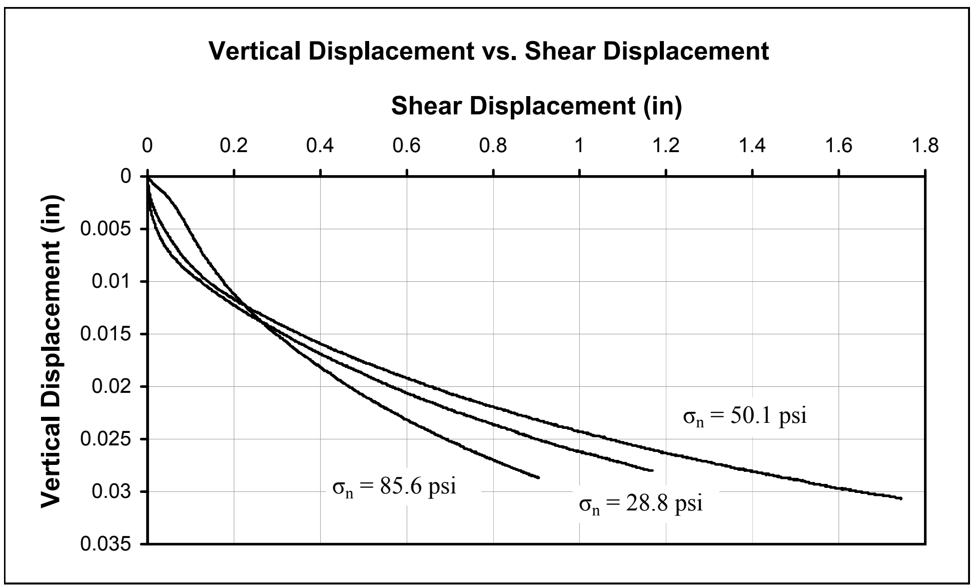
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-071903-1	8000	4149	28.81	2.07	1437	9.98	0.72	0.028
R1-071903-2	14000	7216	50.11	3.61	2497	17.34	1.25	0.031
R1-071903-3	24000	12330	85.62	6.16	4010	27.85	2.01	0.029

Notes: Specimen was remolded near LL, and pushed through the #40 sieve
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
No good consolidation data was recorded for this test.

ϕ'_r = 18.4 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Mike Wanger
Data Reduced By:	Chris Meehan
Test Started On:	7/19/03
Test Finished On:	7/24/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/13/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/21/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
7995	4146	28.79	2.07	-	-	-	-
13993	7213	50.09	3.61	5.3	0.0008	4.5	0.0009
23988	12324	85.58	6.16	4.6	0.0009	2.4	0.0017

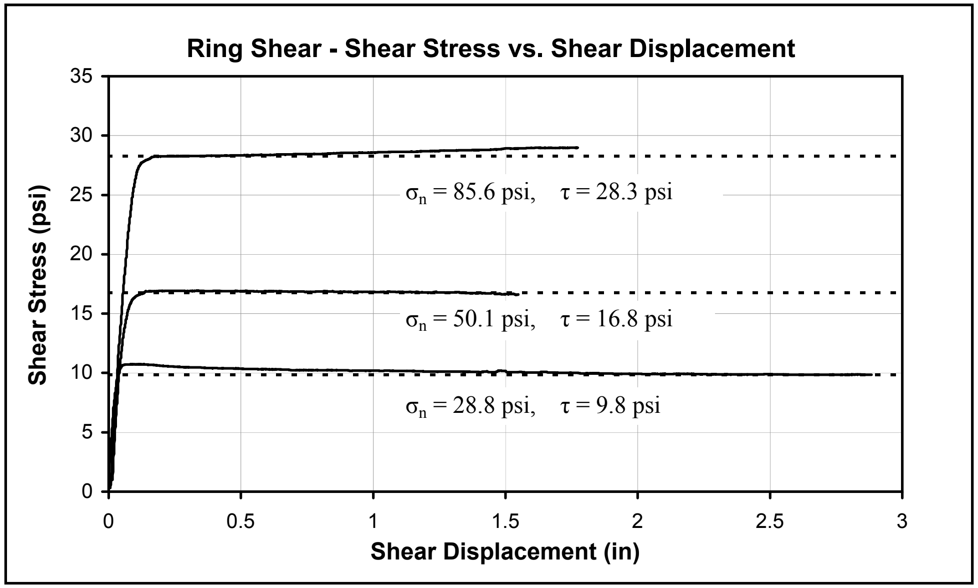
Minimum calc. shear rate =	0.0008	in/min.
Estim. failure displacement =	0.2	in

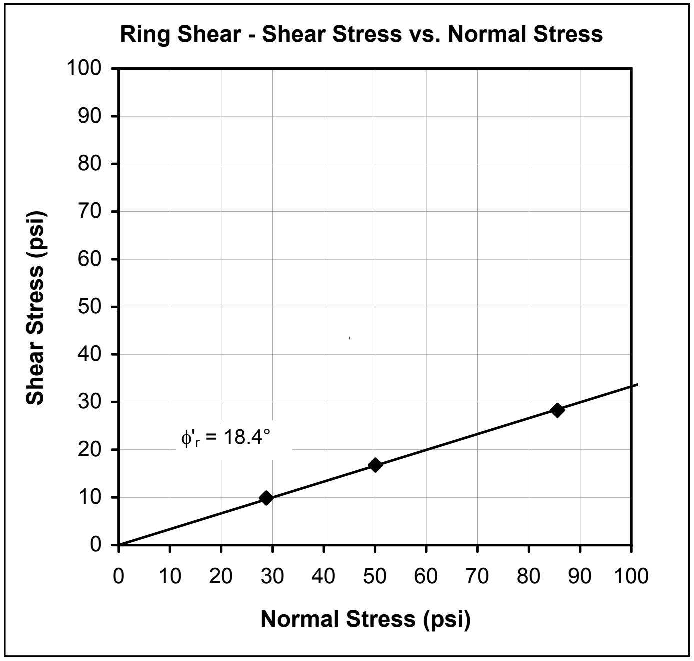
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

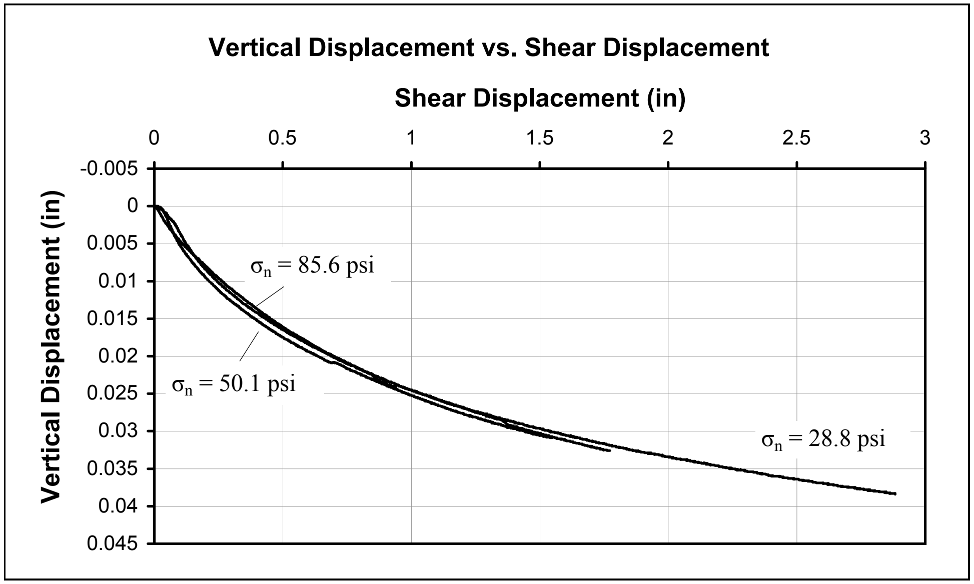
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-081303-1	7995	4146	28.79	2.07	1417	9.84	0.71	0.038
R1-081303-2	13993	7213	50.09	3.61	2415	16.77	1.21	0.031
R1-081303-3	23988	12324	85.58	6.16	4071	28.27	2.04	0.033

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

$\phi'_r =$ 18.4 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	8/13/03
Test Finished On:	8/21/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/13/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/21/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
7995	4146	28.79	2.07	1.5	0.0027	-	-
13993	7213	50.09	3.61	3.9	0.0010	5.7	0.0007
23988	12324	85.58	6.16	4.4	0.0009	2.0	0.0020

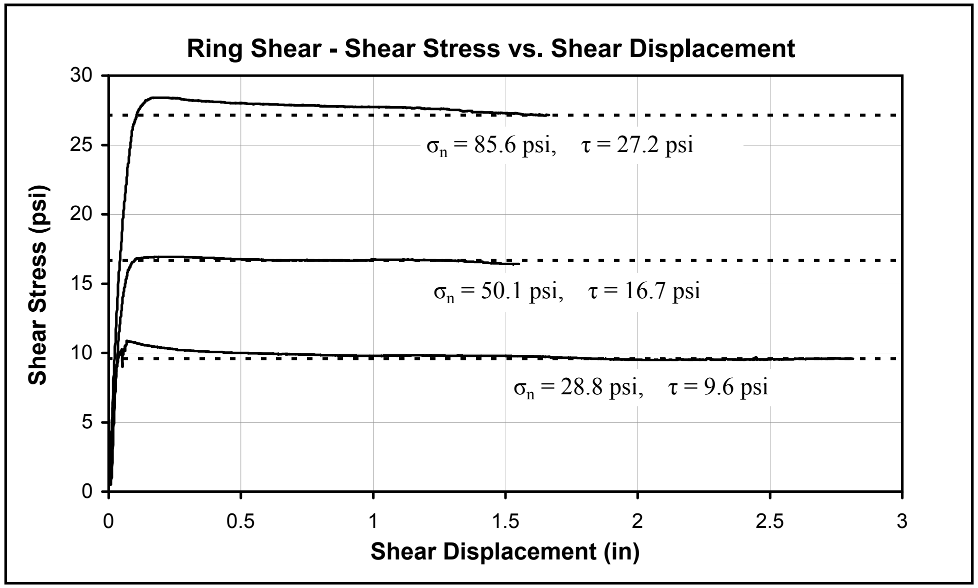
Minimum calc. shear rate =	0.0007	in/min.
Estim. failure displacement =	0.2	in

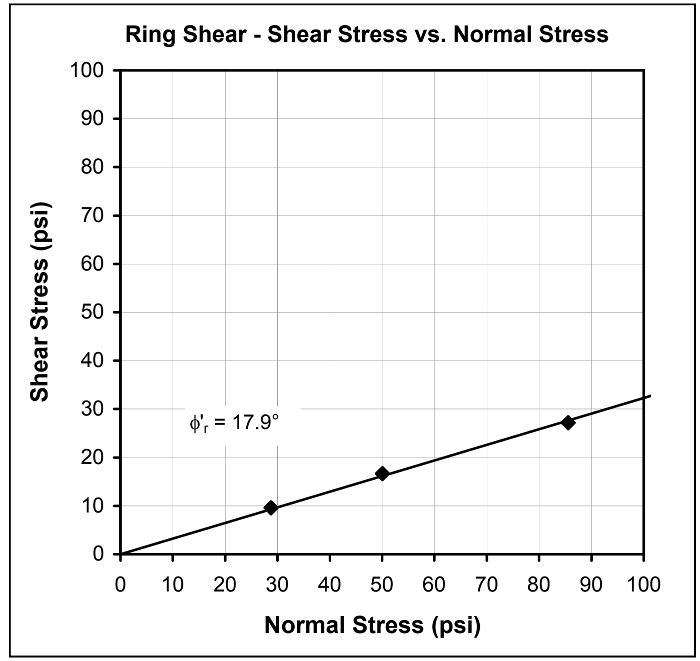
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

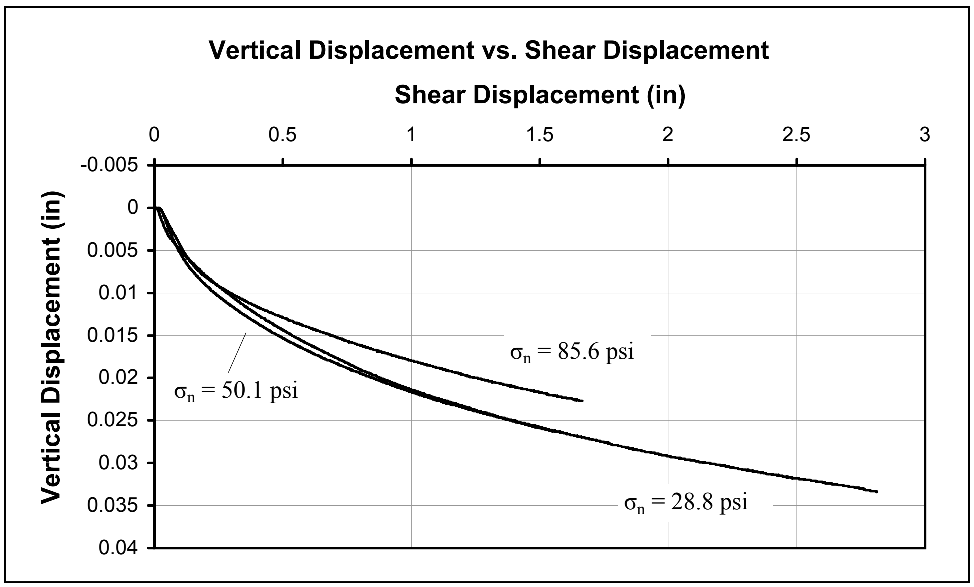
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-081303-1	7995	4146	28.79	2.07	1381	9.59	0.69	0.0334
R2-081303-2	13993	7213	50.09	3.61	2403	16.69	1.20	0.0263
R2-081303-3	23988	12324	85.58	6.16	3911	27.16	1.96	0.0227

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 17.9 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	8/13/03
Test Finished On:	8/21/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/22/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/4/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared	No	in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	Shear rate based on data from earlier tests.			
4000	2103	14.61	1.05				

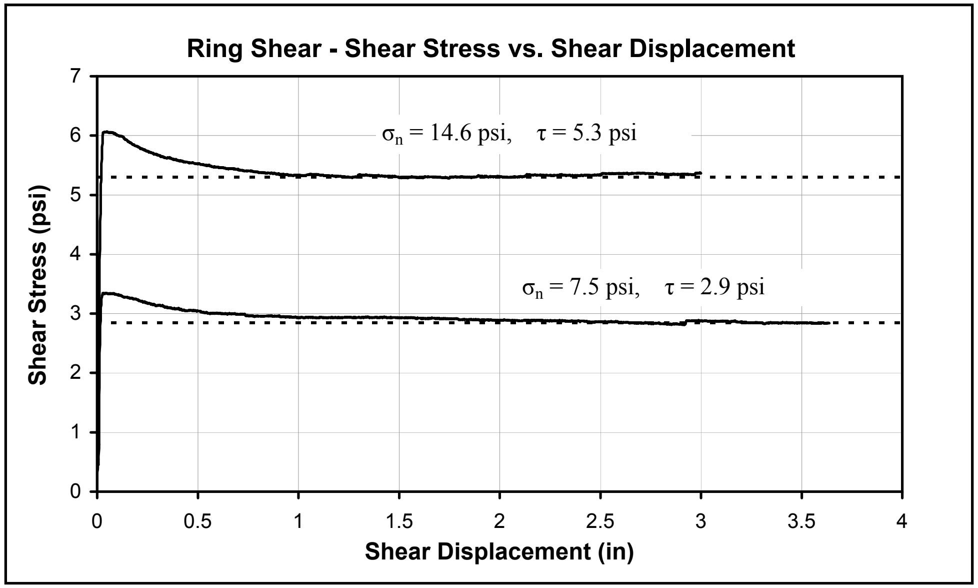
Minimum calc. shear rate =		in/min.
Estim. failure displacement =	0.2	in

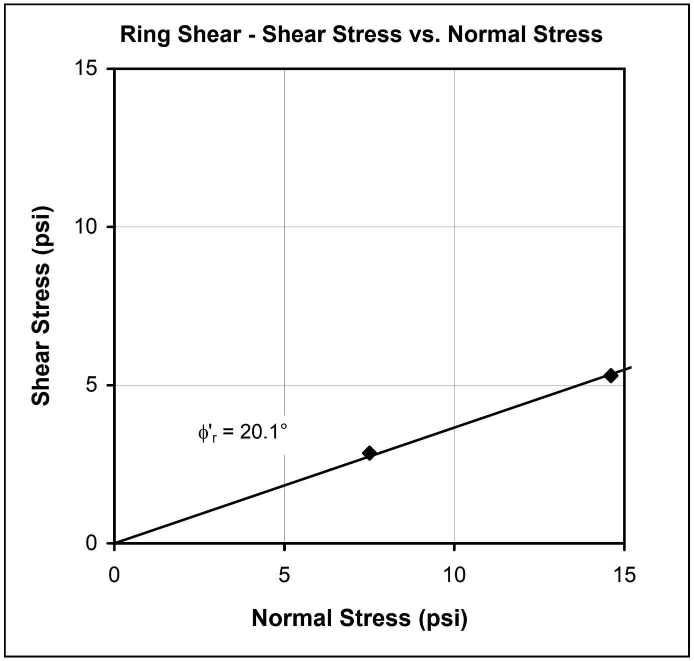
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

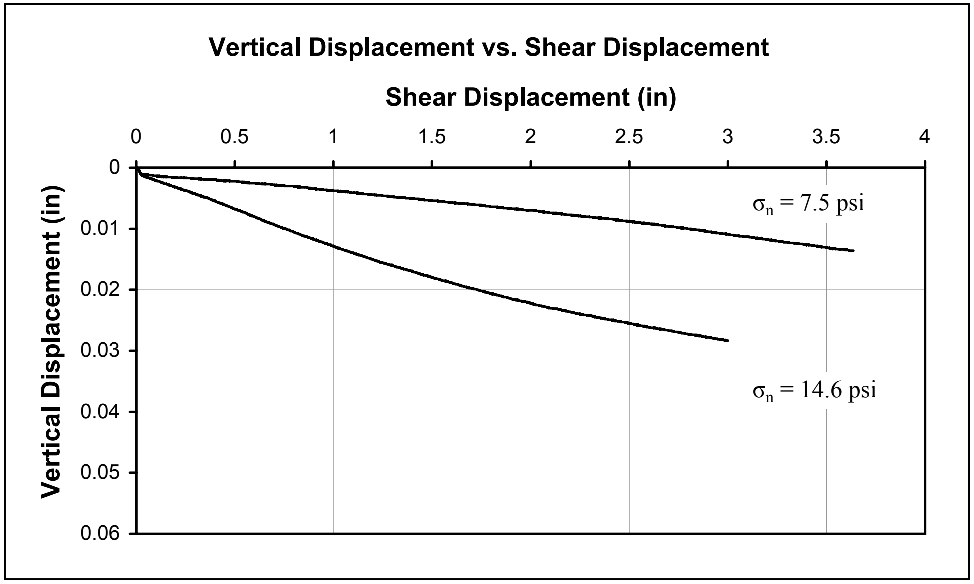
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-082203-1	2000	1081	7.50	0.54	410	2.85	0.21	0.014
R1-082203-2	4000	2103	14.61	1.05	763	5.30	0.38	0.028

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 20.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	8/22/03
Test Finished On:	9/4/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/22/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/4/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	-	-	-	-
4000	2103	14.61	1.05	1.8	0.0022	1.5	0.0027
7995	4146	28.79	2.07				
13993	7213	50.09	3.61				
23988	12324	85.58	6.16	7.5	0.0005	5.8	0.0007

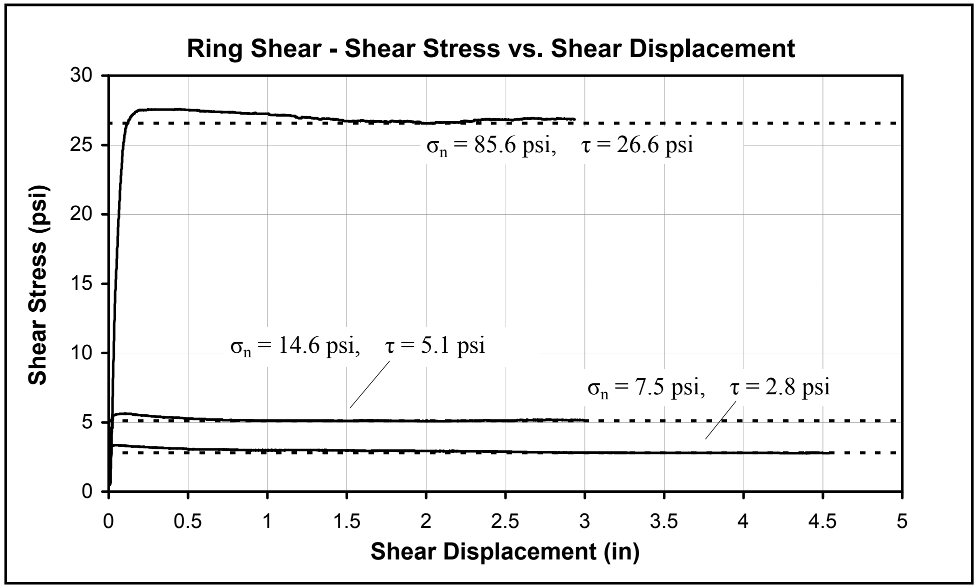
Minimum calc. shear rate =	0.0005	in/min.
Estim. failure displacement =	0.2	in

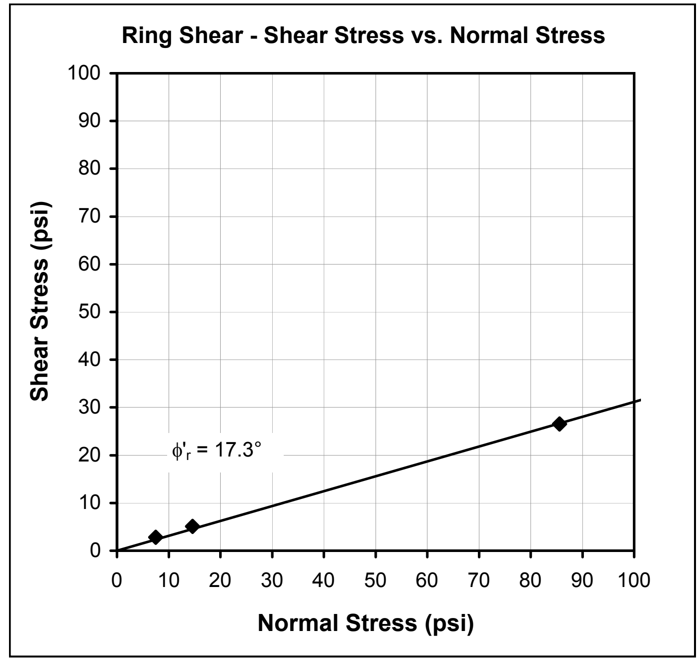
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

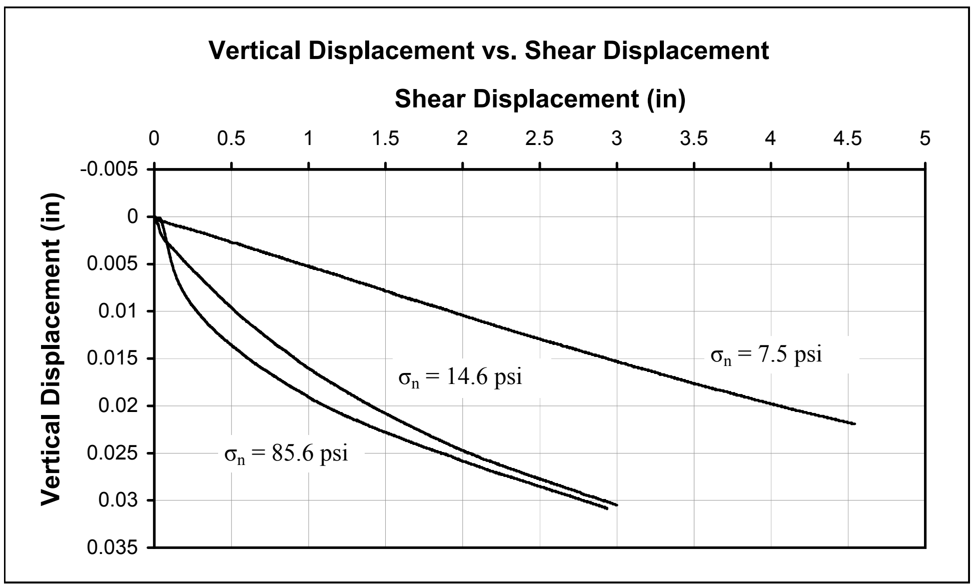
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-082203-1	2000	1081	7.50	0.54	403	2.80	0.20	0.0219
R2-082203-2	4000	2103	14.61	1.05	734	5.10	0.37	0.0305
R2-082203-3	23988	12324	85.58	6.16	3825	26.56	1.91	0.0309

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 17.3 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	8/22/03
Test Finished On:	9/4/03

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/4/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/21/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared: No		in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	-	-	-	-
4000	2103	14.61	1.05	-	-	-	-
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	5.66	0.00071	5.84	0.00068

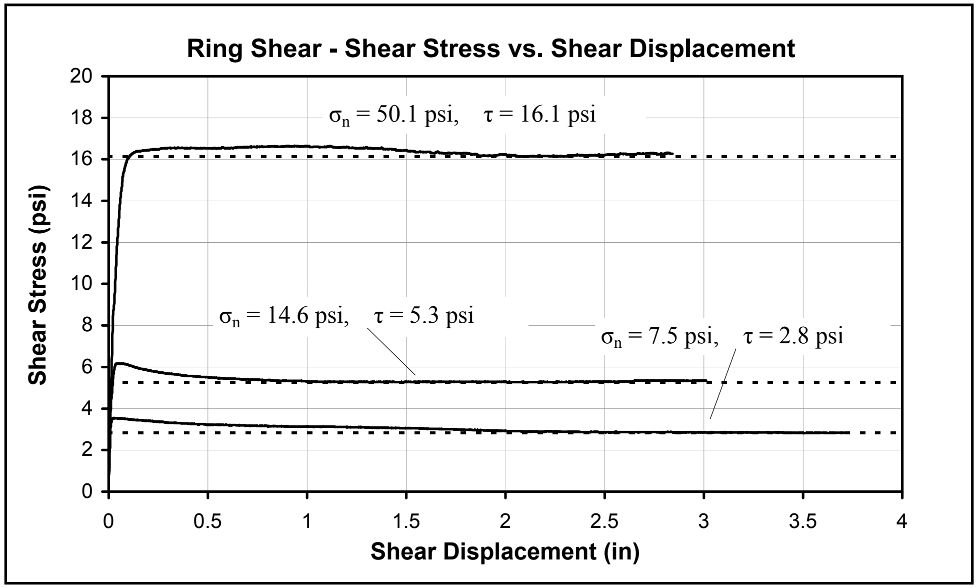
Minimum calc. shear rate =	0.00068	in/min.
Estim. failure displacement =	0.2	in

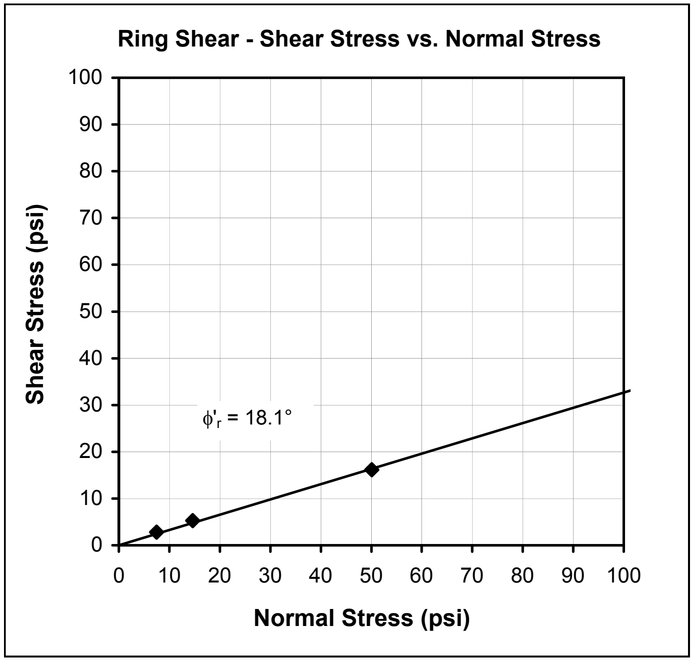
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

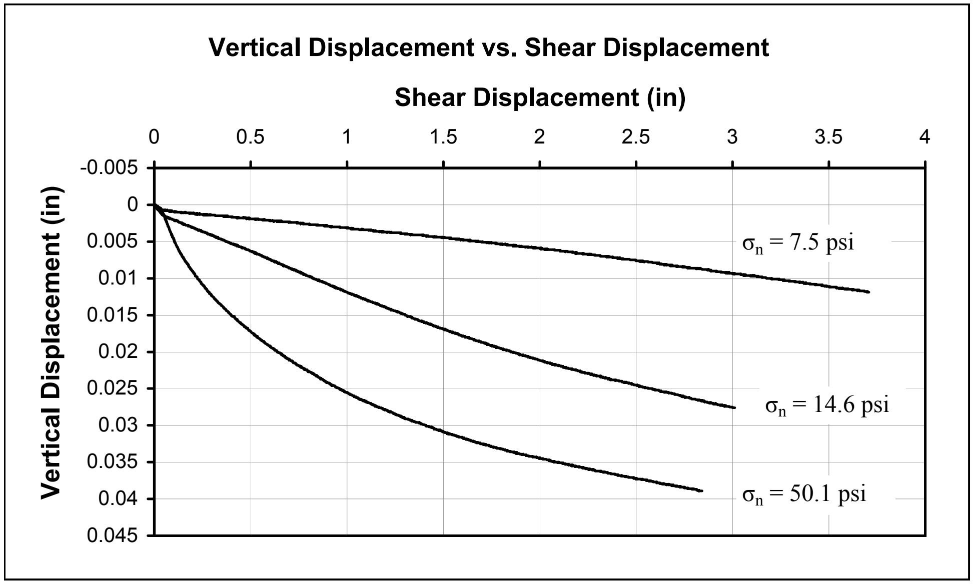
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-090403-1	2000	1081	7.50	0.54	406	2.82	0.20	0.012
R1-090403-2	4000	2103	14.61	1.05	757	5.26	0.38	0.028
R1-090403-3	13993	7213	50.09	3.61	2323	16.13	1.16	0.039

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 18.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	9/4/03
Test Finished On:	9/21/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		Started:	9/4/03
Sample I.D./Loc.:	Rancho Solano Clay #1		Finished:	9/21/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2	

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	-	-	-	-
4000	2103	14.61	1.05	-	-	-	-
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	5.1	0.0008	2.5	0.0016

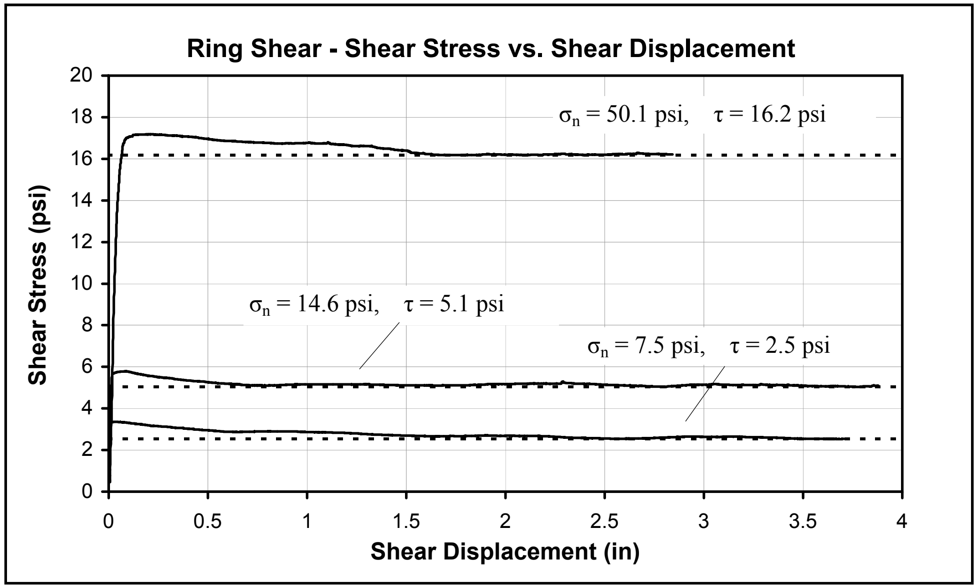
Minimum calc. shear rate =	0.0008	in/min.
Estim. failure displacement =	0.2	in

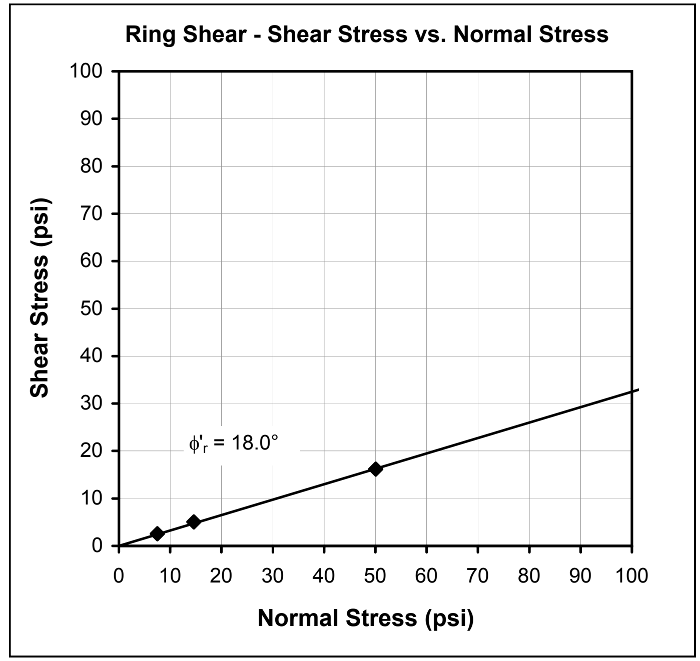
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

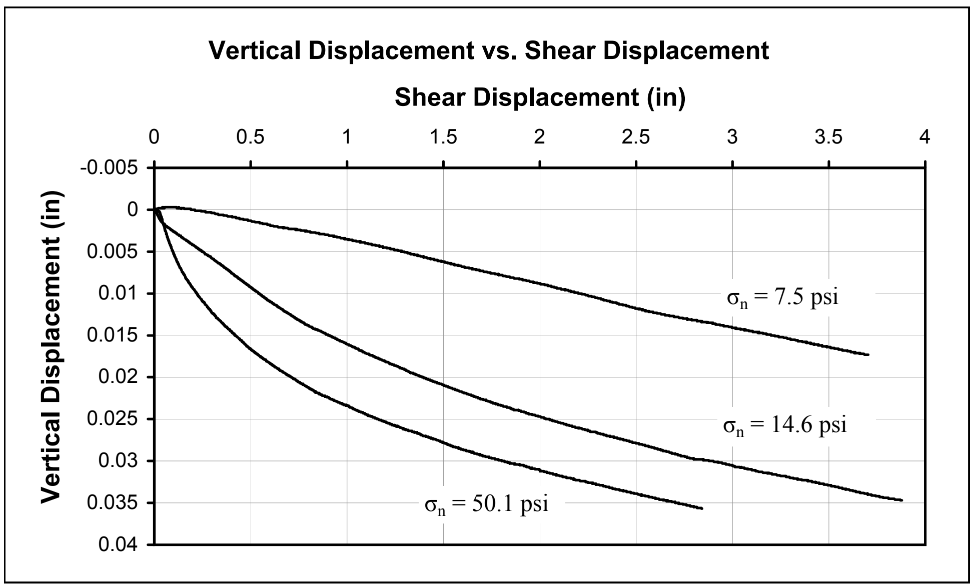
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-090403-1	2000	1081	7.50	0.54	366	2.54	0.18	0.0173
R2-090403-2	4000	2103	14.61	1.05	727	5.05	0.36	0.0347
R2-090403-3	13993	7213	50.09	3.61	2330	16.18	1.16	0.0357

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 18.0 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	9/4/03
Test Finished On:	9/21/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	11/14/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	11/21/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	Shear rate based on data from earlier tests.			

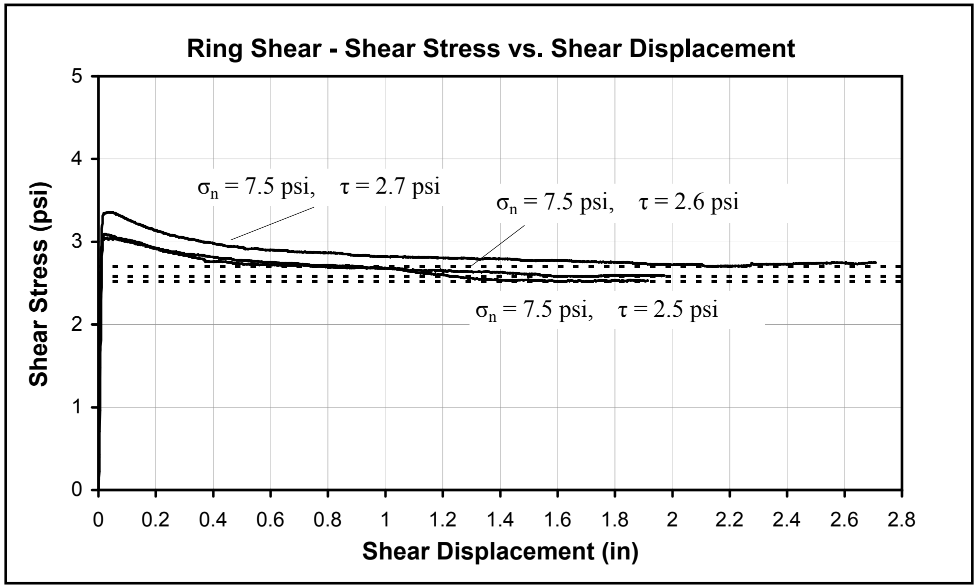
Minimum calc. shear rate =		in/min.
Estim. failure displacement =	0.2	in

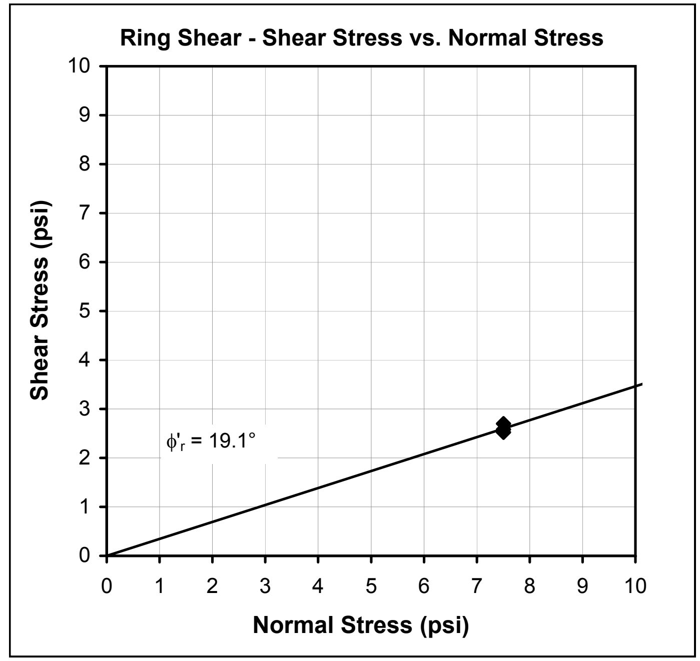
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

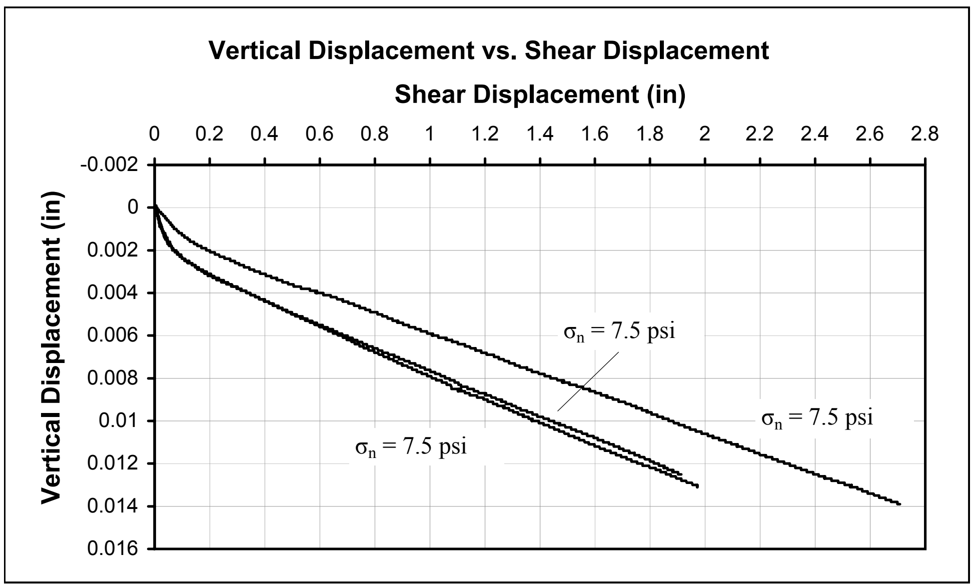
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-111403-1	2000	1081	7.50	0.54	389	2.70	0.19	0.0139
R2-111403-2	2000	1081	7.50	0.54	372	2.58	0.19	0.0131
R2-111403-3	2000	1081	7.50	0.54	363	2.52	0.18	0.0125

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

ϕ'_r = 19.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	11/14/03
Test Finished On:	11/21/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/7/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	10/20/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared	No	in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000	2103	14.61	1.05				
7995	4146	28.79	2.07	-	-	-	-
13993	7213	50.09	3.61	2.8	0.0014	3.2	0.0013

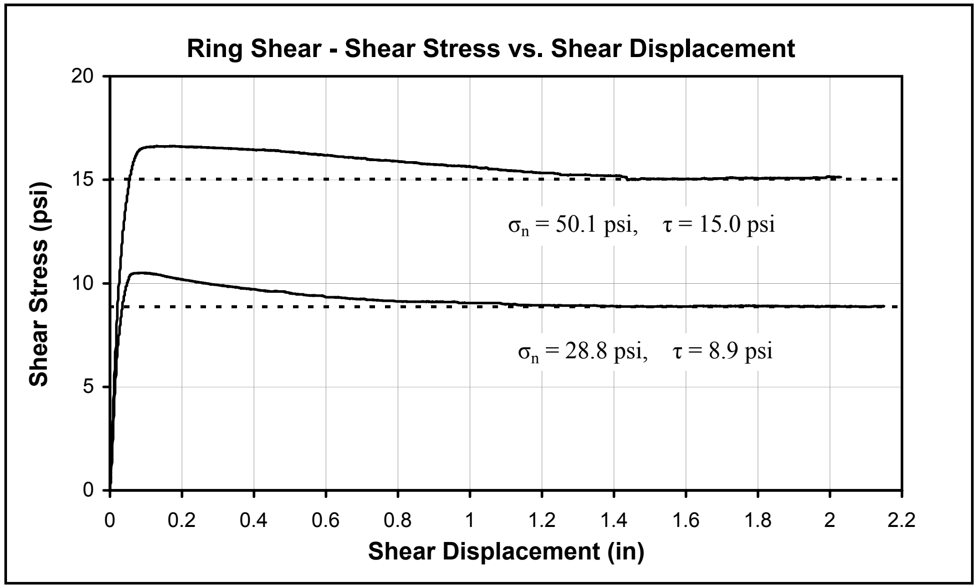
Minimum calc. shear rate =	0.0013	in/min.
Estim. failure displacement =	0.2	in

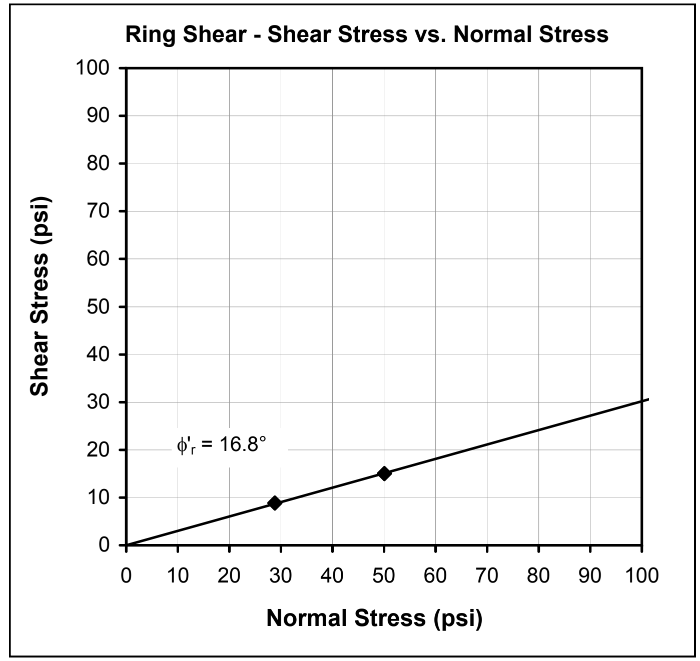
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

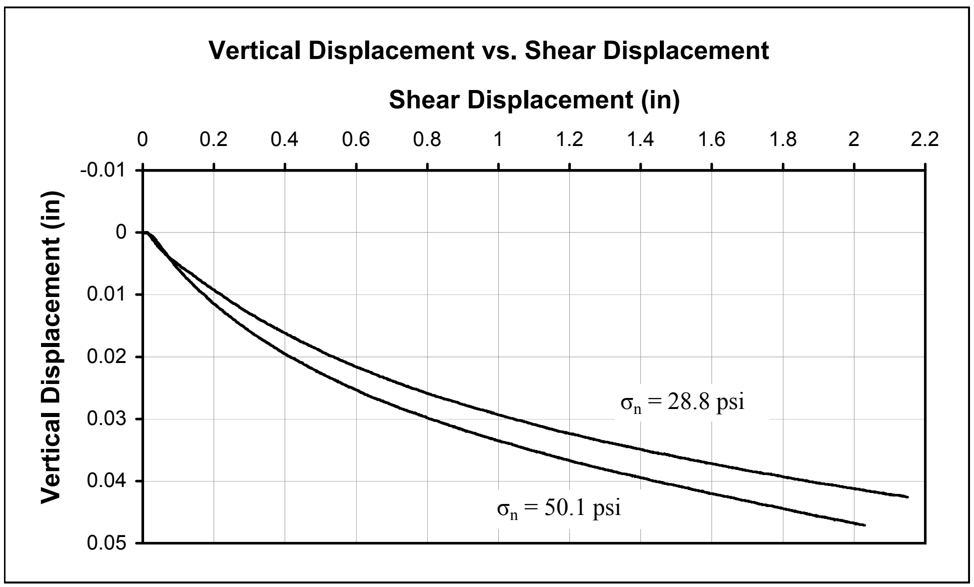
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-100703-1	7995	4146	28.79	2.07	1279	8.88	0.64	0.043
R1-100703-2	13993	7213	50.09	3.61	2164	15.03	1.08	0.047

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 16.8 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	10/7/03
Test Finished On:	10/20/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/20/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	11/8/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	-	-	-	-
4000	2103	14.61	1.05	1.6	0.0025	2.2	0.0018
7995	4146	28.79	2.07	1.2	0.0033	-	-
7995	4146	28.79	2.07	2.6	0.0015	2.1	0.0019

Minimum calc. shear rate =	0.0015	in/min.
Estim. failure displacement =	0.2	in

Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-102003-1	2000	1081	7.50	0.54	341	2.37	0.17	0.026
R1-102003-2	4000	2103	14.61	1.05	618	4.29	0.31	0.040
R1-102003-3	7995	4146	28.79	2.07	1171	8.13	0.59	0.042
R1-102003-4	7995	4146	28.79	2.07	1207	8.38	0.60	0.045

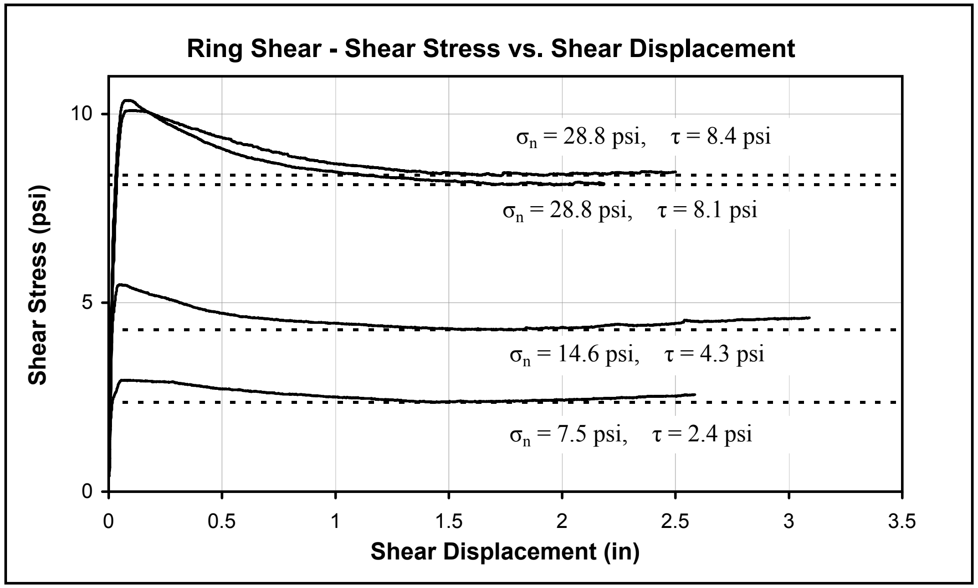
Notes: Specimen was remolded at LL, and pushed through the #40 sieve.

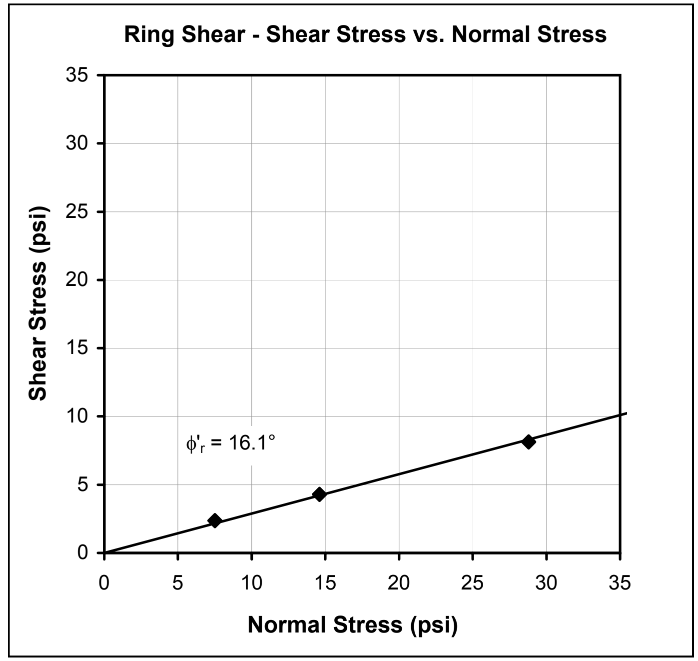
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

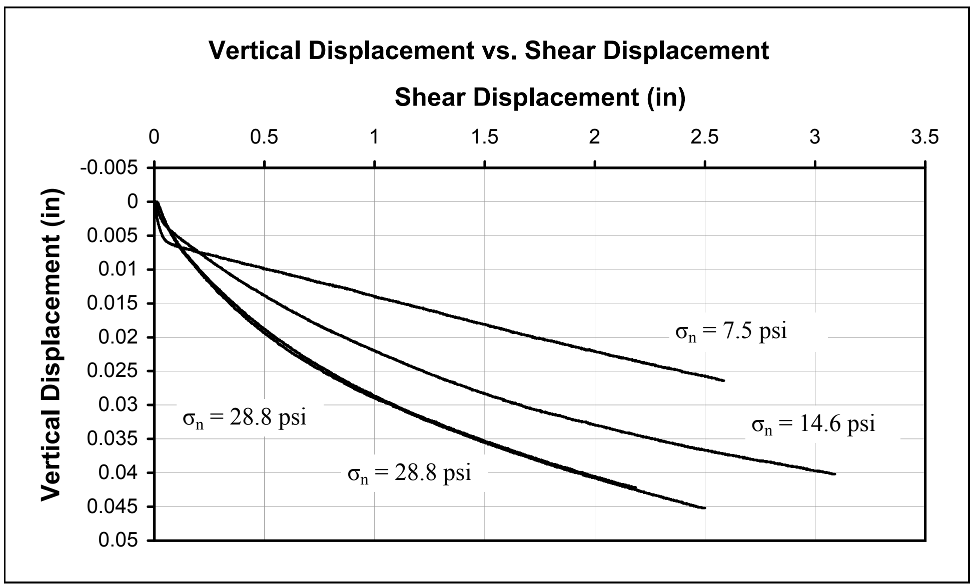
Modified platen used for ring shear tests.

ϕ'_r = 16.1 deg.

(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	10/20/03
Test Finished On:	11/8/03

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	11/8/03
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	12/3/03
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared	No	in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000	2103	14.61	1.05				
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	4.9	0.0008	3.7	0.0011
23988	12324	85.58	6.16	5.8	0.0007	3.2	0.0013

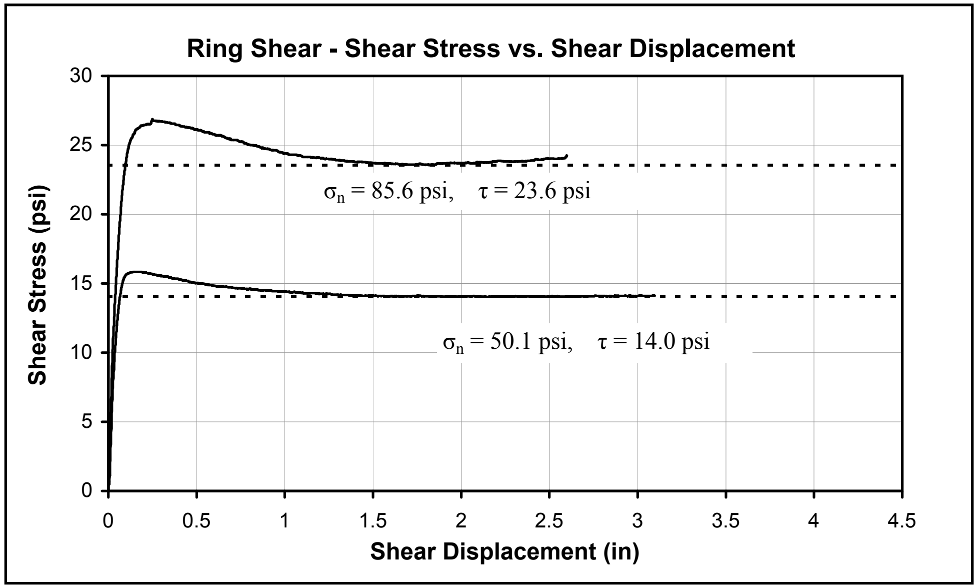
Minimum calc. shear rate =	0.0007	in/min.
Estim. failure displacement =	0.2	in

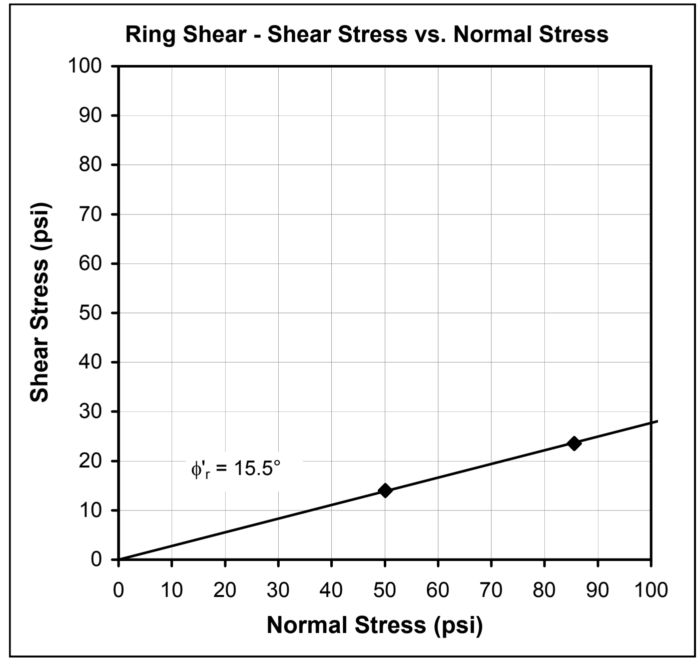
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

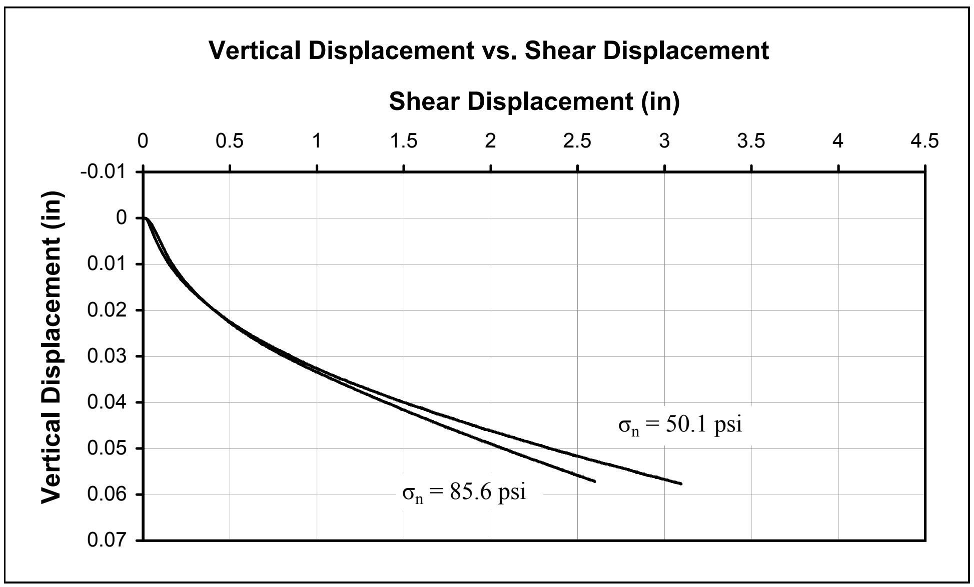
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-110803-1	13993	7213	50.09	3.61	2020	14.03	1.01	0.058
R1-110803-2	23988	12324	85.58	6.16	3394	23.57	1.70	0.057

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 15.5 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	11/8/03
Test Finished On:	12/3/03

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	1/6/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	1/29/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000	2103	14.61	1.05	1.8	0.0023	2.5	0.0016
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	4.5	0.0009	4.7	0.0008
23988	12324	85.58	6.16	1.7	0.0023	-	-

Minimum calc. shear rate =	0.0008	in/min.
Estim. failure displacement =	0.2	in

Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

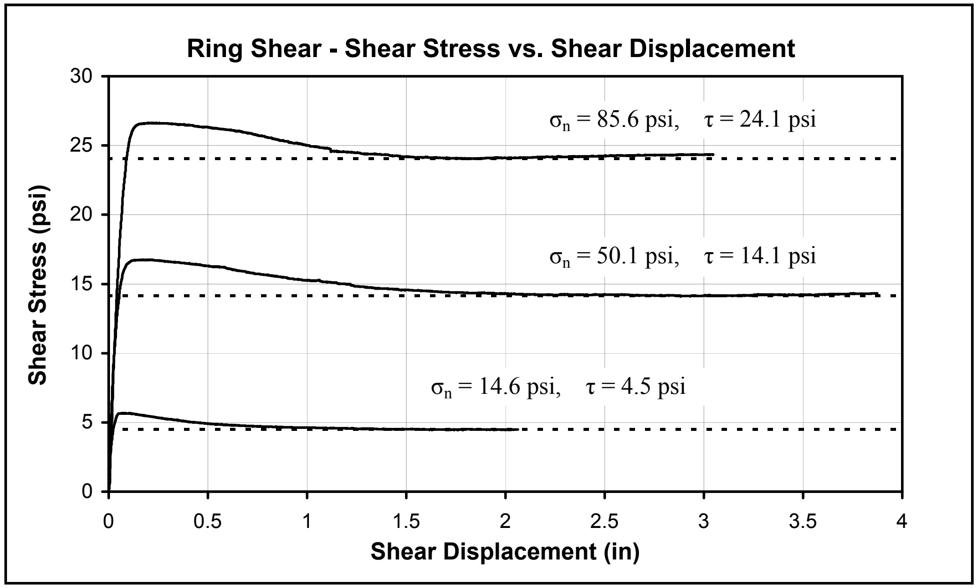
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-010604-1	4000	2103	14.61	1.05	645	4.48	0.32	0.035
R1-010604-2	13993	7213	50.09	3.61	2035	14.13	1.02	0.066
R1-010604-3	23988	12324	85.58	6.16	3463	24.05	1.73	0.065

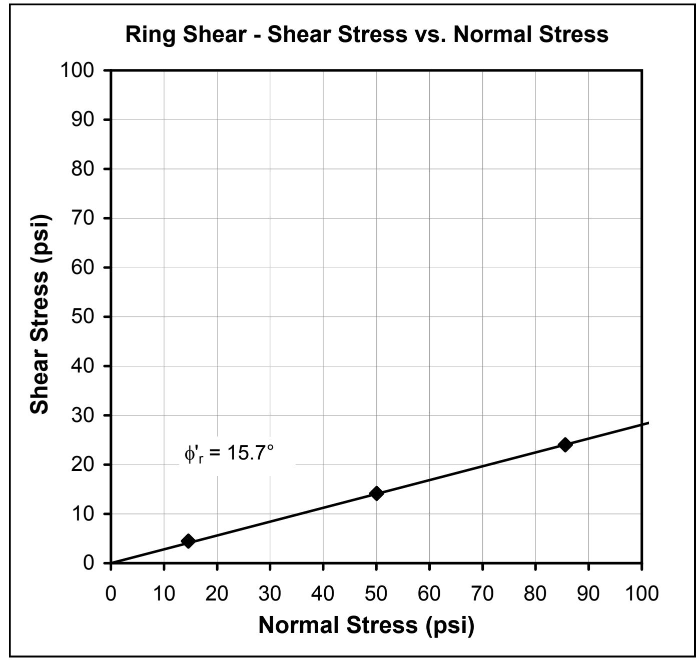
ϕ'_r = 15.7 deg.
(based on a best fit line for c'_r = 0)

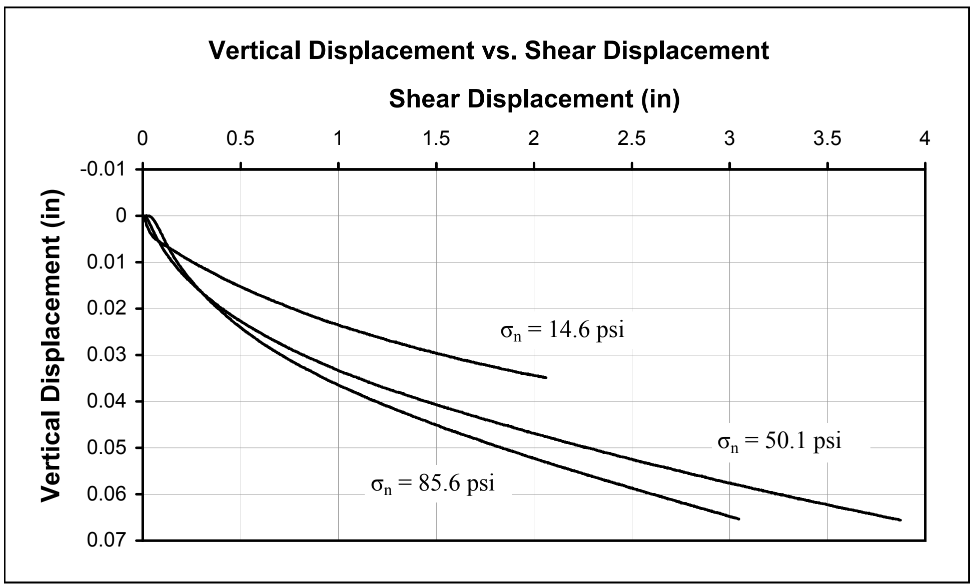
Notes: Specimen was remolded at LL, and pushed through the #40 sieve.

Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.

Modified platen used for ring shear tests.







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	1/6/04
Test Finished On:	1/29/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	3/8/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	3/27/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared	No	in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000	2103	14.61	1.05				
7995	4146	28.79	2.07	1.5	0.0026	-	-
13993	7213	50.09	3.61	4.6	0.0009	3.7	0.0011
23988	12324	85.58	6.16	3.1	0.0013	2.9	0.0014

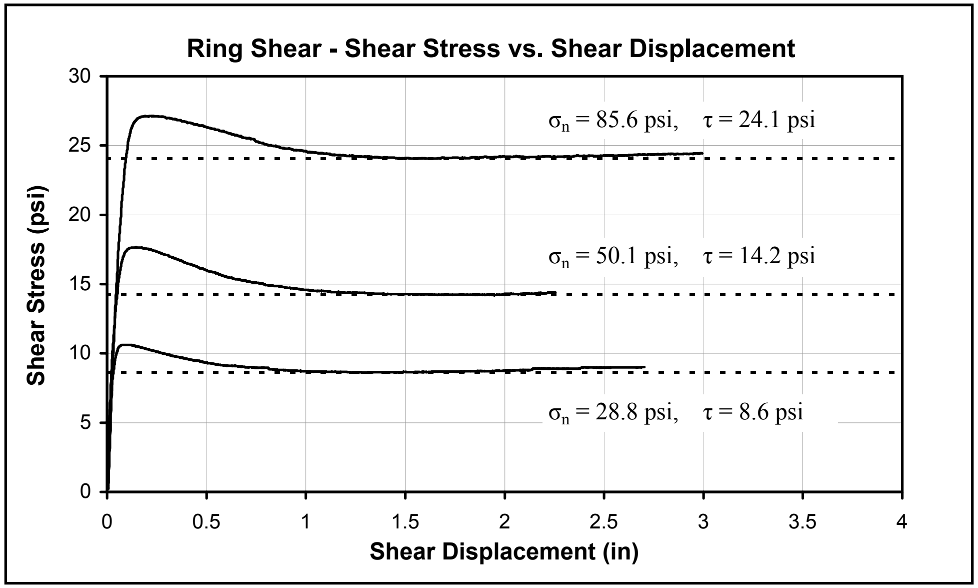
Minimum calc. shear rate =	0.0009	in/min.
Estim. failure displacement =	0.2	in

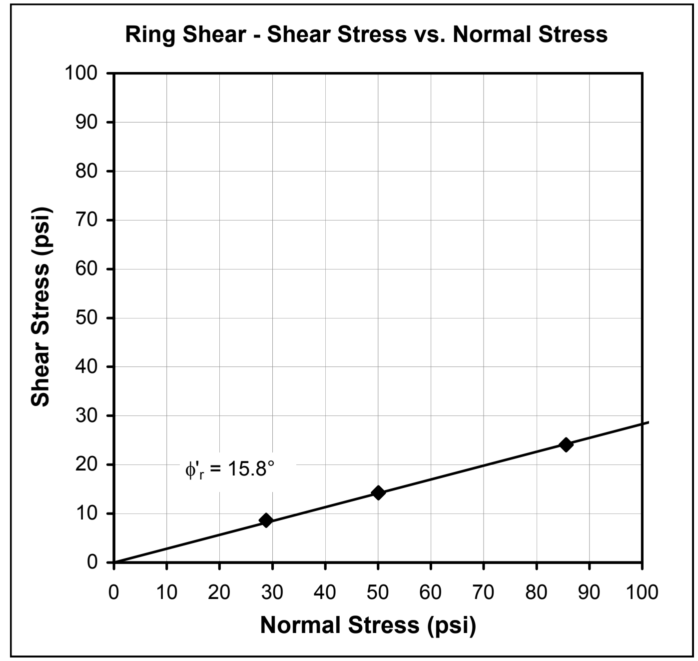
Test performed at shear rate =	0.000709	in/min.
Test performed at shear rate =	0.024	deg/min.

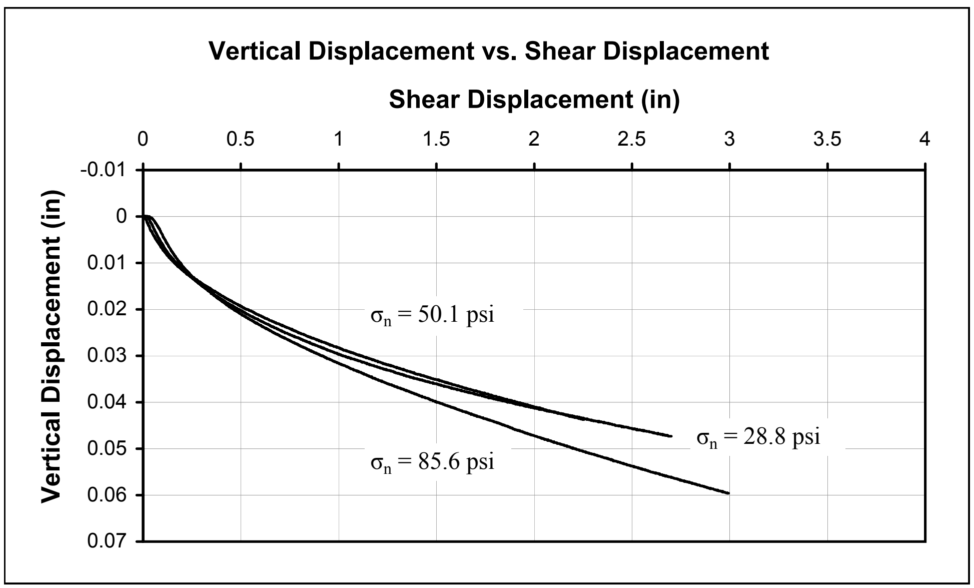
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-030804-1	7995	4146	28.79	2.07	1243	8.63	0.62	0.047
R1-030804-2	13993	7213	50.09	3.61	2049	14.23	1.02	0.044
R1-030804-3	23988	12324	85.58	6.16	3463	24.05	1.73	0.060

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 15.8 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	3/8/04
Test Finished On:	3/27/04

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	4/15/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	4/27/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000	2103	14.61	1.05	1.2	0.0034	0.7	0.0061
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	5.2	0.0008	3.3	0.0012

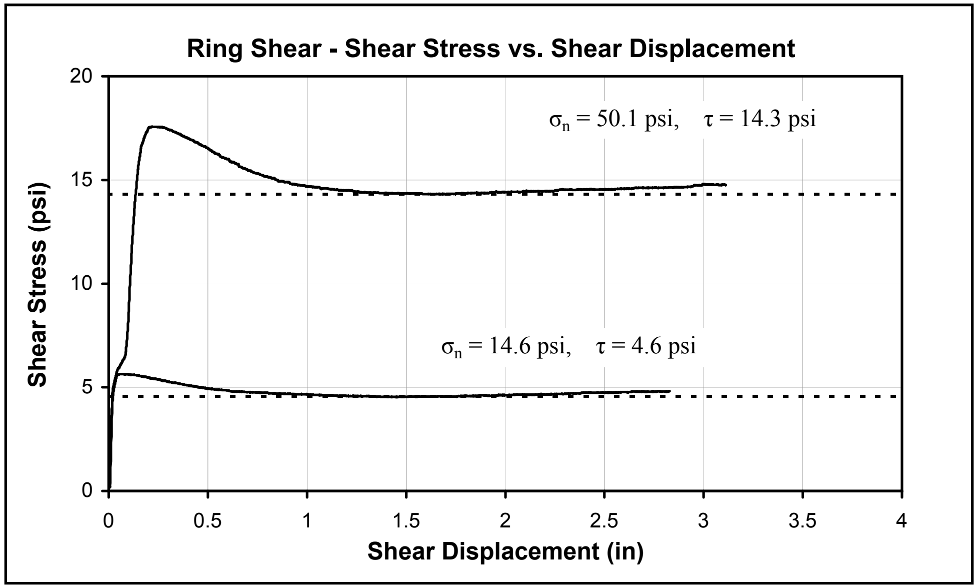
Minimum calc. shear rate =	0.0008	in/min.
Estim. failure displacement =	0.2	in

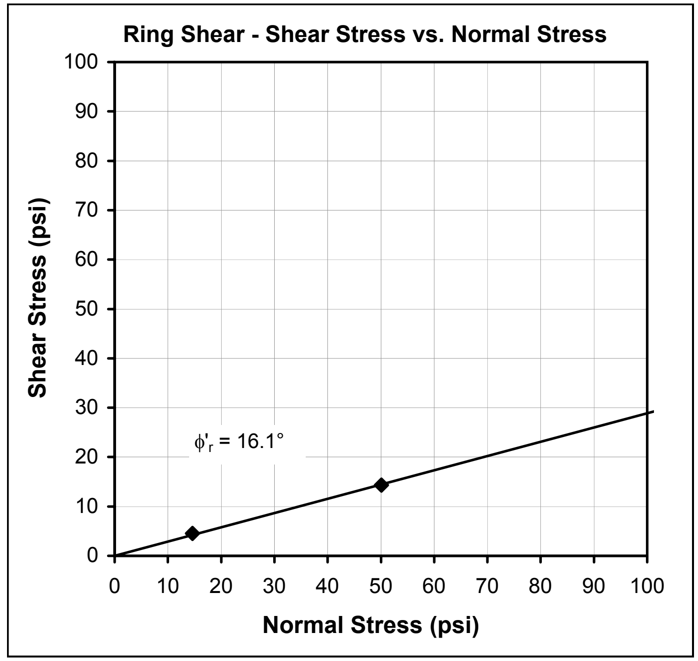
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

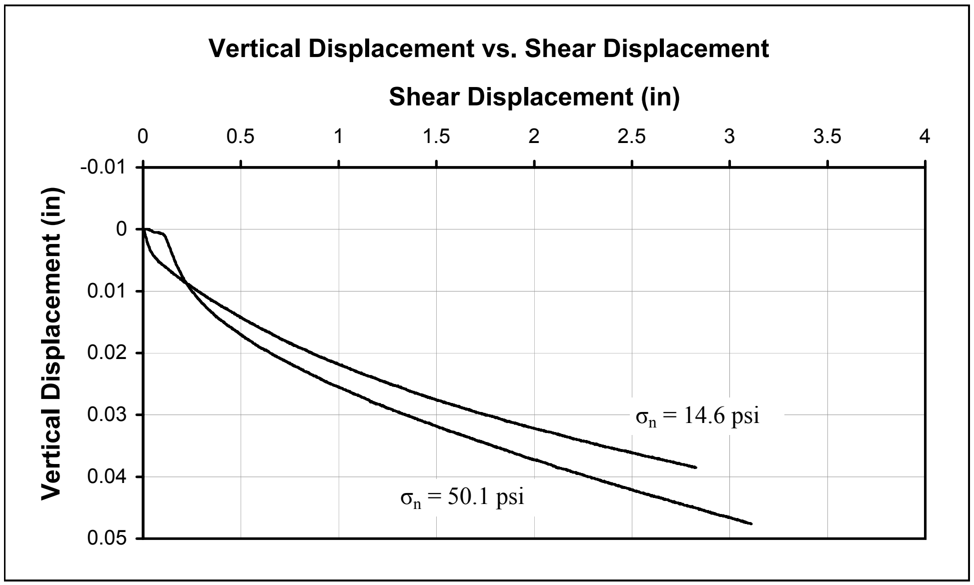
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-041504-1	4000	2103	14.61	1.05	655	4.55	0.33	0.039
R1-041504-2	13993	7213	50.09	3.61	2061	14.31	1.03	0.048

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 16.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	4/15/04
Test Finished On:	4/27/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	4/27/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	5/8/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.65	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared	No	in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	1.9	0.0021	0.9	0.0043

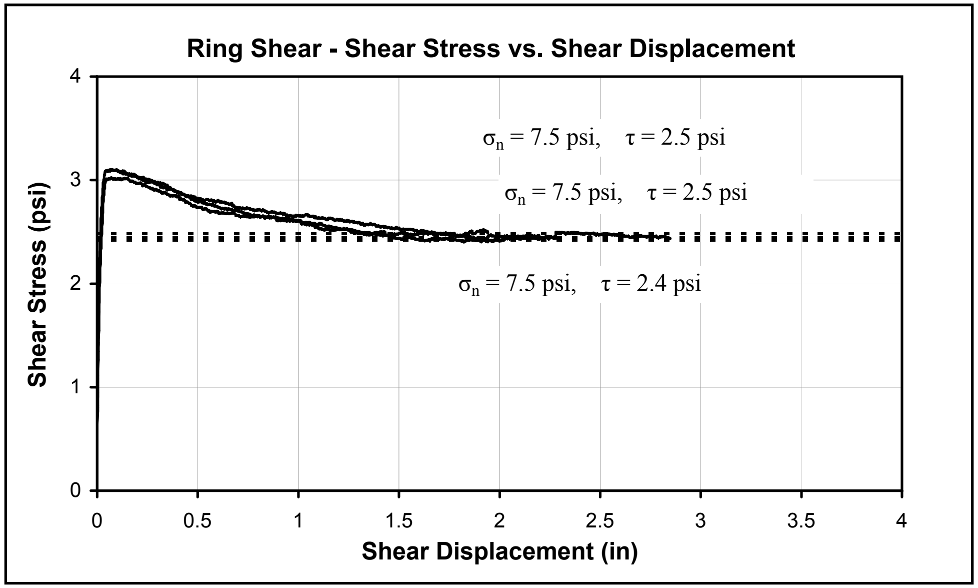
Minimum calc. shear rate =	0.0021	in/min.
Estim. failure displacement =	0.2	in

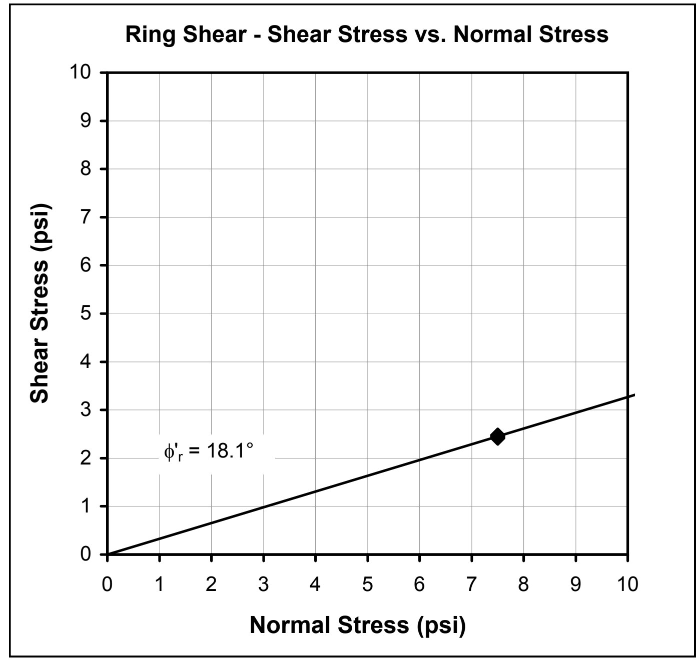
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

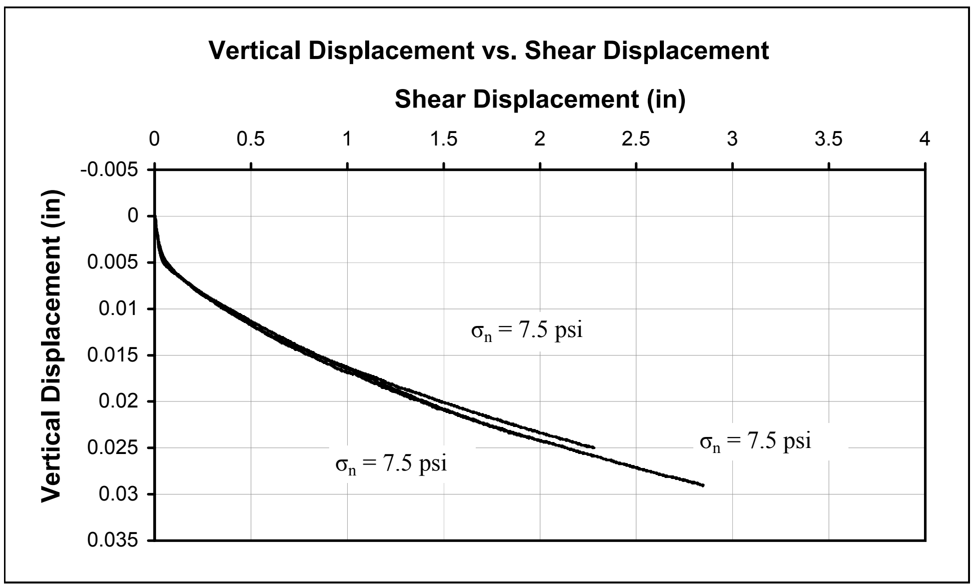
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-042704-1	2000	1081	7.50	0.54	348	2.42	0.17	0.025
R1-042704-2	2000	1081	7.50	0.54	353	2.45	0.18	0.029
R1-042704-3	2000	1081	7.50	0.54	357	2.48	0.18	0.024

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 18.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	4/27/04
Test Finished On:	5/8/04

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/7/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	6/18/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps					Casagrande		Taylor	
Consolidation Load		Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)		(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1999		1080	7.50	0.54				
3998		2102	14.60	1.05				
7995		4146	28.79	2.07				
13990		7211	50.08	3.61				
23985	(test 1)	12322	85.57	6.16	5.6	0.0007	3.7	0.0011
23985	(test 2)	12322	85.57	6.16	4.5	0.0009	4.2	0.0009
23985	(test 3)	12322	85.57	6.16	4.0	0.0010	2.2	0.0018

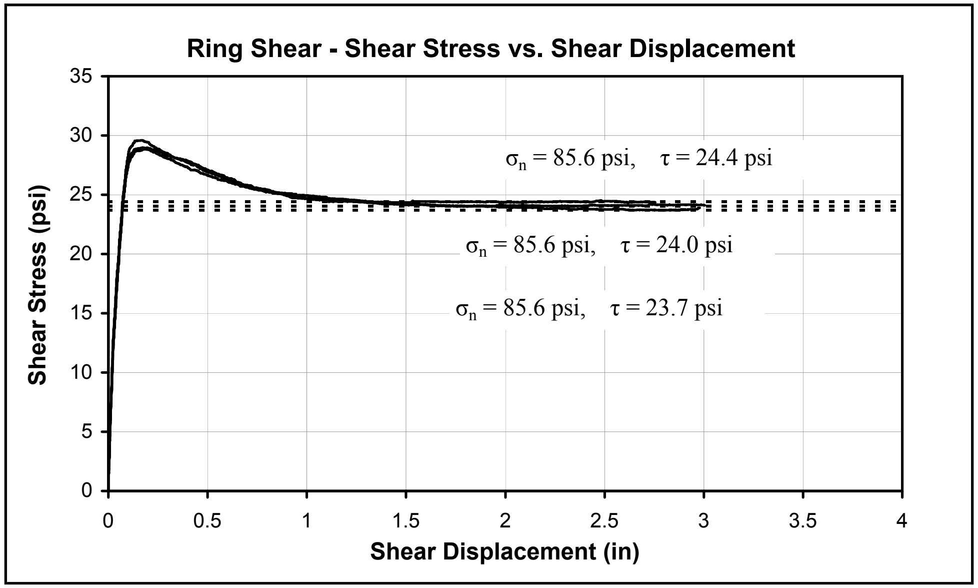
Minimum calc. shear rate =	0.0007	in/min.
Estim. failure displacement =	0.2	in

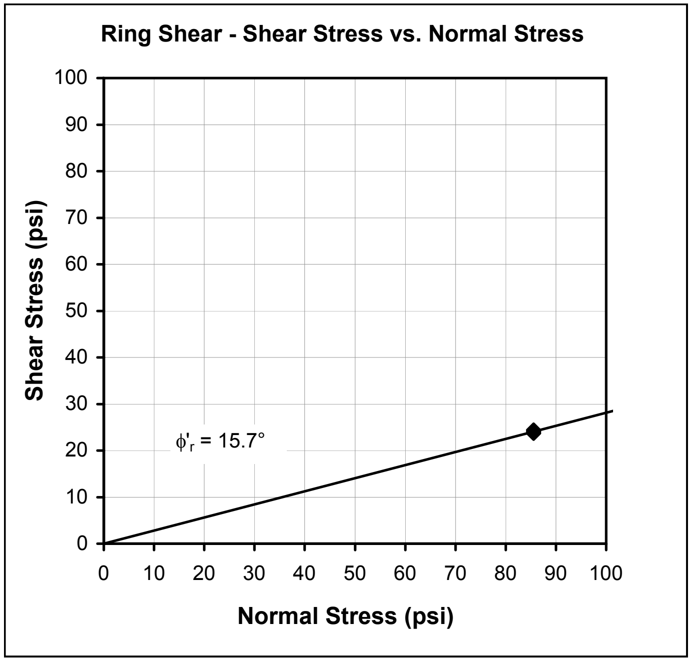
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

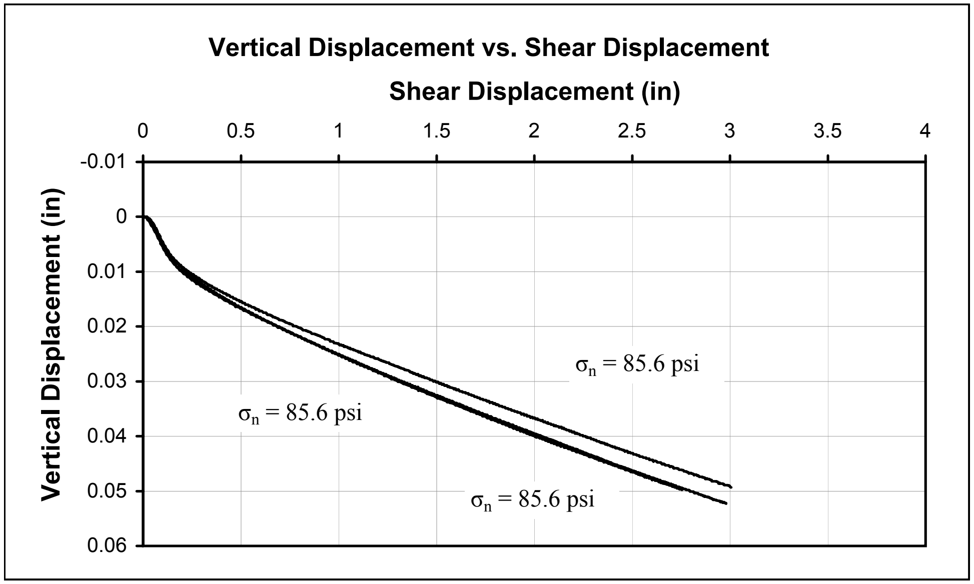
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-060704-1	23985	12322	85.57	6.16	3414	23.71	1.71	0.052
R2-060704-2	23985	12322	85.57	6.16	3516	24.42	1.76	0.050
R2-060704-3	23985	12322	85.57	6.16	3462	24.04	1.73	0.049

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 15.7 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	6/7/04
Test Finished On:	6/18/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/27/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	7/7/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1999	1080	7.50	0.54	-	-	-	-
3998	2102	14.60	1.05	2.7	0.0015	0.9	0.0043

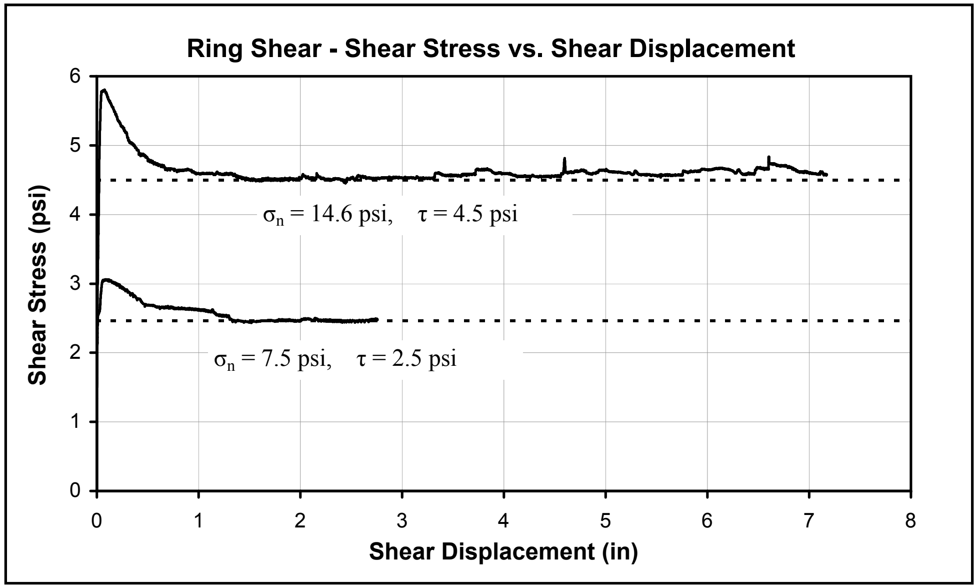
Minimum calc. shear rate =	0.0015	in/min.
Estim. failure displacement =	0.2	in

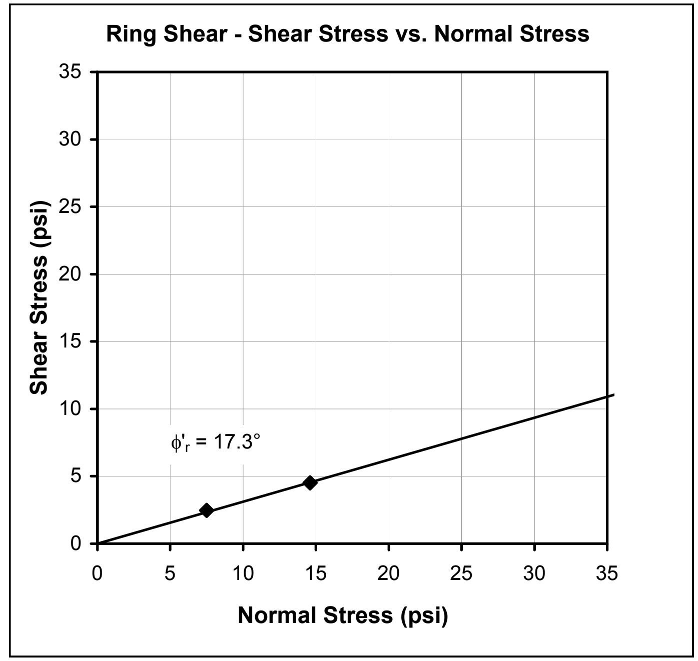
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

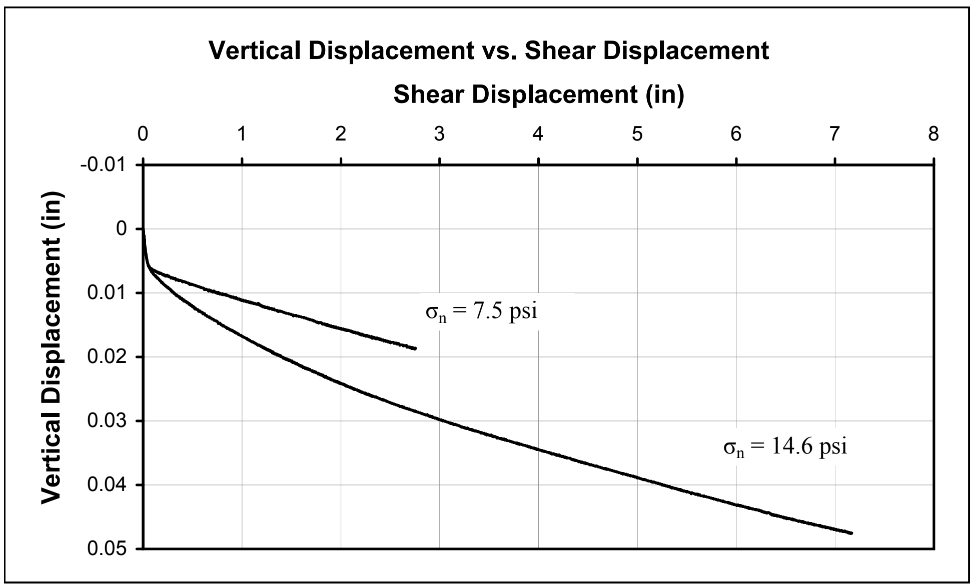
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-062704-1	1999	1080	7.50	0.54	354	2.46	0.18	0.019
R2-062704-2	3998	2102	14.60	1.05	648	4.50	0.32	0.048

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 17.3 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	6/27/04
Test Finished On:	7/7/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	7/14/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	7/22/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.65	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1999	1080	7.50	0.54				
3998	2102	14.60	1.05				
7995	4146	28.79	2.07	3.5	0.0012	1.3	0.0030
13990	7211	50.08	3.61	4.9	0.0008	4.5	0.0009

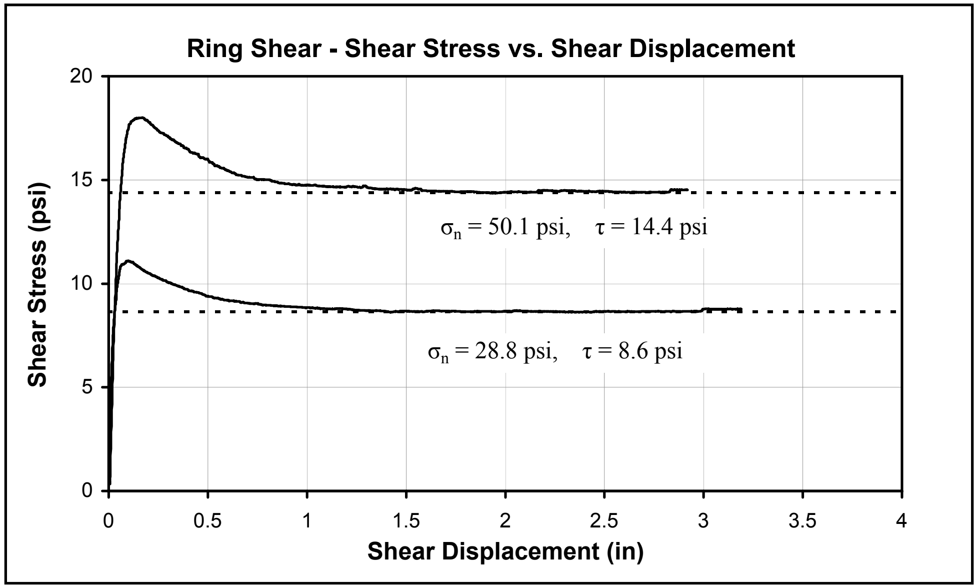
Minimum calc. shear rate =	0.0008	in/min.
Estim. failure displacement =	0.2	in

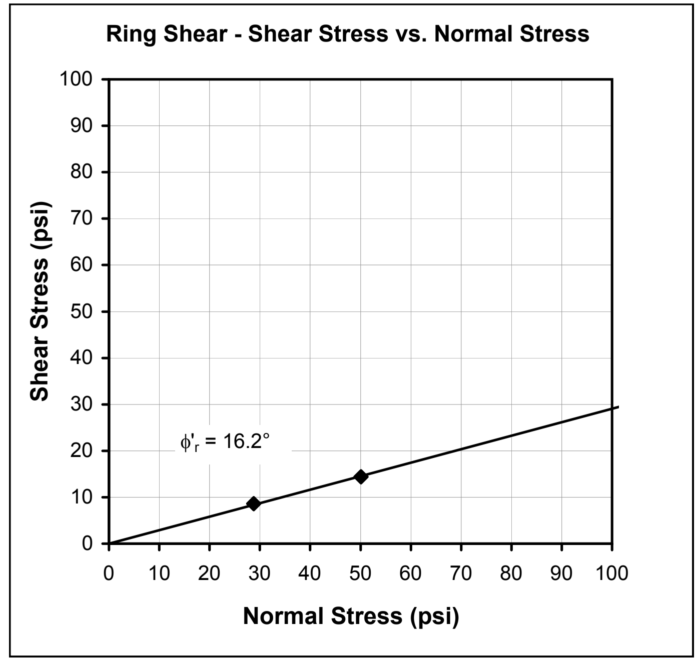
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

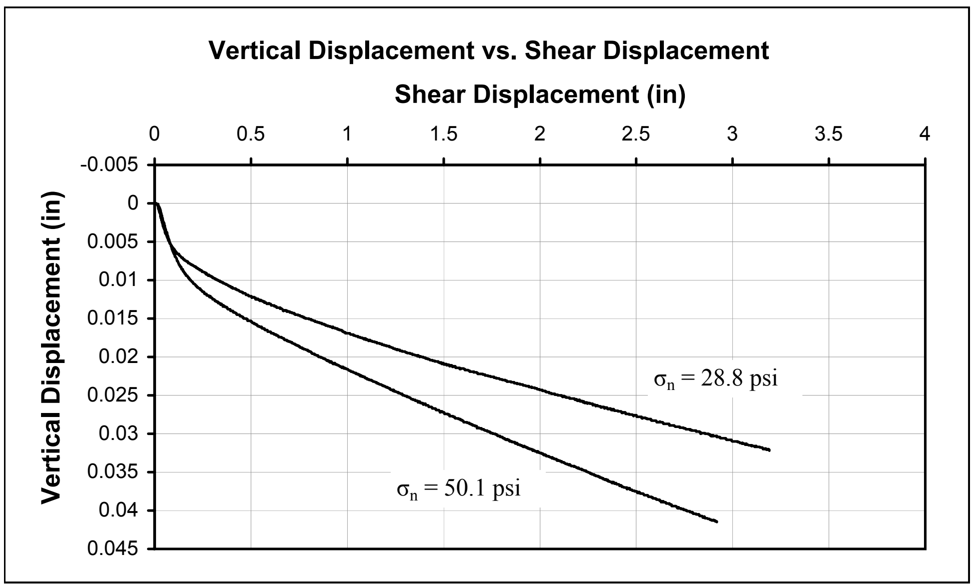
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-071404-1	7995	4146	28.79	2.07	1244	8.64	0.62	0.032
R2-071404-2	13990	7211	50.08	3.61	2072	14.39	1.04	0.042

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 16.2 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #1
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	7/14/04
Test Finished On:	7/22/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/23/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/5/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.79	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	0.6	0.0069	0.3	0.0122

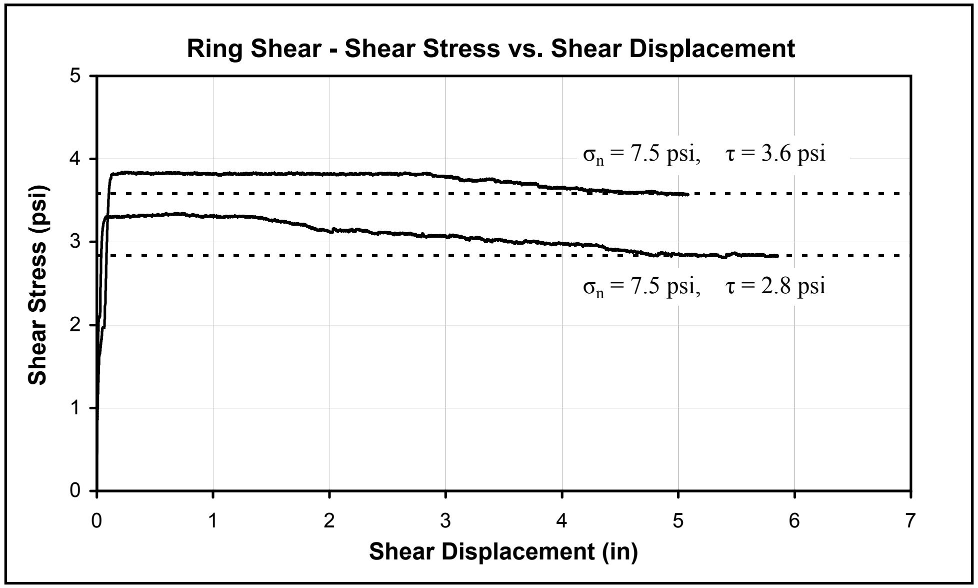
Minimum calc. shear rate =	0.0069	in/min.
Estim. failure displacement =	0.2	in

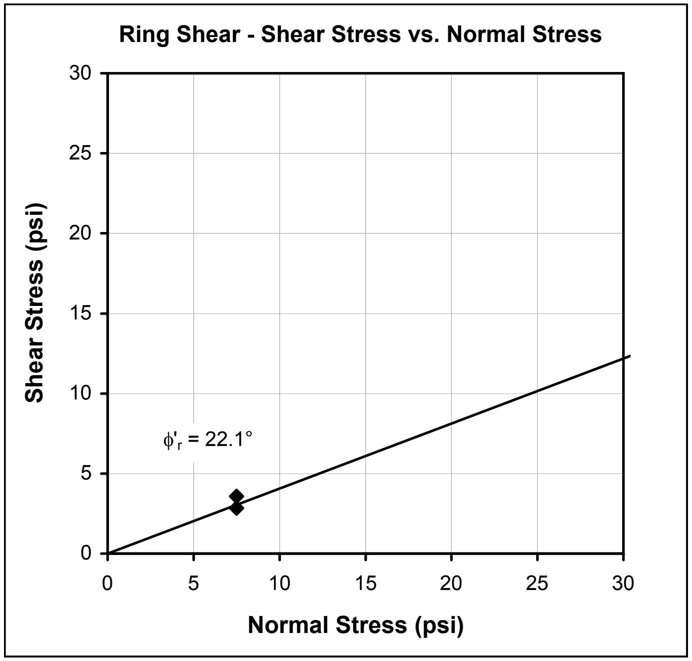
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

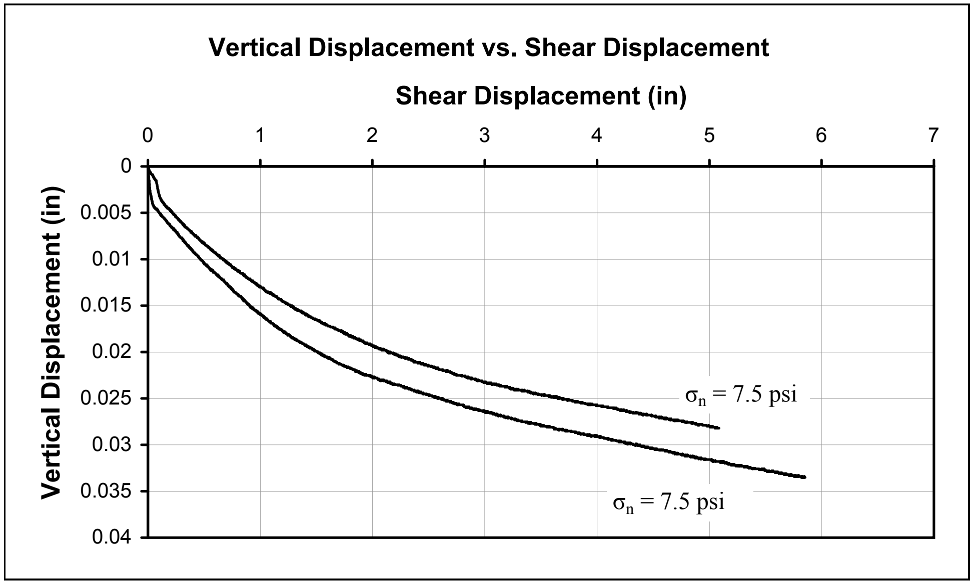
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-092305-1	2000	1081	7.50	0.54	516	3.58	0.26	0.028
R1-092305-2	2000	1081	7.50	0.54	408	2.83	0.20	0.034

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 22.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Derek Martowska
Data Reduced By:	Chris Meehan
Test Started On:	9/23/05
Test Finished On:	10/5/05

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/23/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/11/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.79	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	0.5	0.0074	0.5	0.0087
4000	2103	14.61	1.05	0.5	0.0077	0.4	0.0094
7995	4146	28.79	2.07	0.5	0.0083	0.2	0.0211

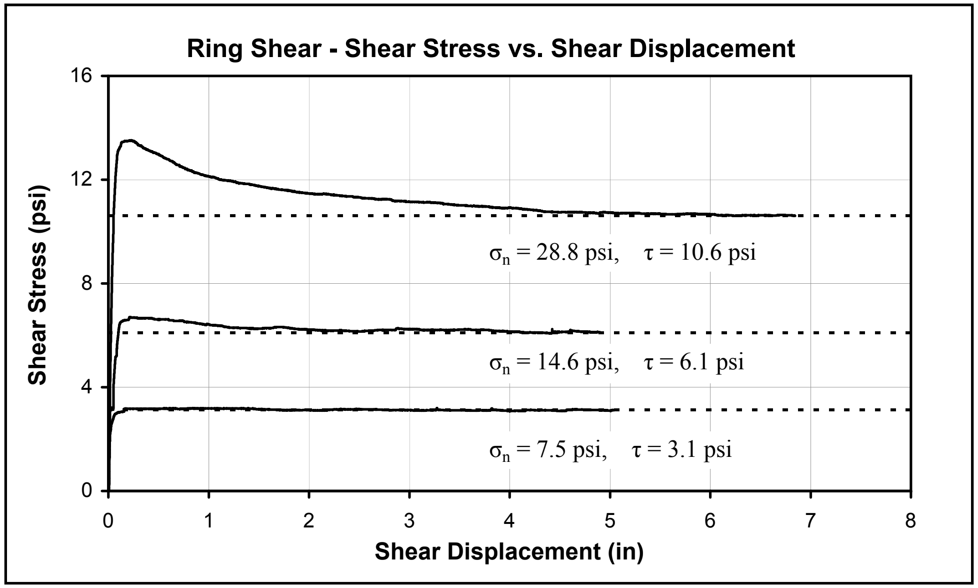
Minimum calc. shear rate =	0.0074	in/min.
Estim. failure displacement =	0.2	in

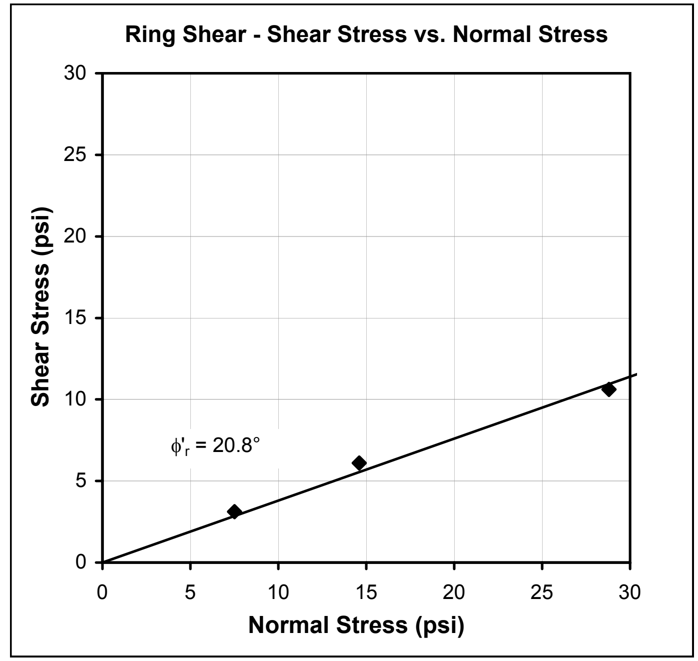
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

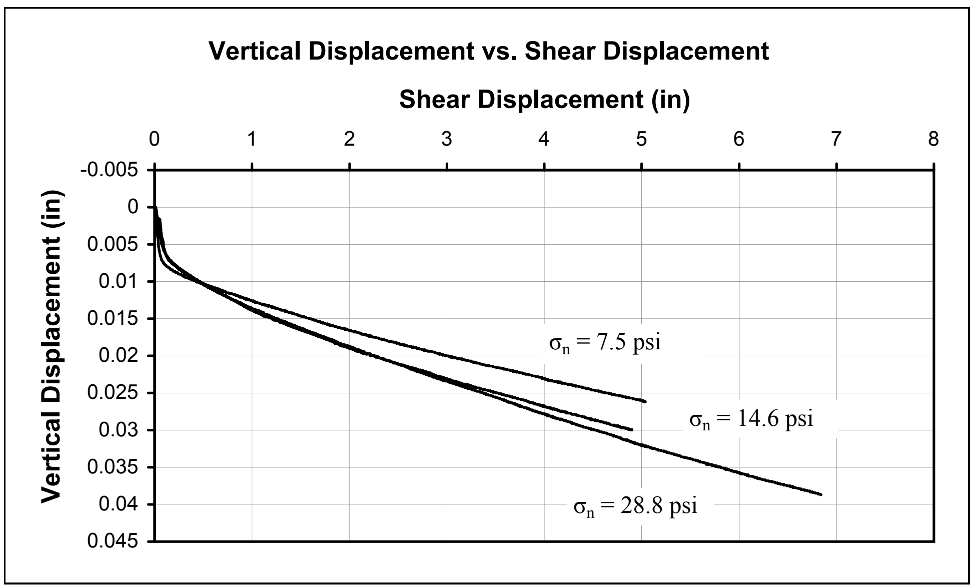
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-092305-1	2000	1081	7.50	0.54	449	3.12	0.22	0.026
R2-092305-2	4000	2103	14.61	1.05	878	6.10	0.44	0.030
R2-092305-3	7995	4146	28.79	2.07	1529	10.62	0.76	0.039

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 20.8 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Derek Martowska
Data Reduced By:	Chris Meehan
Test Started On:	9/23/05
Test Finished On:	10/11/05

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/11/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/31/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.79	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	0.4	0.0091	0.1	0.0476
4000	2103	14.61	1.05	0.7	0.0059	0.4	0.0101
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	0.8	0.0053	0.4	0.0101

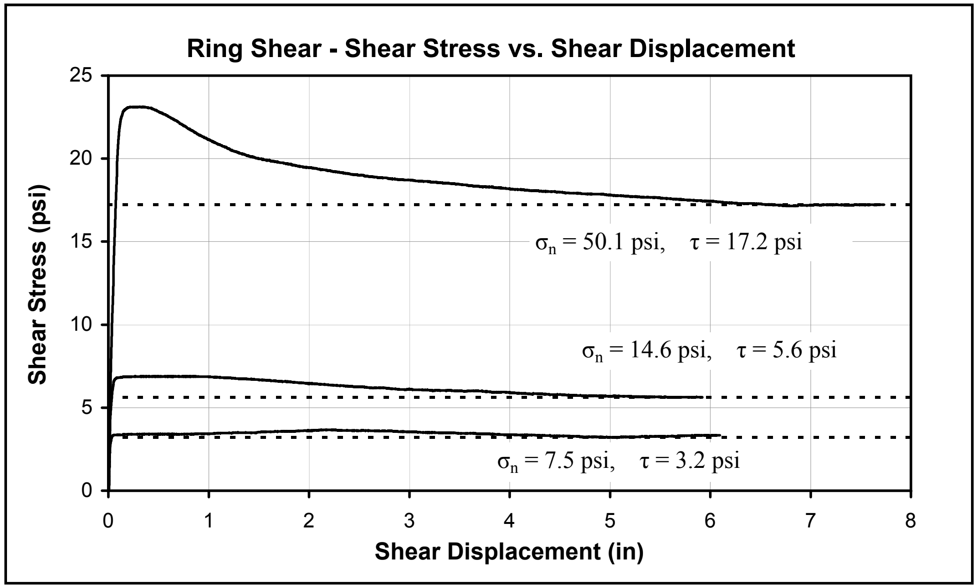
Minimum calc. shear rate =	0.0053	in/min.
Estim. Failure displacement =	0.2	in

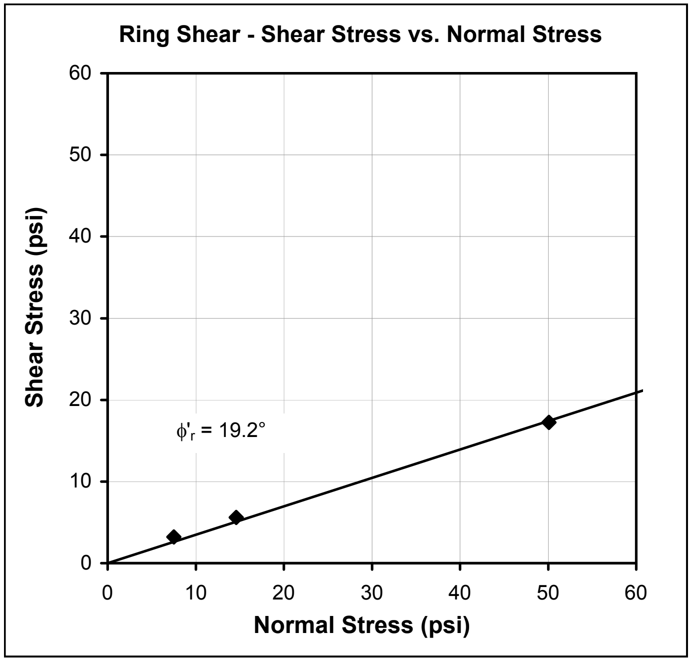
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

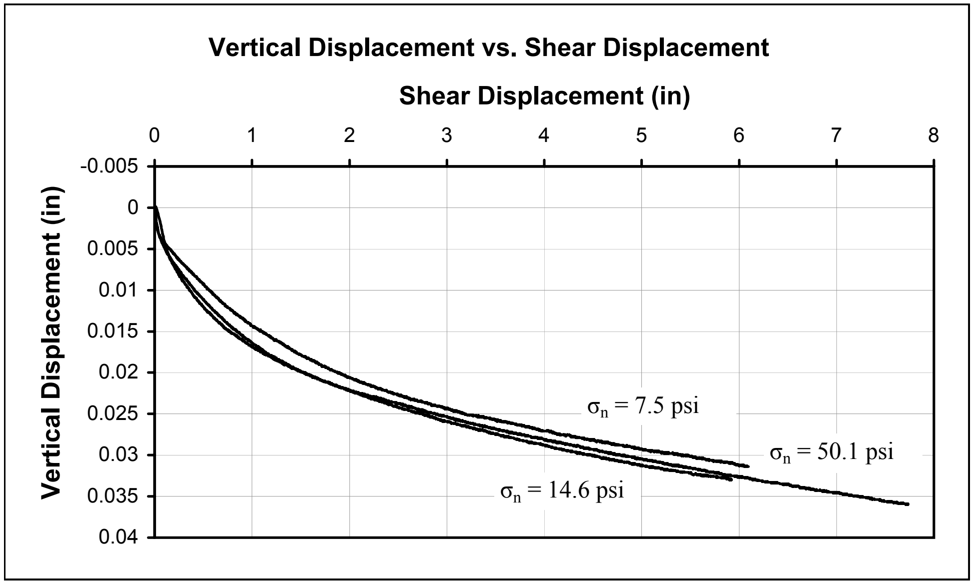
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-101105-1	2000	1081	7.50	0.54	462	3.21	0.23	0.031
R1-101105-2	4000	2103	14.61	1.05	809	5.62	0.40	0.033
R1-101105-3	13993	7213	50.09	3.61	2481	17.23	1.24	0.036

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 19.2 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Derek Martowska
Data Reduced By:	Chris Meehan
Test Started On:	10/11/05
Test Finished On:	10/31/05

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/11/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/31/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.79	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared: No		in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	0.5	0.0077	0.4	0.0110
4000	2103	14.61	1.05	0.5	0.0083	0.1	0.0268
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	0.6	0.0063	0.4	0.0110

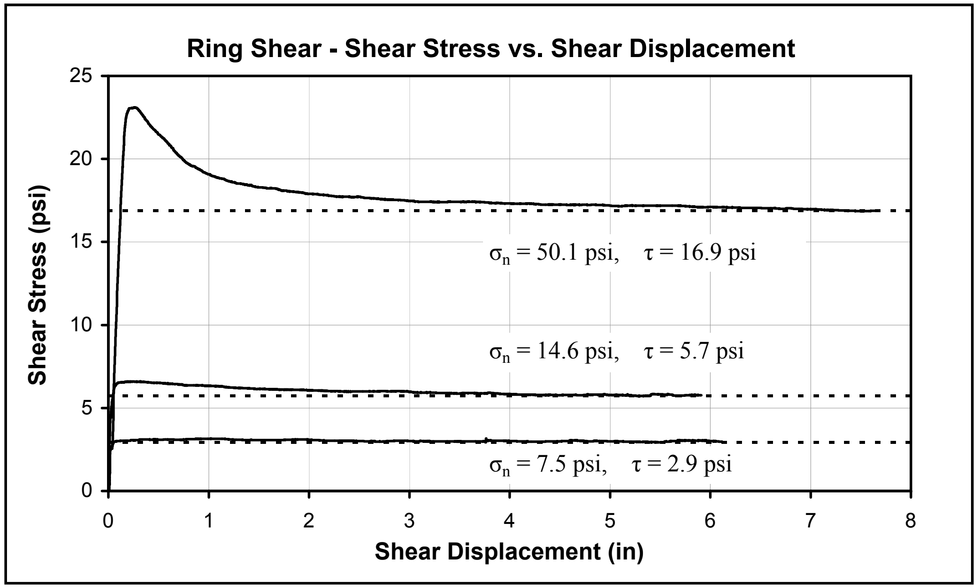
Minimum calc. shear rate =	0.0063	in/min.
Estim. failure displacement =	0.2	in

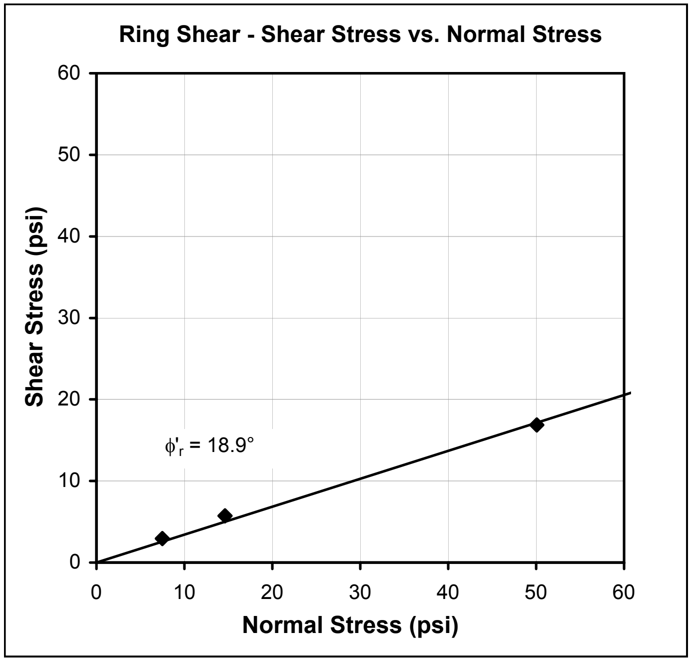
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

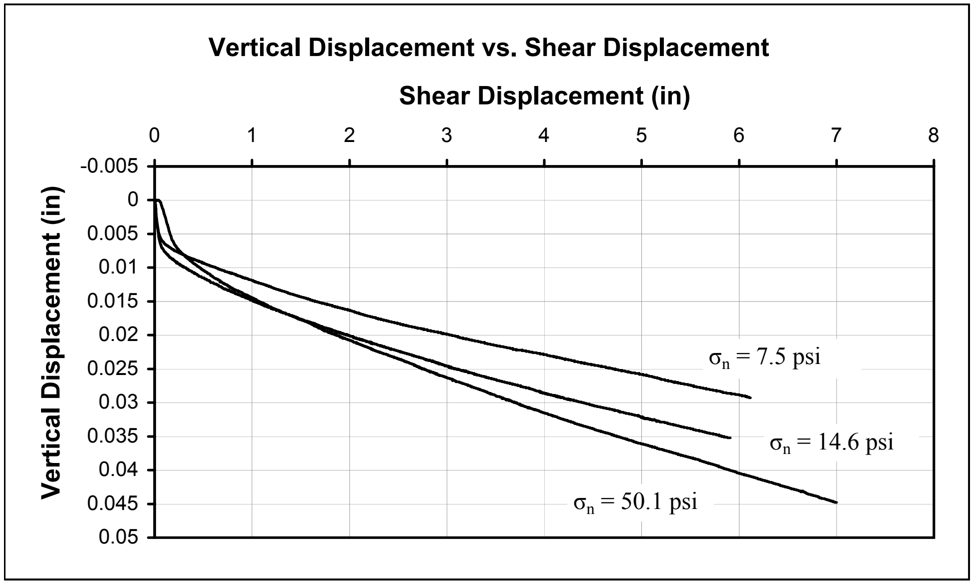
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-101105-1	2000	1081	7.50	0.54	423	2.94	0.21	0.029
R2-101105-2	4000	2103	14.61	1.05	824	5.72	0.41	0.035
R2-101105-3	13993	7213	50.09	3.61	2429	16.87	1.21	0.048

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 18.9 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Derek Martowska
Data Reduced By:	Chris Meehan
Test Started On:	10/11/05
Test Finished On:	10/31/05

Virginia Polytechnic Institute and State University

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Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/31/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	11/12/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.79	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps					Casagrande		Taylor	
Consolidation Load		Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)		(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	(test 1)	1081	7.50	0.54	0.7	0.0061	0.6	0.0071
2000	(test 2)	1081	7.50	0.54	0.5	0.0080	0.4	0.0110

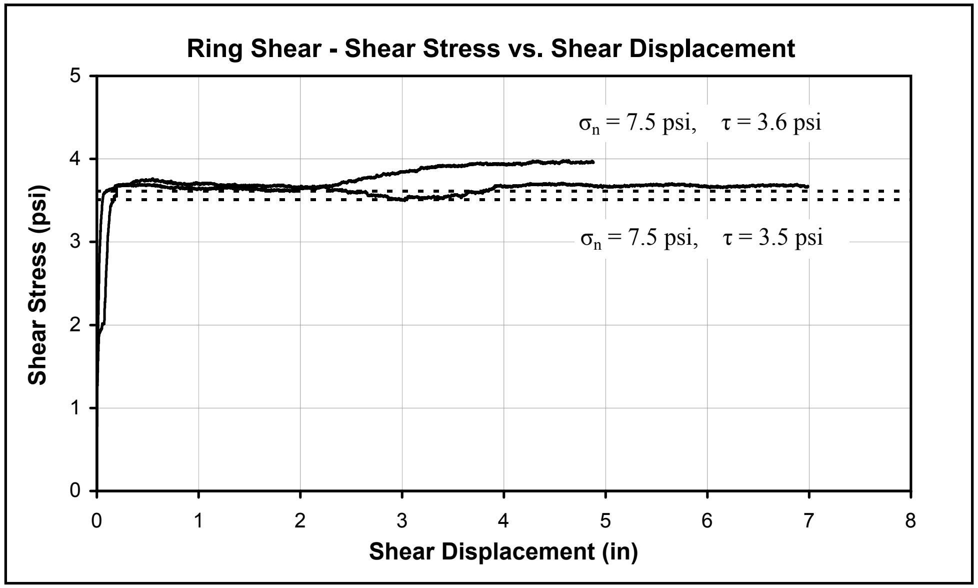
Minimum calc. shear rate =	0.0061	in/min.
Estim. failure displacement =	0.2	in

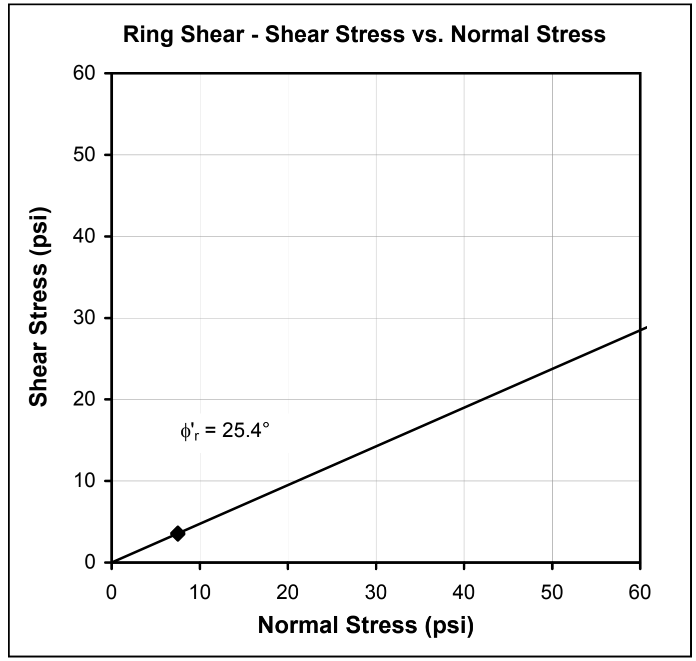
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

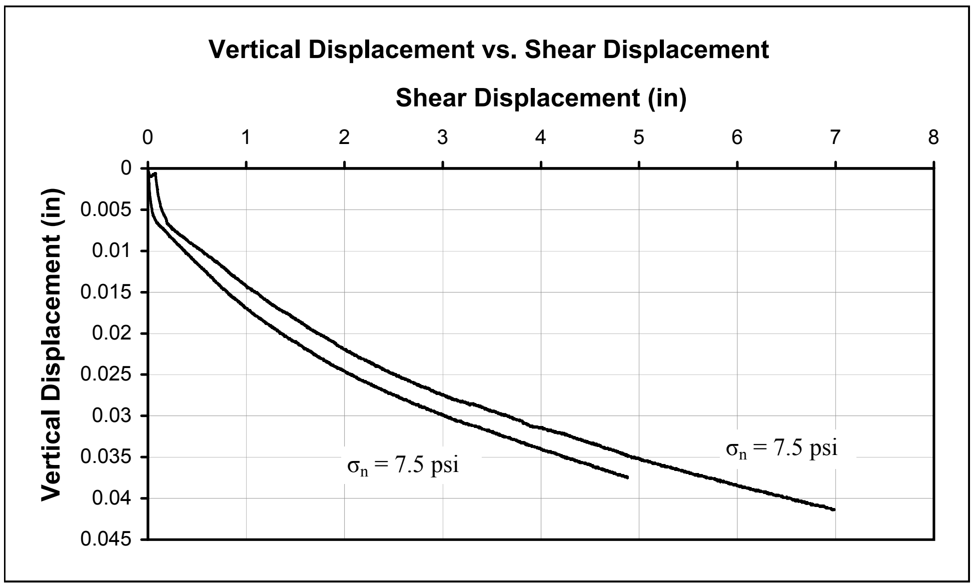
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-103105-1	2000	1081	7.50	0.54	505	3.51	0.25	0.041
R1-103105-2	2000	1081	7.50	0.54	520	3.61	0.26	0.038

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 25.4 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	VT lab testing class
Data Reduced By:	Chris Meehan
Test Started On:	10/31/05
Test Finished On:	11/12/05

Virginia Polytechnic Institute and State University

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Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/31/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	11/12/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.79	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps					Casagrande		Taylor	
Consolidation Load		Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)		(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	(test 1)	1081	7.50	0.54	0.4	0.0093	0.7	0.0059
2001	(test 2)	1081	7.50	0.54	0.8	0.0049	0.3	0.0116

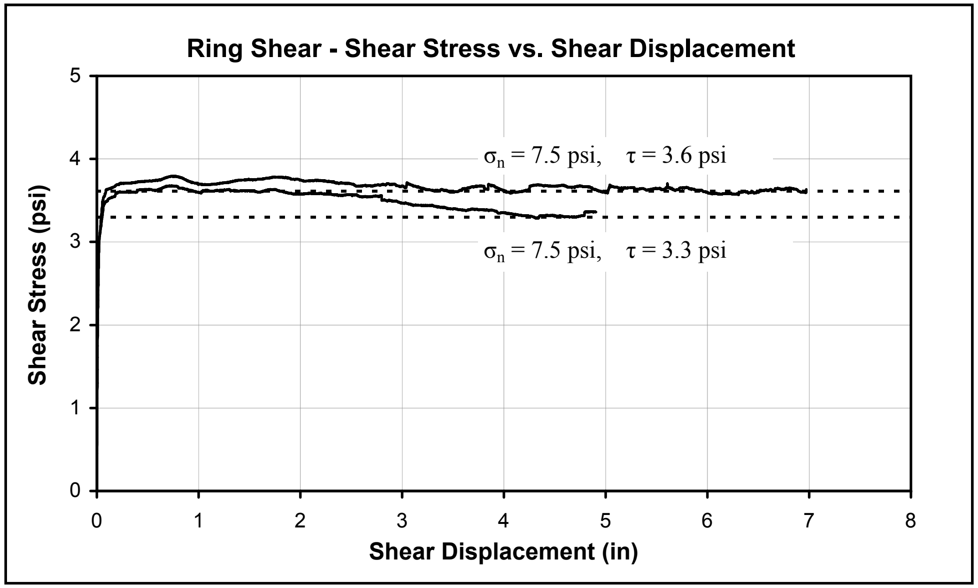
Minimum calc. shear rate =	0.0049	in/min.
Estim. failure displacement =	0.2	in

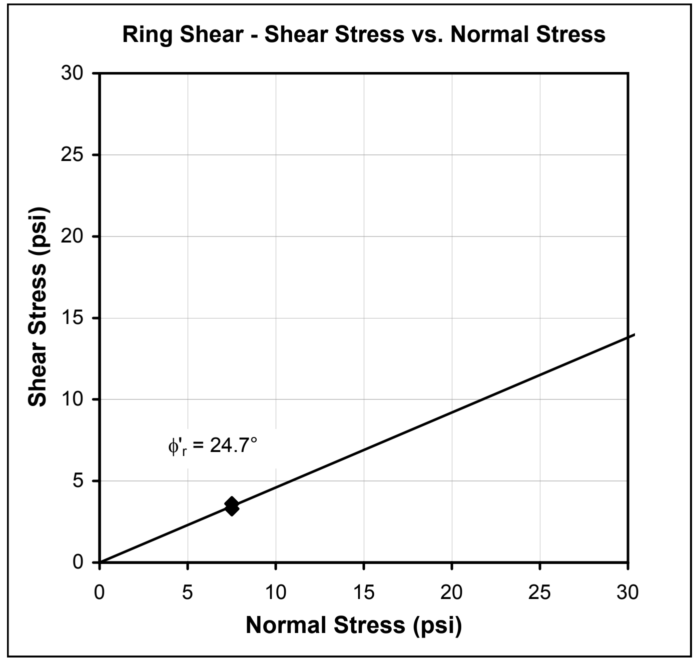
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

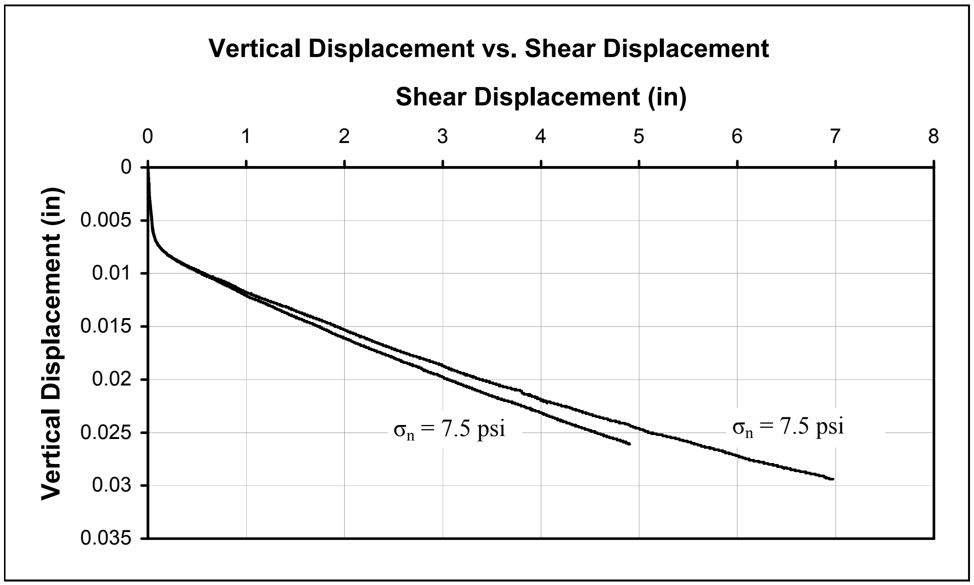
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-103105-1	2000	1081	7.50	0.54	520	3.61	0.26	0.029
R2-103105-2	2000	1081	7.50	0.54	475	3.30	0.24	0.026

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 24.7 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	Rancho Solano Clay #2
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	VT lab testing class
Data Reduced By:	Chris Meehan
Test Started On:	10/31/05
Test Finished On:	11/12/05

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/29/04
Sample I.D./Loc.:	San Francisco Bay Mud Bucket Sample	Finished:	10/17/04
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity	2.70	
Initial Thickness of Specimen	0.2	(in.)
Inner Radius of Specimen	1.38	(in.)
Outer Radius of Specimen	1.97	(in.)
Presheared: No		in/min
Multistage Loading or New Specimen for Each Point?	New Specimen	
Failure Surface Location	Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54	-	-	-	-
4000	2103	14.61	1.05				
7995	4146	28.79	2.07	2.2	0.0018	1.5	0.0027

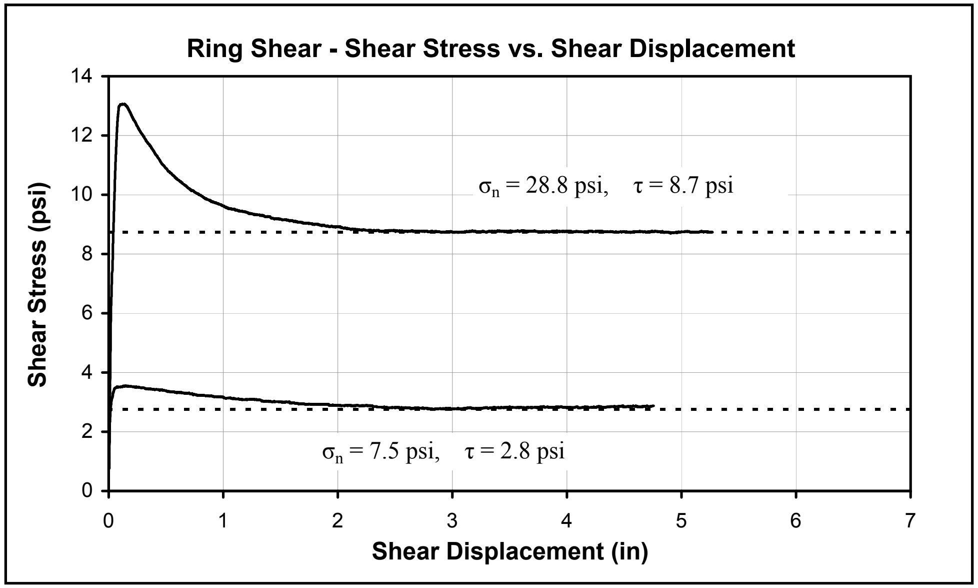
Minimum calc. shear rate =	0.0018	in/min.
Estim. failure displacement =	0.2	in

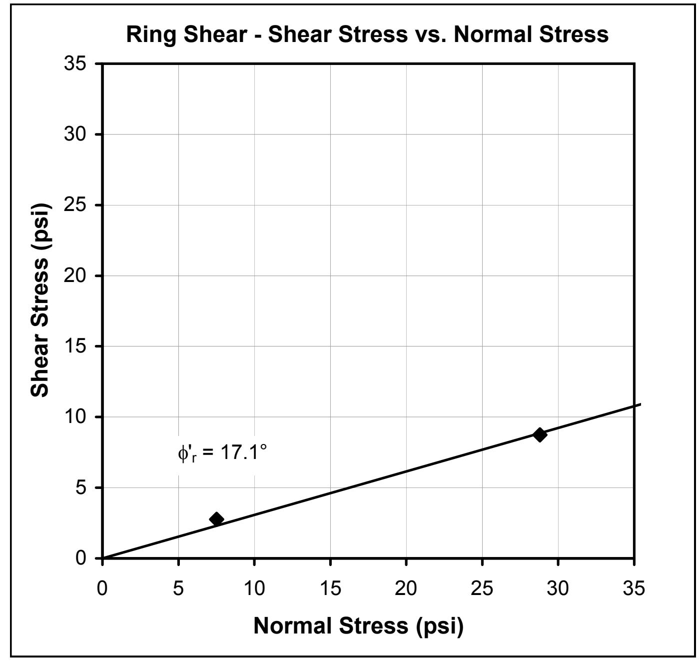
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

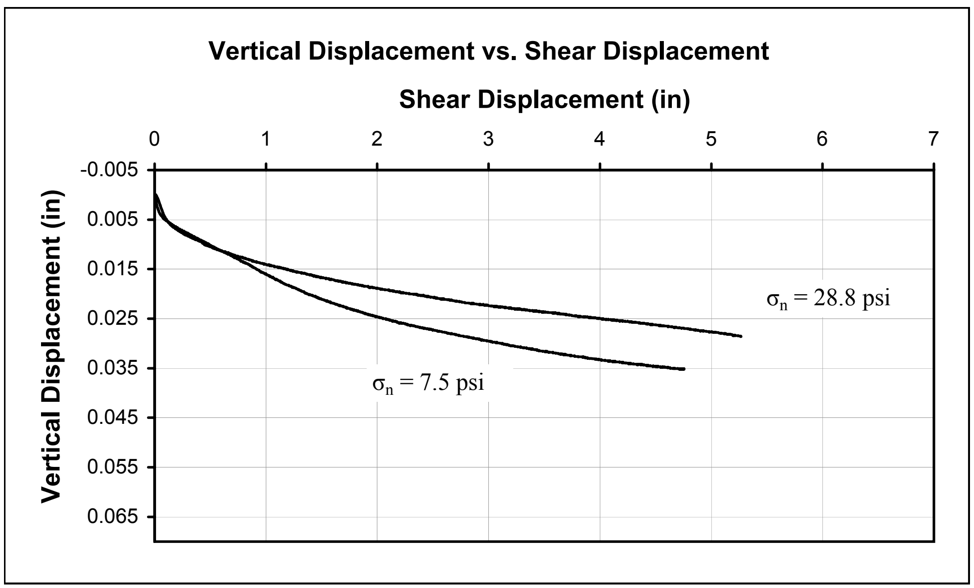
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-092904-1	2000	1081	7.50	0.54	397	2.76	0.20	0.035
R1-092904-2	7995	4146	28.79	2.07	1257	8.73	0.63	0.029

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

ϕ'_r = 17.1 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	SFBM Bucket Sample
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	9/29/04
Test Finished On:	10/17/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		Started:	9/29/04
Sample I.D./Loc.:	San Francisco Bay Mud Bucket Sample		Finished:	10/17/04
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Bromhead Ring Shear #2	

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.70	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1999	1080	7.50	0.54	1.9	0.0021	1.8	0.0022
3998	2102	14.60	1.05	-	-	-	-
7995	4146	28.79	2.07	2.0	0.0020	1.7	0.0023

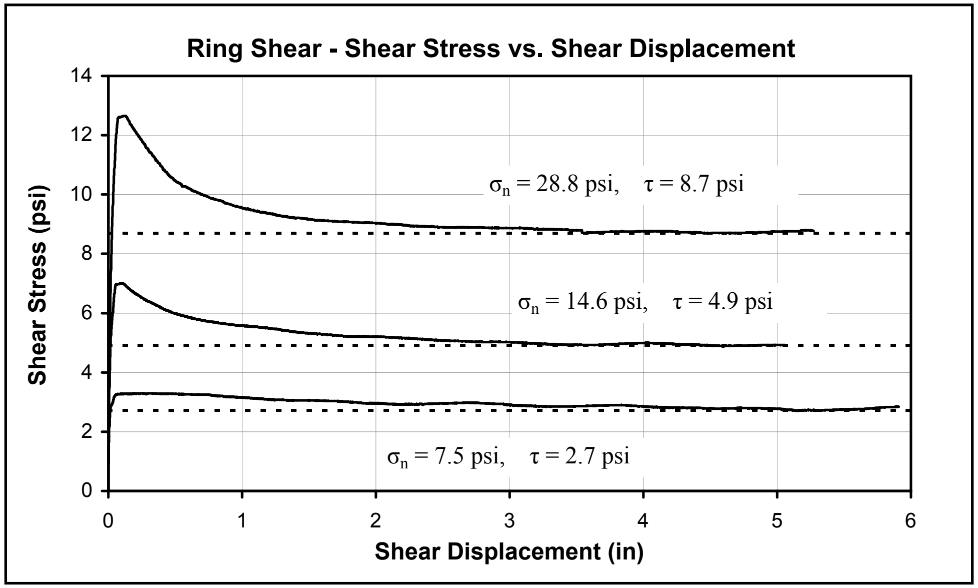
Minimum calc. shear rate =	0.0020	in/min.
Estim. failure displacement =	0.2	in

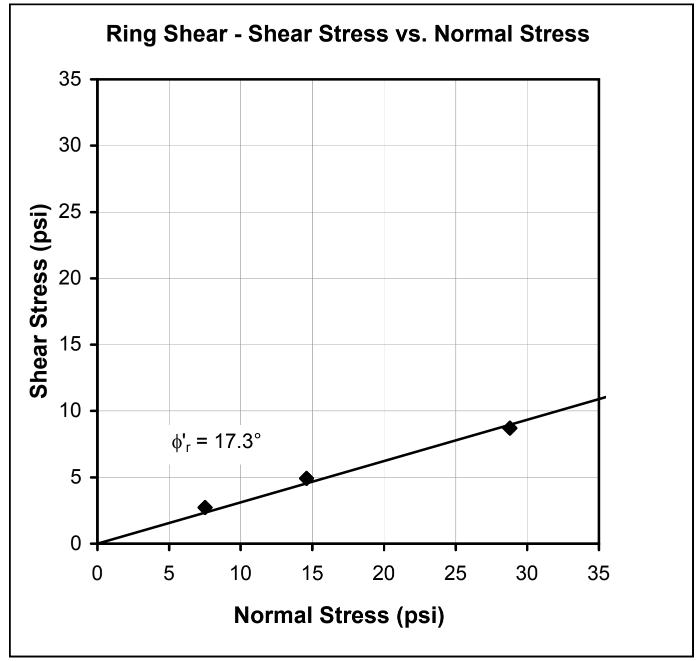
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

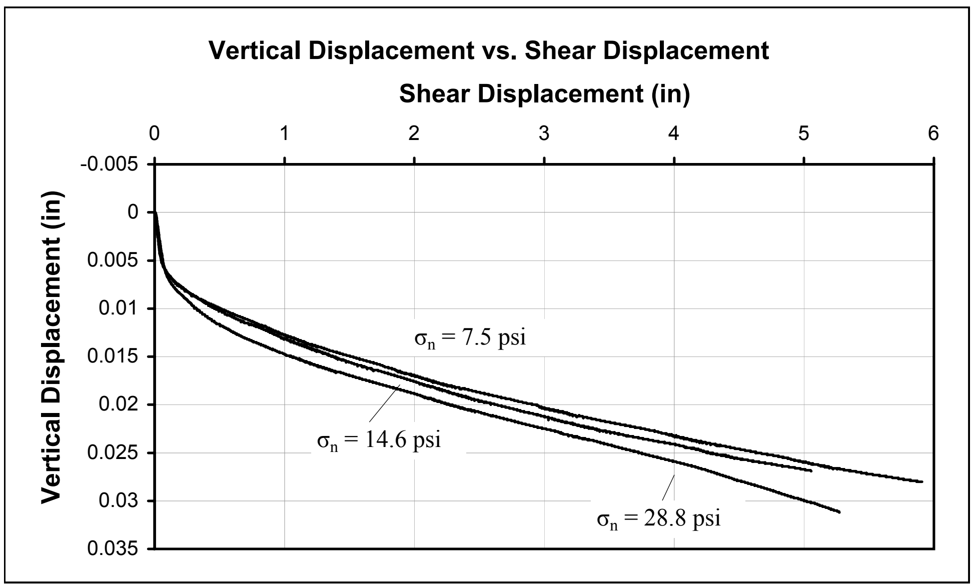
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-092904-1	1999	1080	7.50	0.54	392	2.72	0.20	0.028
R2-092904-2	3998	2102	14.60	1.05	708	4.92	0.35	0.027
R2-092904-3	7995	4146	28.79	2.07	1253	8.70	0.63	0.031

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used fpr ring shear tests.

ϕ'_r = 17.3 deg.
(based on a best fit line for c'_r = 0)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	SFBM Bucket Sample
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	9/29/04
Test Finished On:	10/17/04

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Geotechnical Engineering Laboratory
Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/18/04
Sample I.D./Loc.:	San Francisco Bay Mud Bucket Sample	Finished:	11/15/04
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Bromhead Ring Shear #1

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.70	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
2000	1081	7.50	0.54				
4000 (test 1)	2103	14.61	1.05	1.7	0.0024	1.5	0.0027
4000 (test 4)	2103	14.61	1.05	1.6	0.0025	1.0	0.0039
7995	4146	28.79	2.07				
13993	7213	50.09	3.61	2.0	0.0020	1.5	0.0027
23988	12324	85.58	6.16	1.7	0.0023	1.5	0.0027

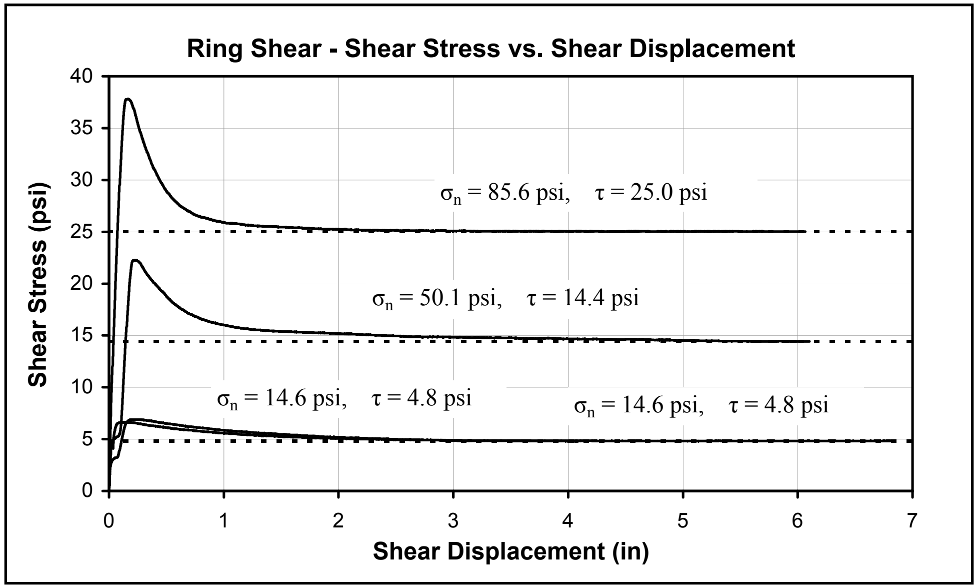
Minimum calc. shear rate =	0.0020	in/min.
Estim. failure displacement =	0.2	in

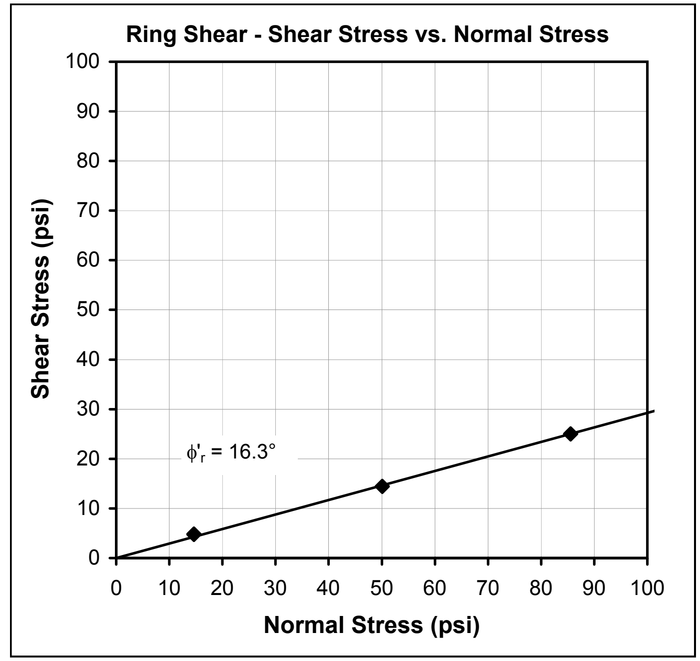
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

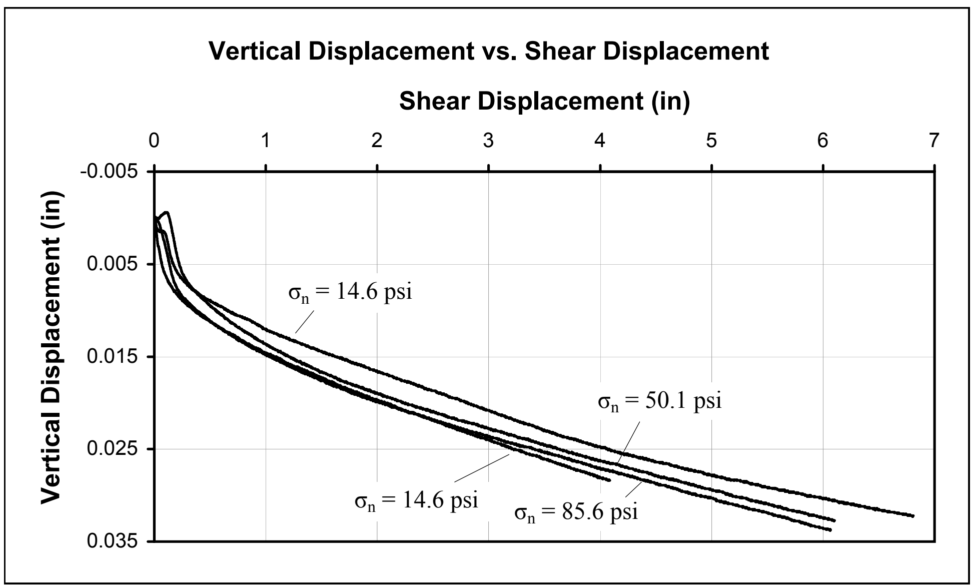
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R1-101804-1	4000	2103	14.61	1.05	696	4.83	0.35	0.032
R1-101804-2	13993	7213	50.09	3.61	2078	14.43	1.04	0.033
R1-101804-3	23988	12324	85.58	6.16	3604	25.03	1.80	0.034
R1-101804-4	4000	2103	14.61	1.05	691	4.80	0.35	0.035

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

$\phi'_r =$ 16.3 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	SFBM Bucket Sample
Device Used:	WF Bromhead Ring Shear #1
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	10/18/04
Test Finished On:	11/15/04

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Ring Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/18/04
Sample I.D./Loc.:	San Francisco Bay Mud Bucket Sample	Finished:	11/15/04
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Bromhead Ring Shear #2

Initial Water Content of Prepared Specimen		
Wt. of Moist Soil + Tare	Water content not measured for this test.	
Wt. of Dry Soil + Tare		
Wt. of Tare		
Wt. of Moist Soil		
Wt. of Dry Soil		
Water Content, w%		
Wet Weight of Entire Specimen		(g)

Assumed Specific Gravity		2.70	
Initial Thickness of Specimen		0.2	(in.)
Inner Radius of Specimen		1.38	(in.)
Outer Radius of Specimen		1.97	(in.)
Presheared	No		in/min
Multistage Loading or New Specimen for Each Point?		New Specimen	
Failure Surface Location		Top of Specimen	

Consolidation Steps				Casagrande		Taylor	
Consolidation Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(g)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
1999	1080	7.50	0.54				
3998	2102	14.60	1.05	1.8	0.0022	0.9	0.0045
7995	4146	28.79	2.07				
13990	7211	50.08	3.61	1.9	0.0021	1.7	0.0023
23985	12322	85.57	6.16	2.0	0.0020	1.3	0.0030

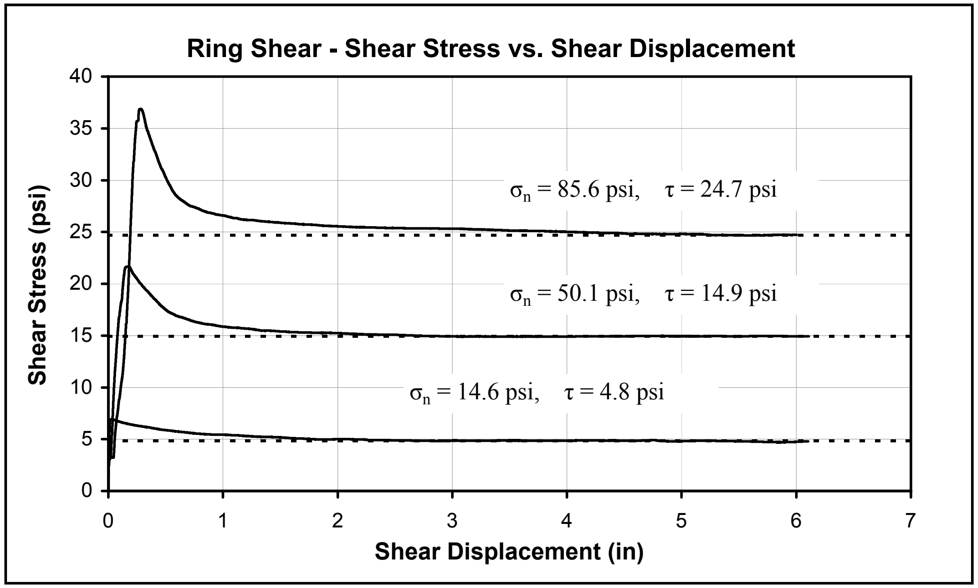
Minimum calc. shear rate =	0.0020	in/min.
Estim. failure displacement =	0.2	in

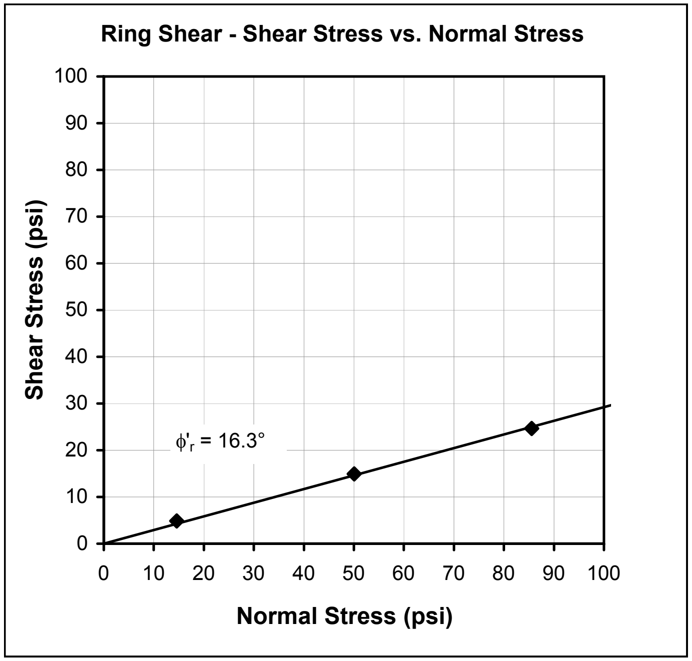
Test performed at shear rate =	0.00071	in/min.
Test performed at shear rate =	0.024	deg/min.

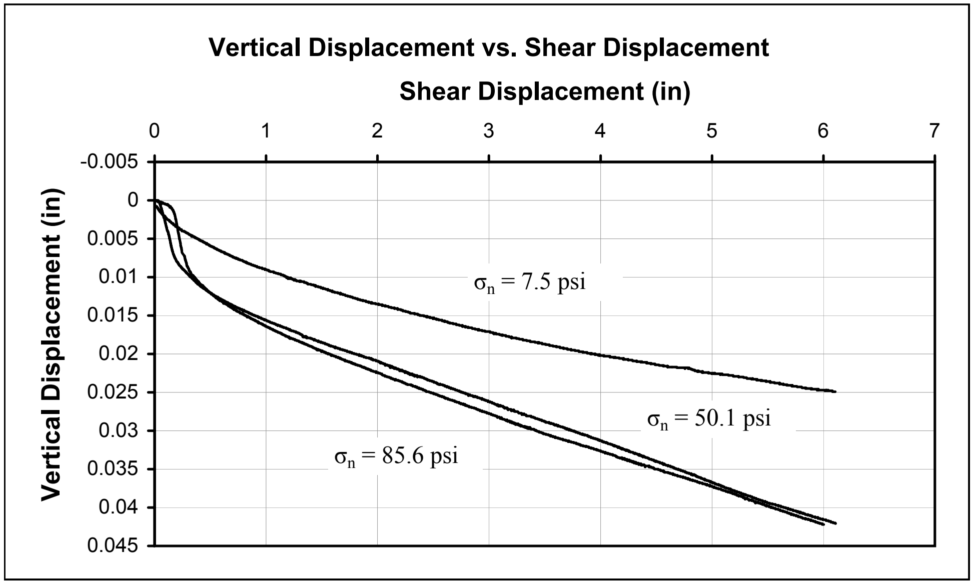
Test Number	Normal Load	Normal Stress			Residual Shear Stress			ΔH
	(g)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
R2-101804-1	13990	7211	50.08	3.61	2151	14.94	1.08	0.042
R2-101804-2	23985	12322	85.57	6.16	3554	24.68	1.78	0.042
R2-101804-3	3998	2102	14.60	1.05	697	4.84	0.35	0.025

Notes: Specimen was remolded at LL, and pushed through the #40 sieve.
Specimen was preconsolidated to approximately 50 psi prior to placement in ring shear apparatus.
Modified platen used for ring shear tests.

$\phi'_r =$ 16.3 deg.
(based on a best fit line for $c'_r = 0$)







Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Soil Being Tested:	SFBM Bucket Sample
Device Used:	WF Bromhead Ring Shear #2
Test Performed By:	Chris Meehan
Data Reduced By:	Chris Meehan
Test Started On:	10/18/04
Test Finished On:	11/15/04

APPENDIX B

DIRECT SHEAR DATA

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	5/12/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	5/19/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

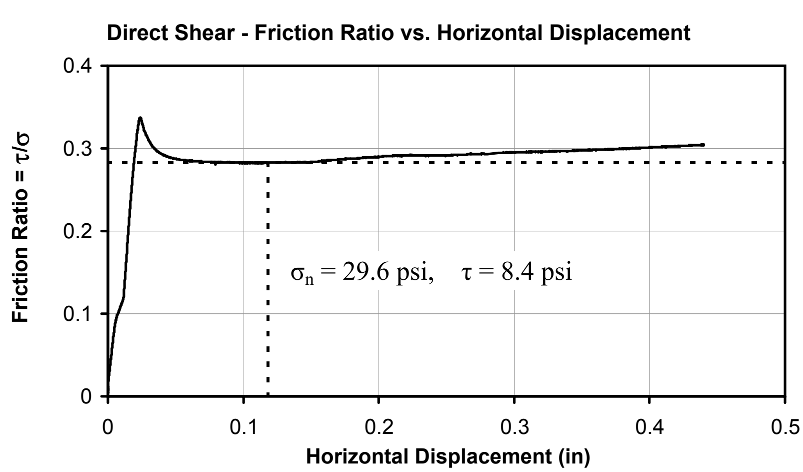
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Hanger Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(kg)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
10.0	2580	17.91	1.29	Shear rate based on data from earlier tests.			
20.0	4530	31.46	2.27				
40.0	8430	58.54	4.22				
70.0	14286	99.21	7.14				

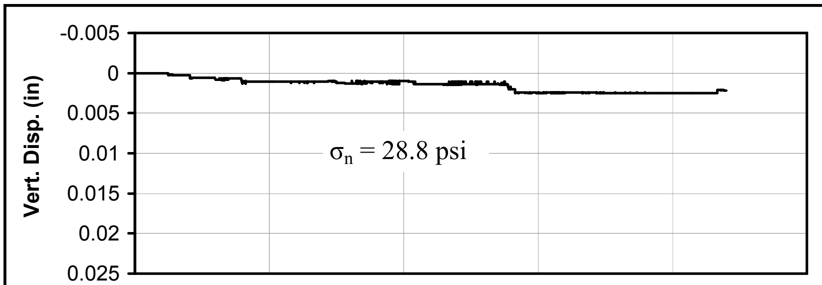
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-051204-1	18.0	4266	29.62	2.13	1206	8.38	0.60	0.0022



Initial Normal Stress:	
28.75	(psi)
Friction Ratio at Failure	
0.2828	(none)
Displacement at Failure:	
0.118	(in)
Arctan(Friction Ratio):	
15.8	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		Started:	5/19/04
Sample I.D./Loc.:	Rancho Solano Clay #1		Finished:	5/27/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1	

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

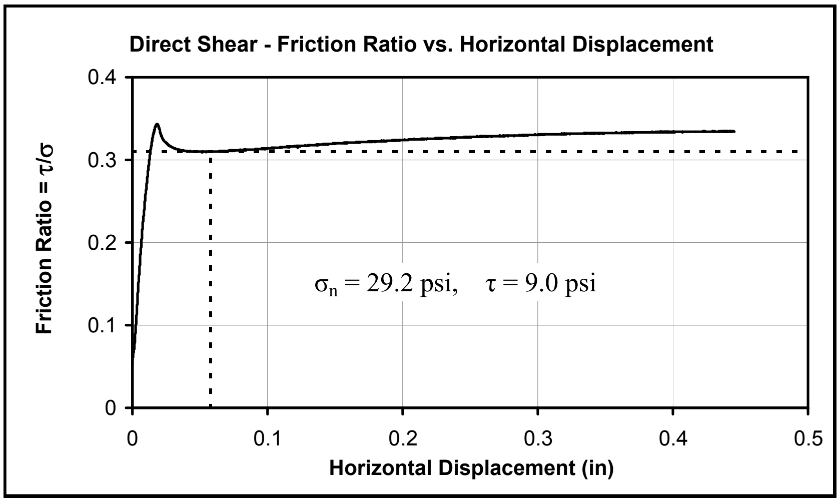
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Hanger Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(kg)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
10.0	2583	17.94	1.29	Shear rate based on data from earlier tests.			
20.0	4541	31.53	2.27				
40.0	8433	58.56	4.22				
70.0	14284	99.19	7.14				

Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

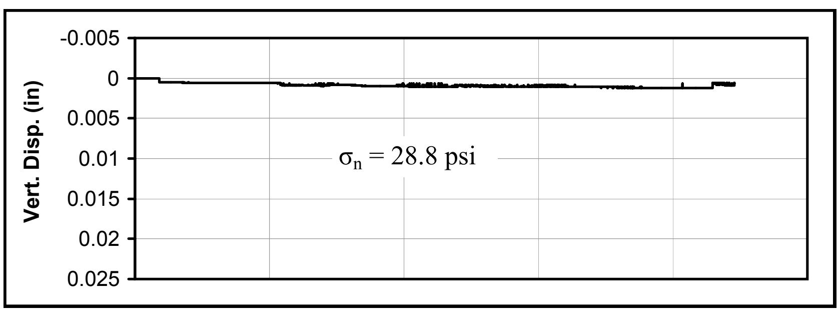
Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-051904-1	18.0	4201	29.17	2.10	1302	9.04	0.65	0.0009



Initial Normal Stress:	
28.75	(psi)
Friction Ratio at Failure	
0.3099	(none)
Displacement at Failure:	
0.058	(in)
Arctan(Friction Ratio):	
17.2	(deg)

Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.



Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		Started:	5/27/04
Sample I.D./Loc.:	Rancho Solano Clay #1		Finished:	6/3/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1	

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

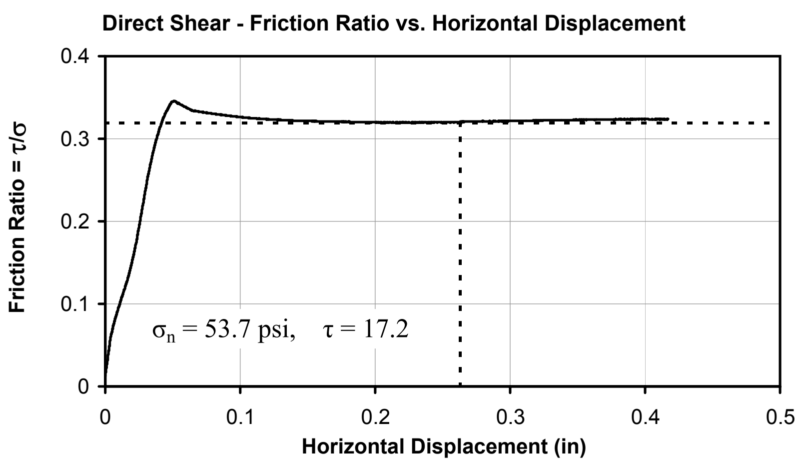
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Hanger Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(kg)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
10.0	2583	17.94	1.29	Shear rate based on data from earlier tests.			
20.0	4541	31.53	2.27				
40.0	8433	58.56	4.22				
70.0	14284	99.19	7.14				

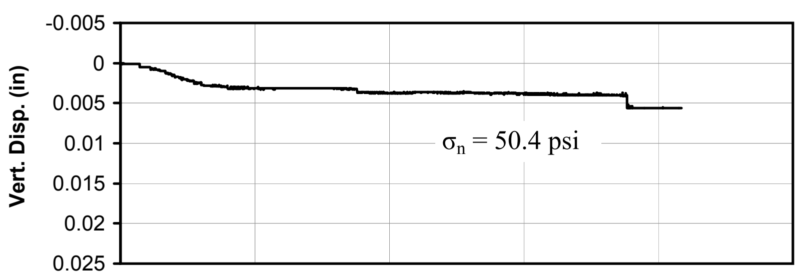
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-052704-1	34.0	7771	53.96	3.89	2484	17.25	1.24	0.0056



Initial Normal Stress:	
50.42	(psi)
Friction Ratio at Failure	
0.3197	(none)
Displacement at Failure:	
0.263	(in)
Arctan(Friction Ratio):	
17.7	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/10/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	6/12/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

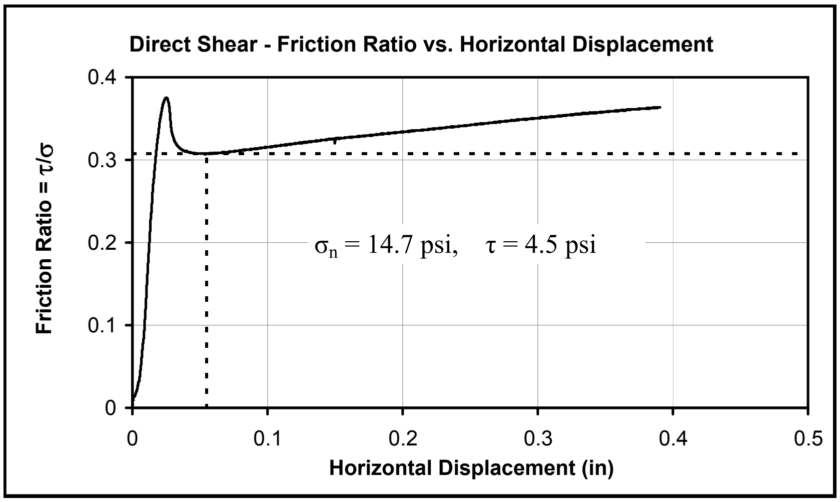
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

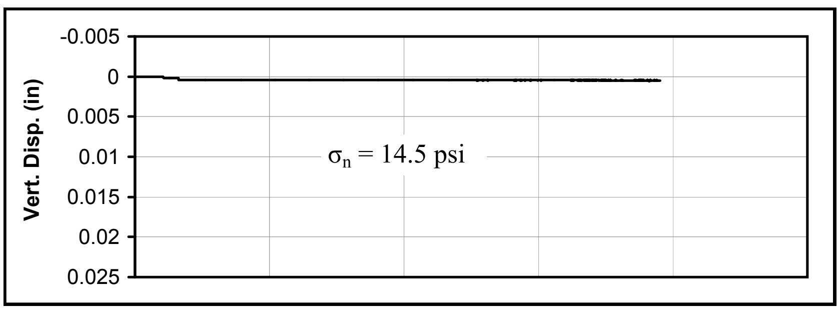
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-061004-1	7.5	2123	14.74	1.06	652	4.53	0.33	0.0005



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.3074	(none)
Displacement at Failure:	
0.055	(in)
Arctan(Friction Ratio):	
17.1	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 6/7/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	6/27/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	7/2/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

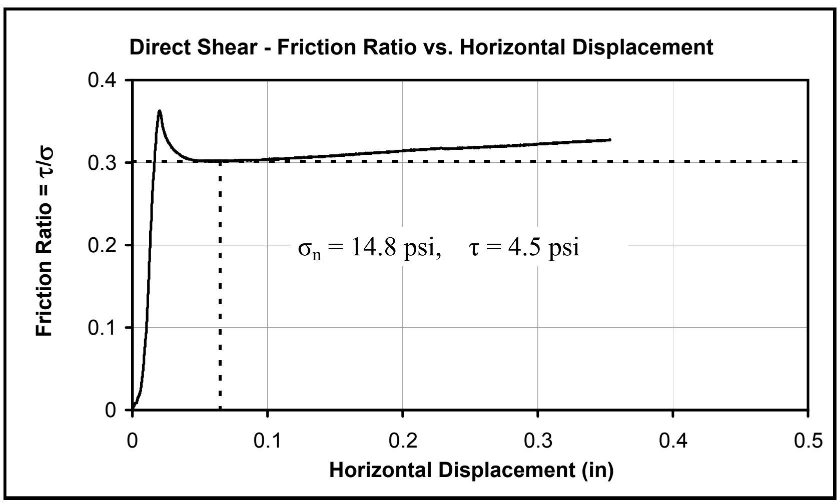
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

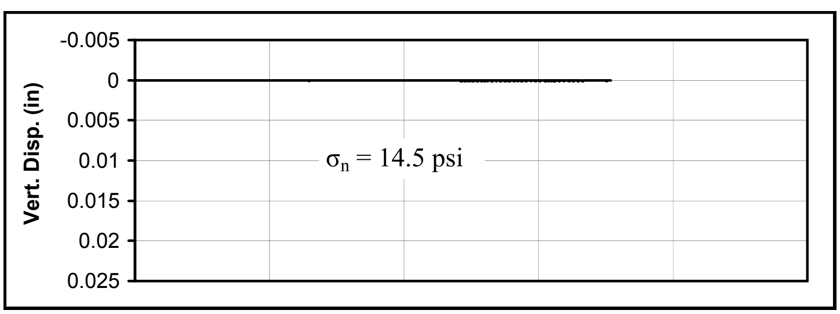
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-062704-1	7.5	2128	14.78	1.06	642	4.46	0.32	0.0001



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.3018	(none)
Displacement at Failure:	
0.065	(in)
Arctan(Friction Ratio):	
16.8	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 6/27/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	7/30/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/4/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

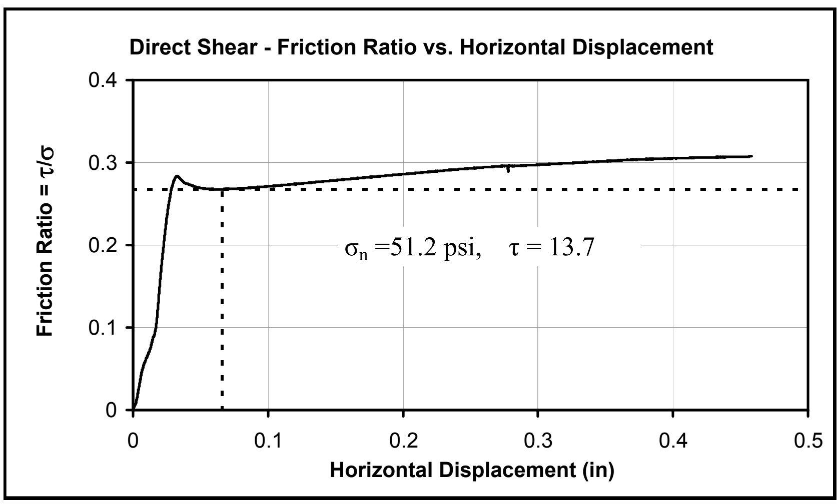
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

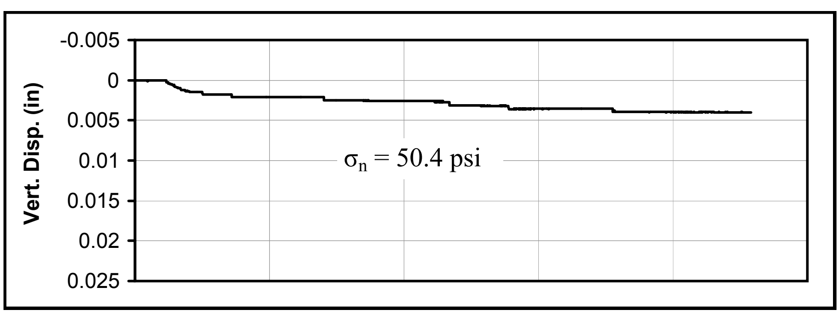
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-073004-1	34.0	7378	51.23	3.69	1973	13.70	0.99	0.0040



Initial Normal Stress:	
50.39	(psi)
Friction Ratio at Failure	
0.2675	(none)
Displacement at Failure:	
0.066	(in)
Arctan(Friction Ratio):	
15.0	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 7/27/04.

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/16/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/19/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

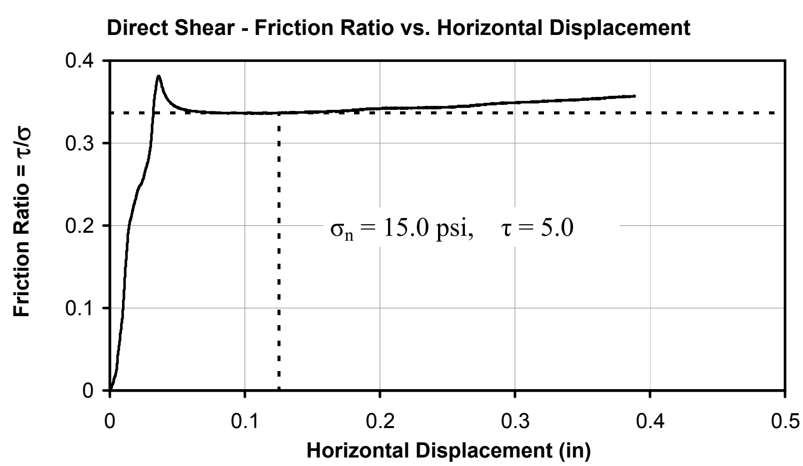
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

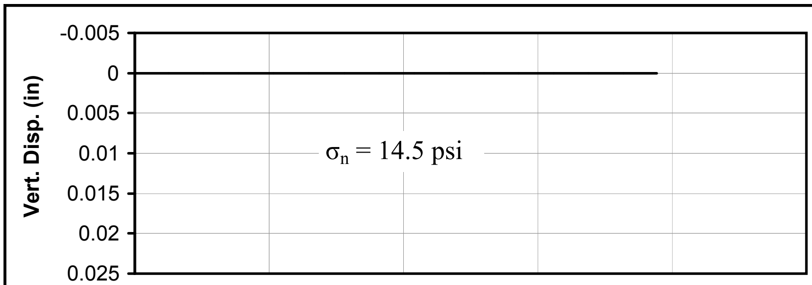
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-081604-2	7.5	2161	15.01	1.08	726	5.04	0.36	0.0001



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.3361	(none)
Displacement at Failure:	
0.125	(in)
Arctan(Friction Ratio):	
18.6	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/13/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/19/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/21/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

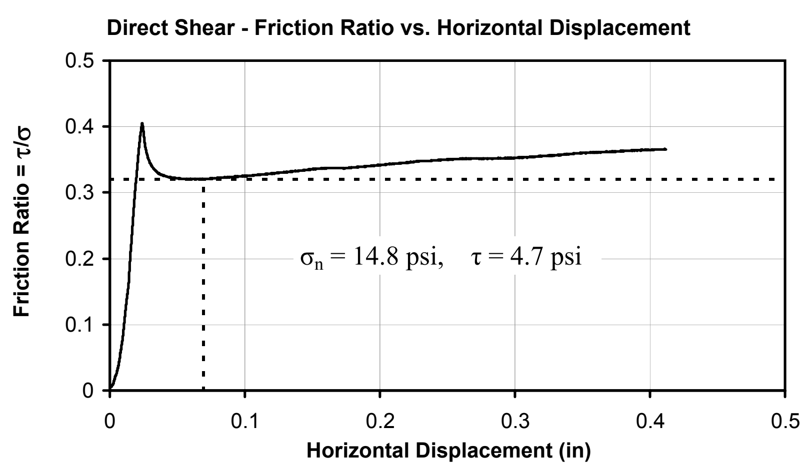
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

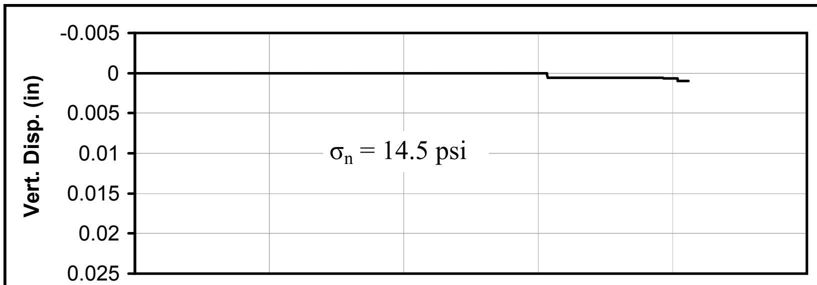
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-081904-1	7.5	2130	14.79	1.07	682	4.74	0.34	0.0010



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.3202	(none)
Displacement at Failure:	
0.069	(in)
Arctan(Friction Ratio):	
17.8	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/16/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/23/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/26/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

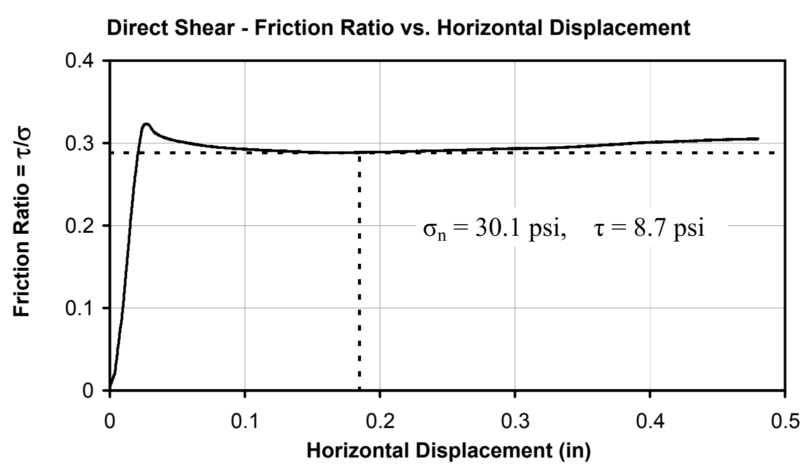
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

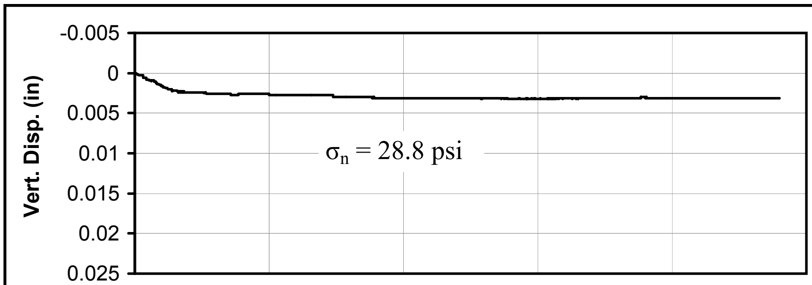
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-082304-2	18.0	4340	30.14	2.17	1252	8.70	0.63	0.0031



Initial Normal Stress:	
28.75	(psi)
Friction Ratio at Failure	
0.2885	(none)
Displacement at Failure:	
0.185	(in)
Arctan(Friction Ratio):	
16.1	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/19/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/3/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/6/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

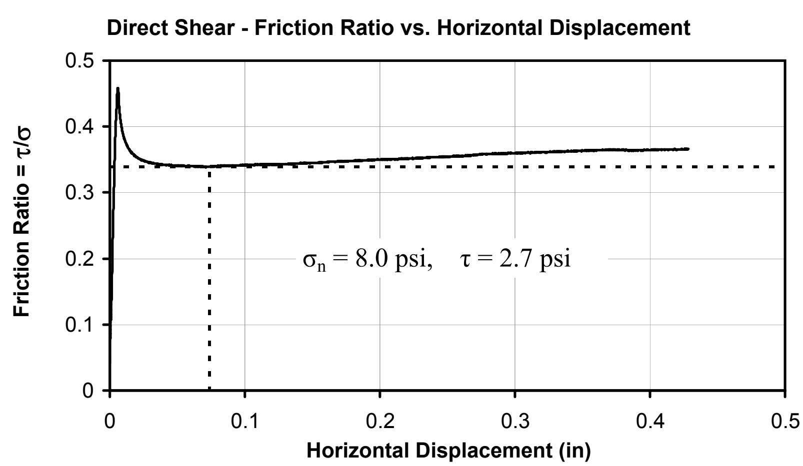
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

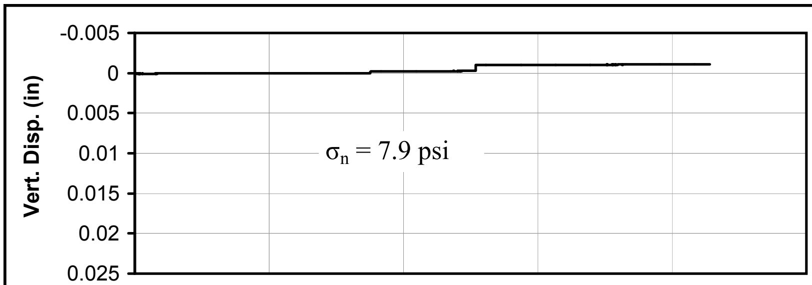
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-090304-1	2.3	1156	8.03	0.58	392	2.72	0.20	-0.0011



Initial Normal Stress:	
7.88	(psi)
Friction Ratio at Failure	
0.3392	(none)
Displacement at Failure:	
0.074	(in)
Arctan(Friction Ratio):	
18.7	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/30/04.

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/15/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/18/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

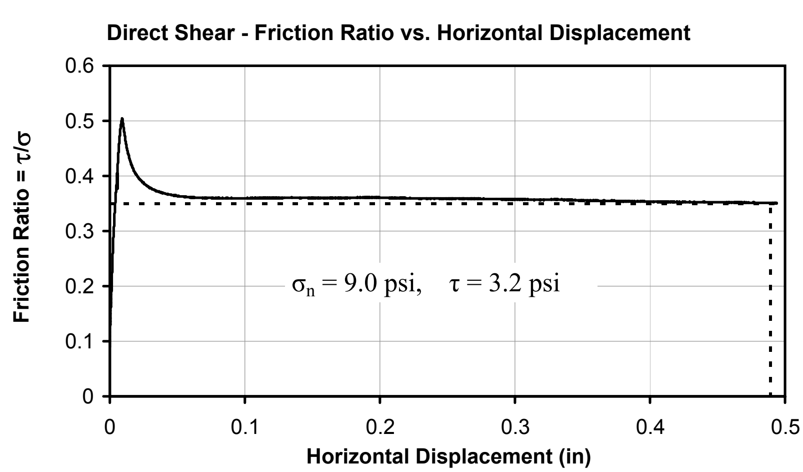
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

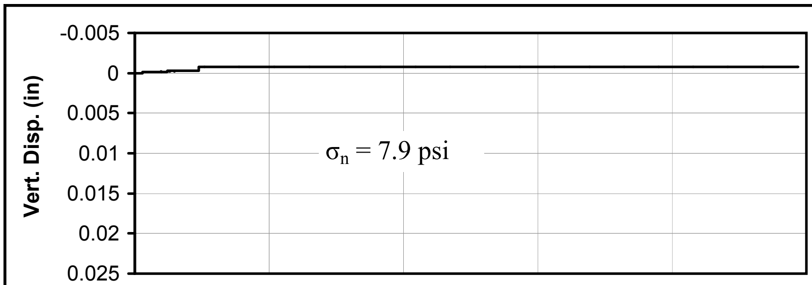
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-091504-1	2.3	1293	8.98	0.65	453	3.15	0.23	-0.0008



Initial Normal Stress:	
7.88	(psi)
Friction Ratio at Failure	
0.3505	(none)
Displacement at Failure:	
0.489	(in)
Arctan(Friction Ratio):	
19.3	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 9/8/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/27/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	9/29/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

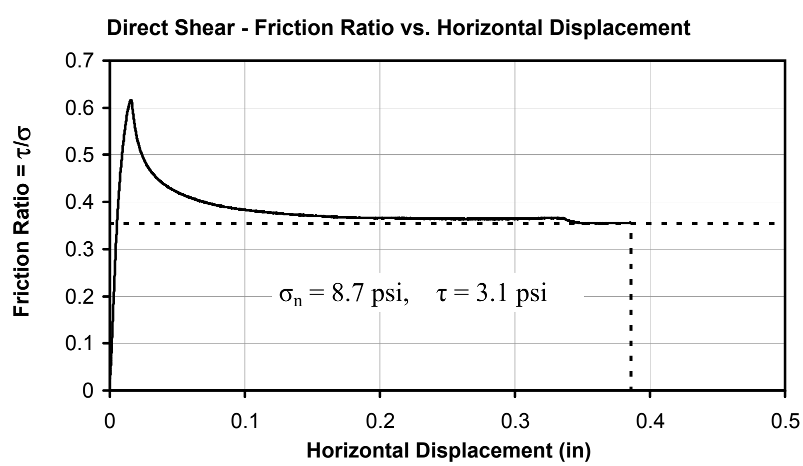
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

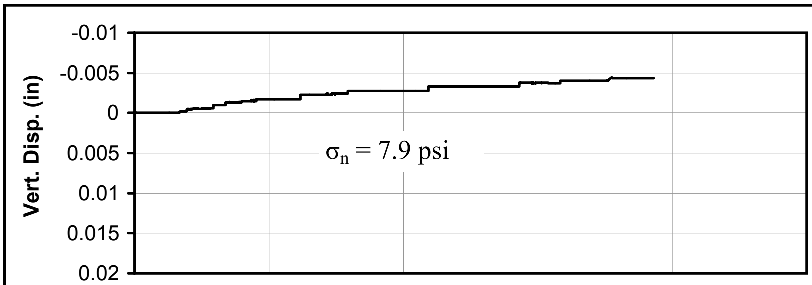
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-092704-1	2.3	1256	8.72	0.63	446	3.10	0.22	-0.0043



Initial Normal Stress:	
7.88	(psi)
Friction Ratio at Failure	
0.3552	(none)
Displacement at Failure:	
0.386	(in)
Arctan(Friction Ratio):	
19.6	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 9/24/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		Started:	10/1/04
Sample I.D./Loc.:	Rancho Solano Clay #1		Finished:	10/3/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1	

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

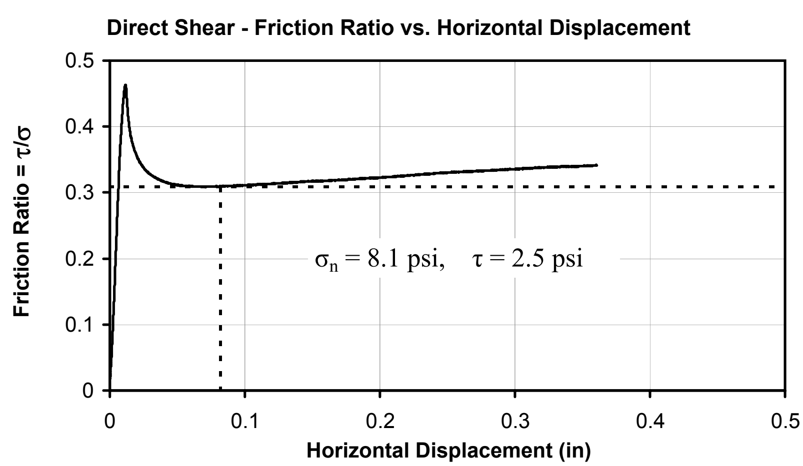
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

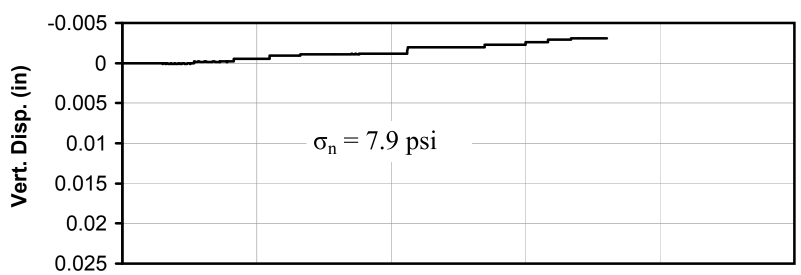
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-100104-1	2.3	1159	8.05	0.58	358	2.48	0.18	-0.0031



Initial Normal Stress:	
7.88	(psi)
Friction Ratio at Failure	
0.3087	(none)
Displacement at Failure:	
0.082	(in)
Arctan(Friction Ratio):	
17.2	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 9/24/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/22/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	9/25/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

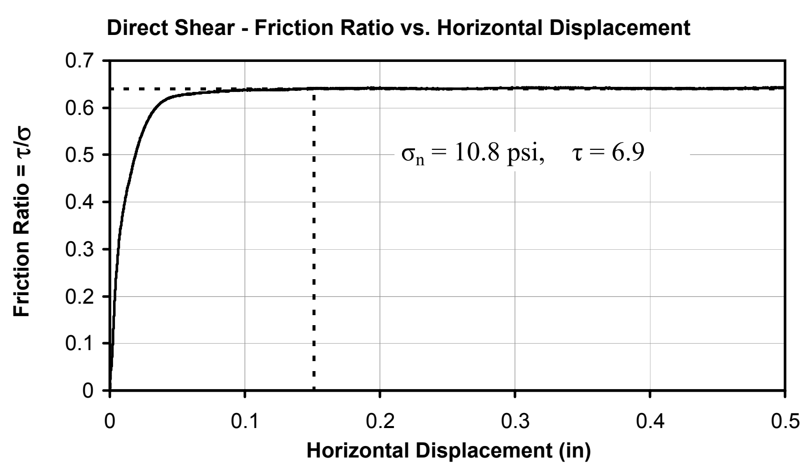
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

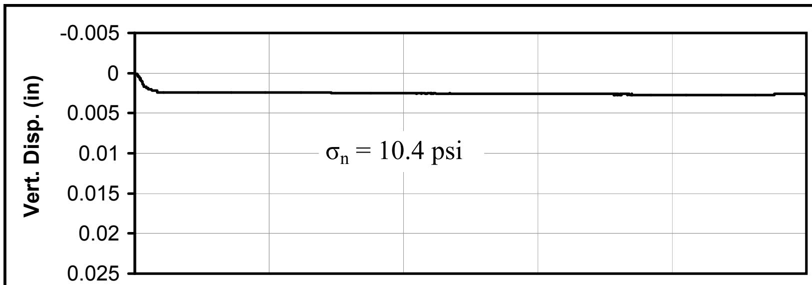
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-092205-1	4.2	1561	10.84	0.78	1000	6.94	0.50	0.0026



Initial Normal Stress:	
10.43	(psi)
Friction Ratio at Failure	
0.6406	(none)
Displacement at Failure:	
0.151	(in)
Arctan(Friction Ratio):	
32.6	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 9/20/05.

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	9/26/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	9/29/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

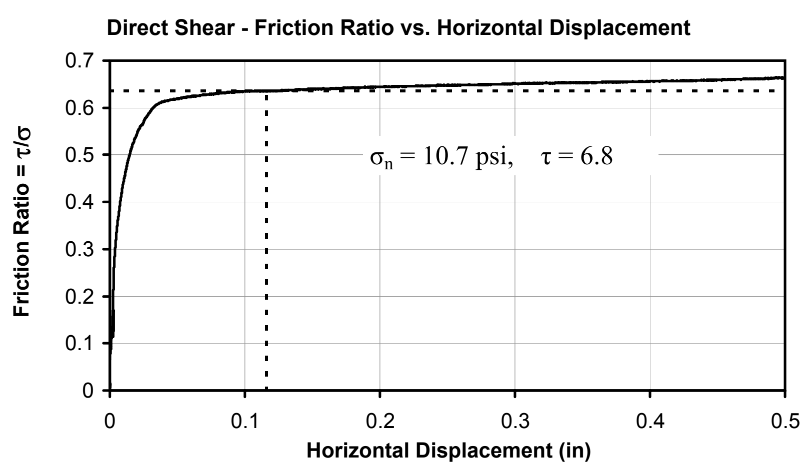
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

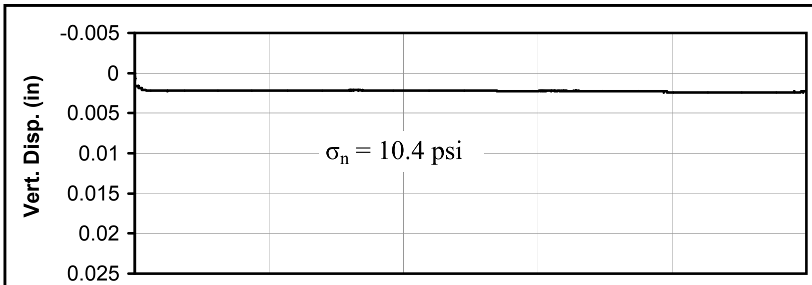
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-092605-1	4.2	1547	10.74	0.77	984	6.83	0.49	0.0024



Initial Normal Stress:	
10.43	(psi)
Friction Ratio at Failure	
0.6361	(none)
Displacement at Failure:	
0.116	(in)
Arctan(Friction Ratio):	
32.5	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 9/24/05.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/15/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/18/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

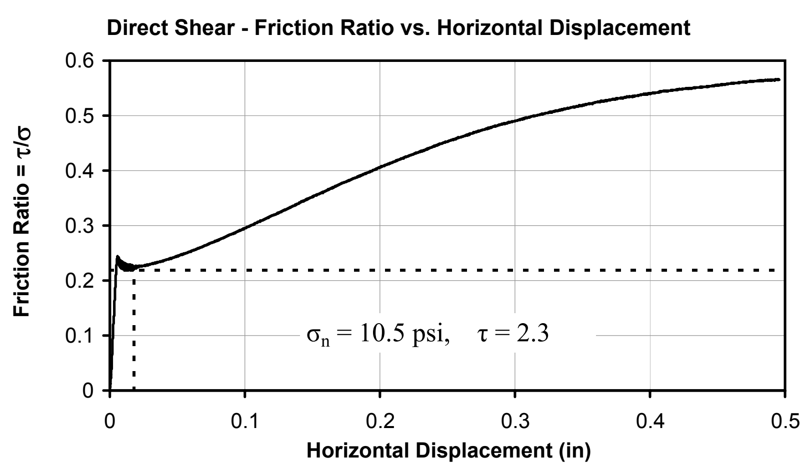
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

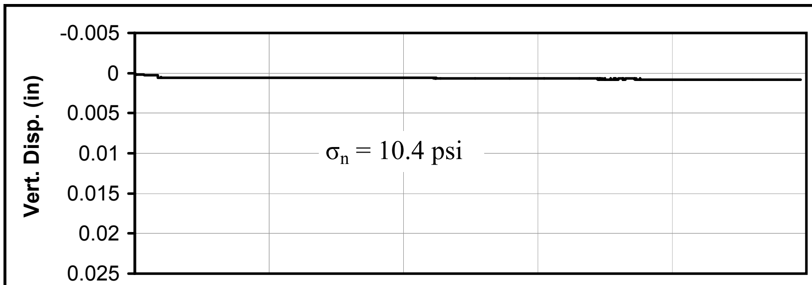
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-101505-2	4.2	1509	10.48	0.75	329	2.29	0.16	0.0008



Initial Normal Stress:	
10.43	(psi)
Friction Ratio at Failure	
0.2183	(none)
Displacement at Failure:	
0.018	(in)
Arctan(Friction Ratio):	
12.3	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 10/13/05.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	10/25/05
Sample I.D./Loc.:	Rancho Solano Clay #2	Finished:	10/28/05
Classification:	Brown Lean Clay (CL)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

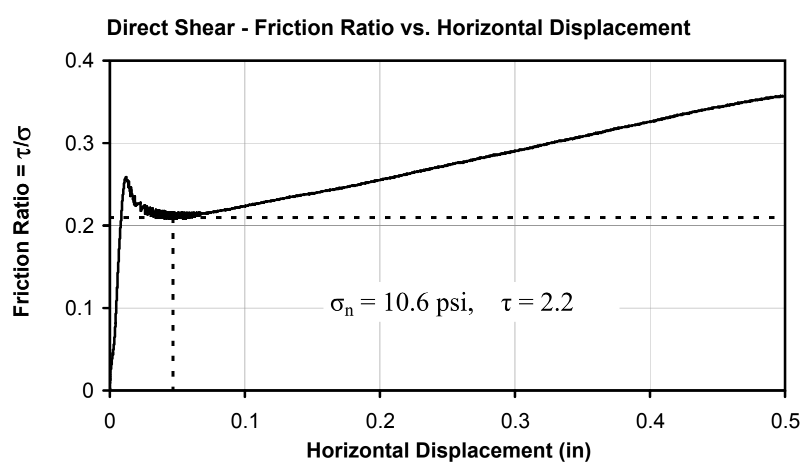
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

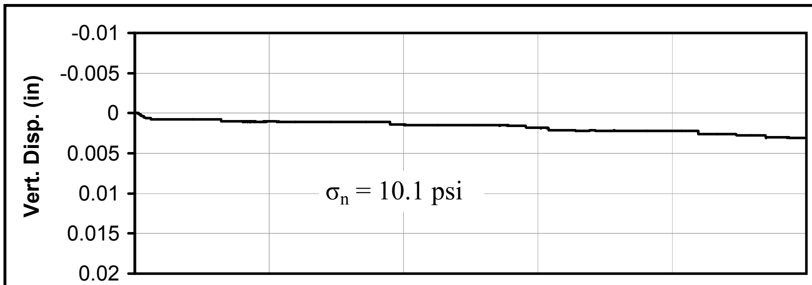
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-102505-1	4.2	1520	10.55	0.76	318	2.21	0.16	0.0031



Initial Normal Stress:	
10.43	(psi)
Friction Ratio at Failure	
0.2093	(none)
Displacement at Failure:	
0.047	(in)
Arctan(Friction Ratio):	
11.8	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 10/23/05.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	11/2/04
Sample I.D./Loc.:	San Francisco Bay Mud	Finished:	11/5/04
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

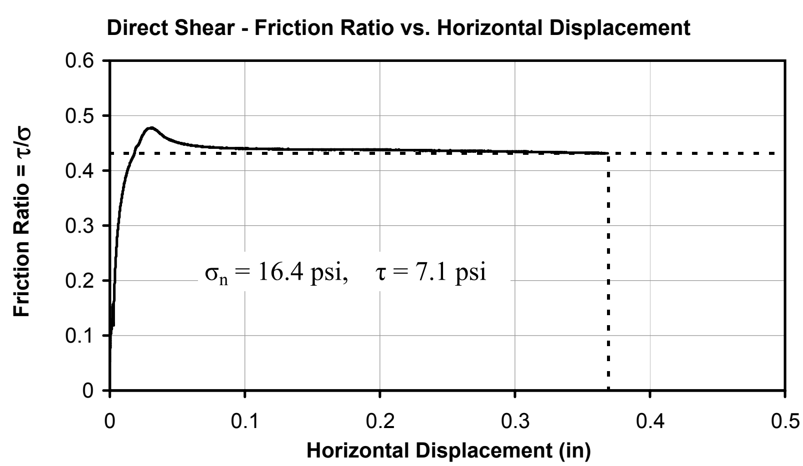
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

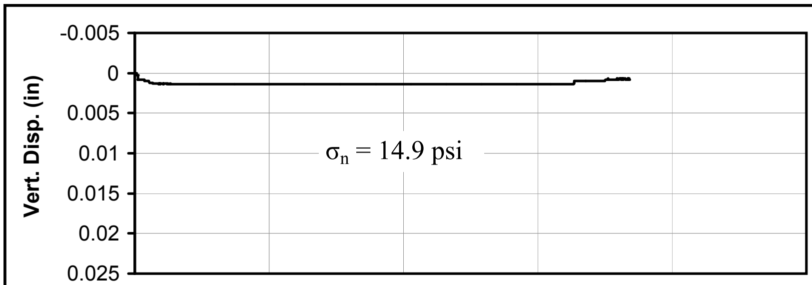
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-110204-1	7.5	2365	16.42	1.18	1020	7.08	0.51	0.0014



Initial Normal Stress:	
14.91	(psi)
Friction Ratio at Failure	
0.4313	(none)
Displacement at Failure:	
0.369	(in)
Arctan(Friction Ratio):	
23.3	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 10/29/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	11/7/05
Sample I.D./Loc.:	San Francisco Bay Mud	Finished:	11/10/05
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

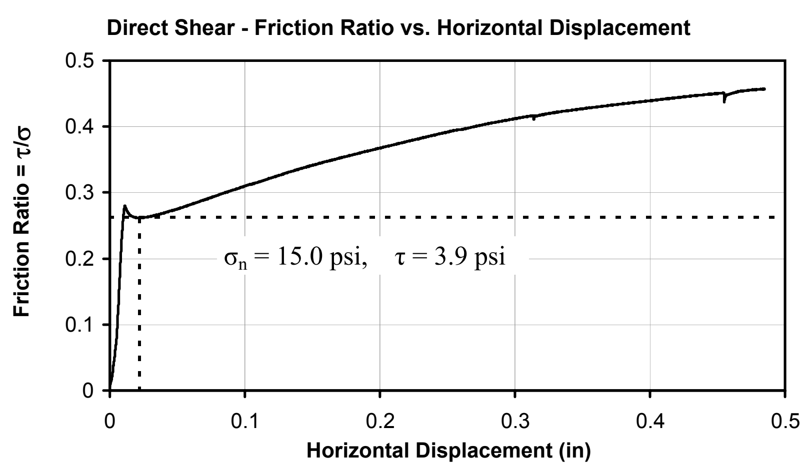
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

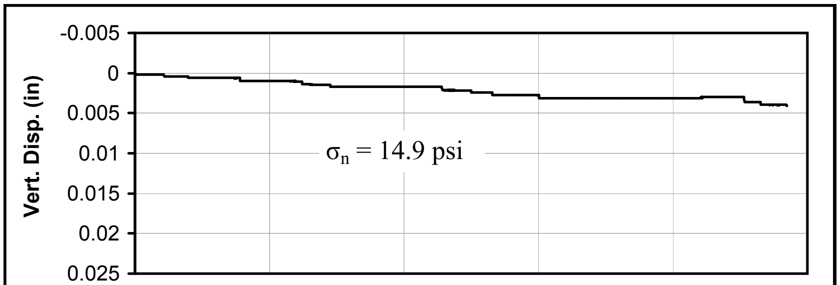
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-110705-1	7.5	2159	14.99	1.08	566	3.93	0.28	0.0041



Initial Normal Stress:	
14.91	(psi)
Friction Ratio at Failure	
0.2622	(none)
Displacement at Failure:	
0.022	(in)
Arctan(Friction Ratio):	
14.7	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 11/4/05.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces			Started:	11/11/05
Sample I.D./Loc.:	San Francisco Bay Mud			Finished:	11/14/05
Classification:	Grey Elastic Silt (MH)	Shear Device Used:	WF Direct Shear #1		

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

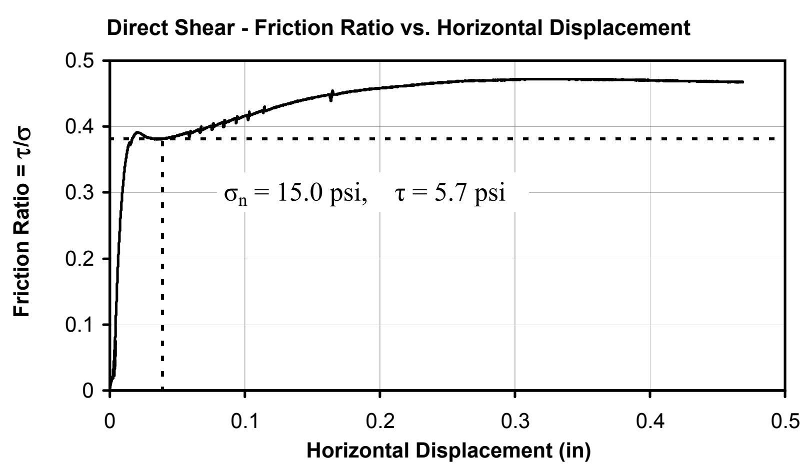
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Shear rate based on data from earlier tests.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

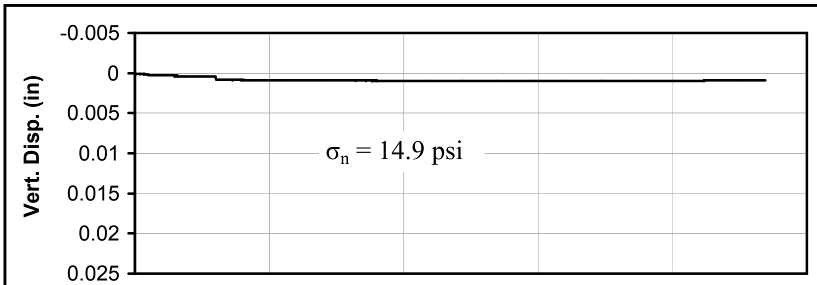
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.000123	in/min.
Test performed at shear rate =	0.003124	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-111105-1	7.5	2168	15.06	1.08	827	5.74	0.41	0.0009



Initial Normal Stress:	
14.91	(psi)
Friction Ratio at Failure	
0.3815	(none)
Displacement at Failure:	
0.039	(in)
Arctan(Friction Ratio):	
20.9	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 11/4/05.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/13/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/13/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

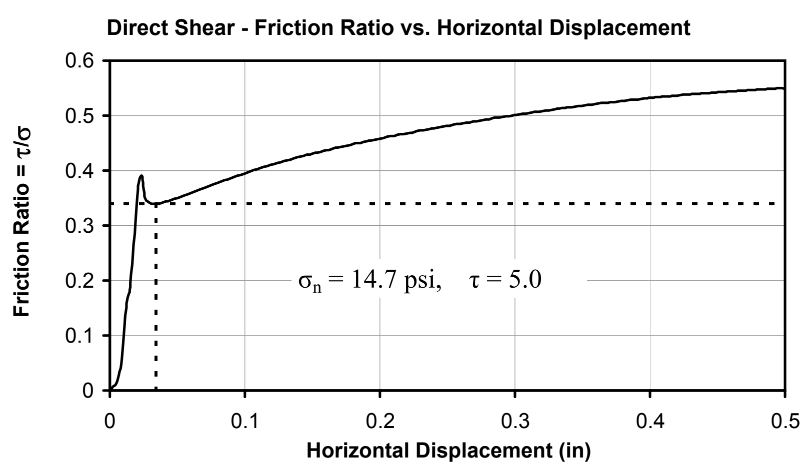
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Fast shear test.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

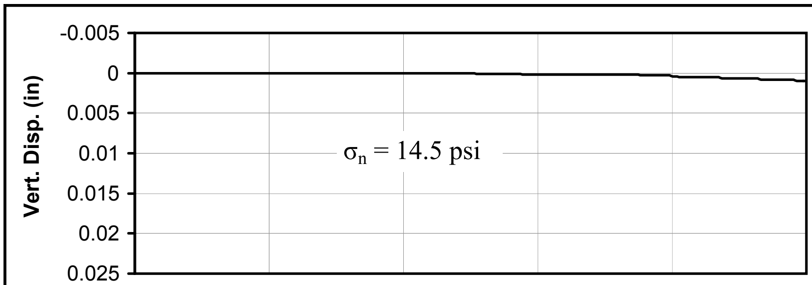
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.048	in/min.
Test performed at shear rate =	1.219	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-081304-1	7.5	2111	14.66	1.06	717	4.98	0.36	0.0010



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.3396	(none)
Displacement at Failure:	
0.034	(in)
Arctan(Friction Ratio):	
18.8	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/11/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/16/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/16/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

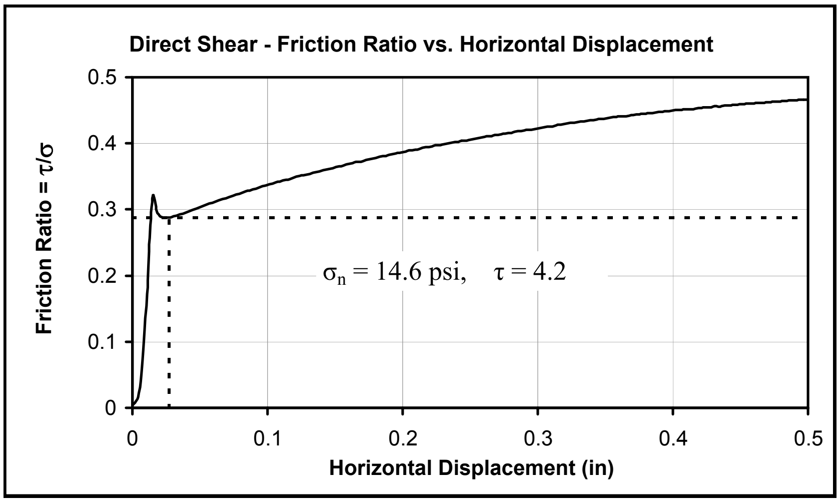
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Fast shear test.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

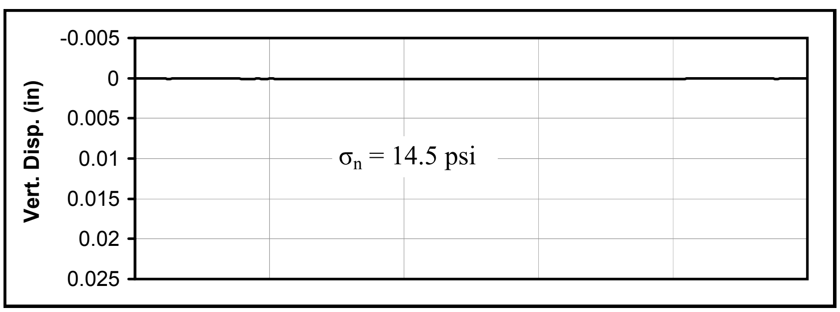
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.048	in/min.
Test performed at shear rate =	1.219	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-081604-1	7.5	2108	14.64	1.05	606	4.21	0.30	0.0001



Initial Normal Stress:	
14.54	(psi)
Friction Ratio at Failure	
0.2876	(none)
Displacement at Failure:	
0.027	(in)
Arctan(Friction Ratio):	
16.0	(deg)



Notes:

1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/13/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/23/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/23/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

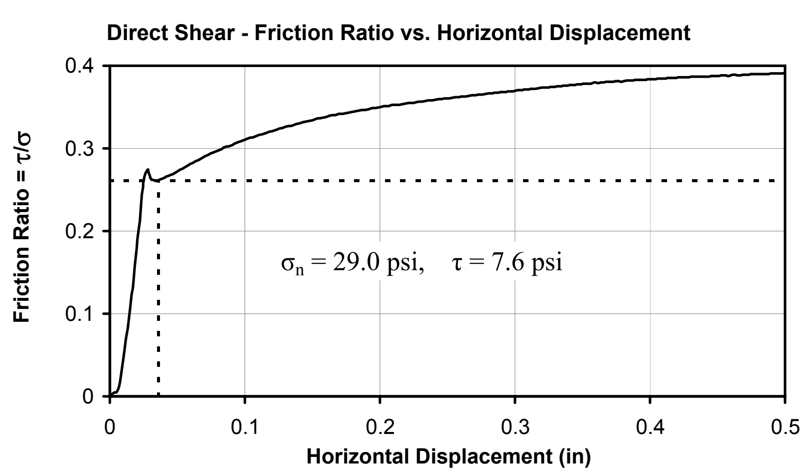
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Fast shear test.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

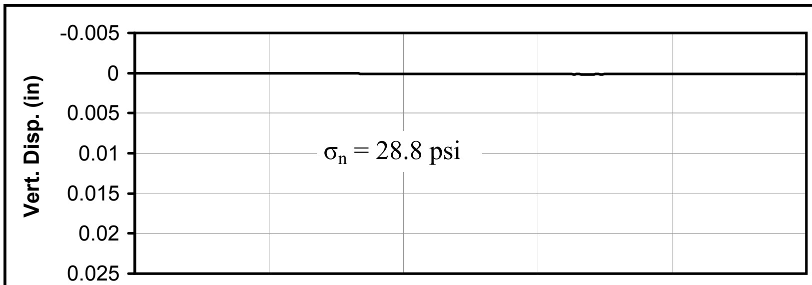
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.048	in/min.
Test performed at shear rate =	1.219	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-082304-1	18.0	4177	29.01	2.09	1092	7.58	0.55	0.0001



Initial Normal Stress:	
28.75	(psi)
Friction Ratio at Failure	
0.2614	(none)
Displacement at Failure:	
0.036	(in)
Arctan(Friction Ratio):	
14.6	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/19/04.

Virginia Polytechnic Institute and State University
Geotechnical Engineering Laboratory
Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces	Started:	8/26/04
Sample I.D./Loc.:	Rancho Solano Clay #1	Finished:	8/26/04
Classification:	Brown Fat Clay (CH)	Shear Device Used:	WF Direct Shear #1

Initial Thickness of Specimen	0.5	(in.)
Square Box Width	4	(in.)

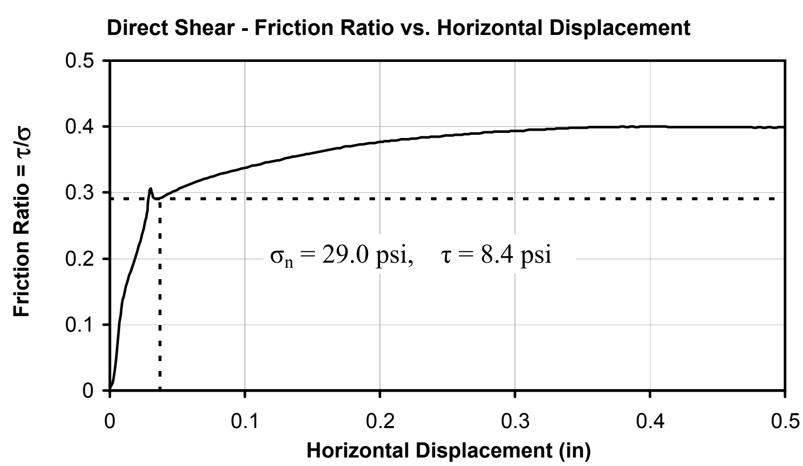
Preformed slickensided plane?	Yes
Failure Surface Location	Middle of Specimen

Consolidation Steps				Casagrande		Taylor	
Air Load	Normal Stress			t_{50}	Max. Shear Rate	t_{50}	Max. Shear Rate
(psi)	(psf)	(psi)	(tsf)	(min.)	(in/min.)	(min.)	(in/min.)
9.5	2602	18.07	1.30	Fast shear test.			
15.5	4505	31.29	2.25				
28.0	8470	58.82	4.23				
46.3	14258	99.01	7.13				

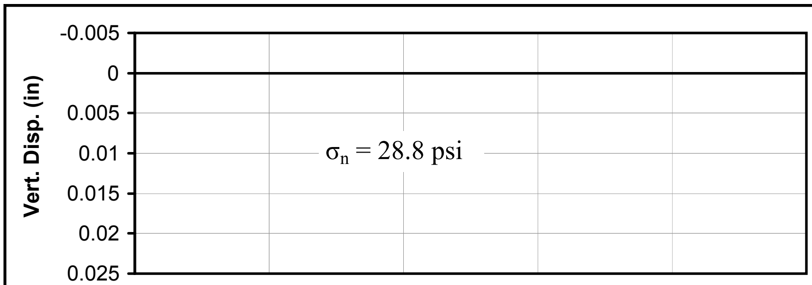
Minimum calc. shear rate =		in/min.
Estim. Failure displacement =	0.5	in.

Test performed at shear rate =	0.048	in/min.
Test performed at shear rate =	1.219	mm/min.

Test Number	Hanger Load	Residual Normal Stress			Residual Shear Stress			ΔH
	(kg)	(psf)	(psi)	(tsf)	(psf)	(psi)	(tsf)	(in.)
D1-082604-1	18.0	4178	29.02	2.09	1215	8.44	0.61	0.0000



Initial Normal Stress:	
28.75	(psi)
Friction Ratio at Failure	
0.2908	(none)
Displacement at Failure:	
0.037	(in)
Arctan(Friction Ratio):	
16.2	(deg)



Notes:
1. Specimen remolded at LL.
2. Specimen pushed through the #40 sieve.
3. Specimen was preconsol. to approximately 50 psi in batch consol., pressed into direct shear box, and consolidated using the above load steps.
4. External load frame used to consolidate specimens.
5. Started consolidating on 8/23/04.

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	4/8/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-040805-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

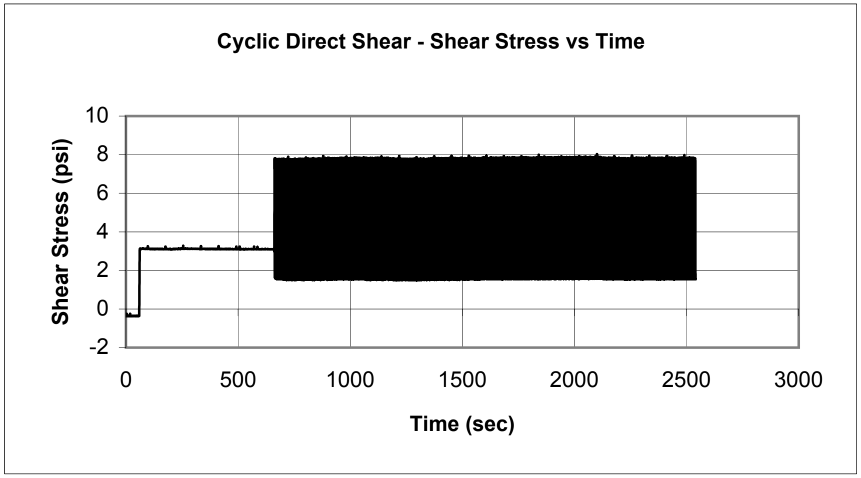
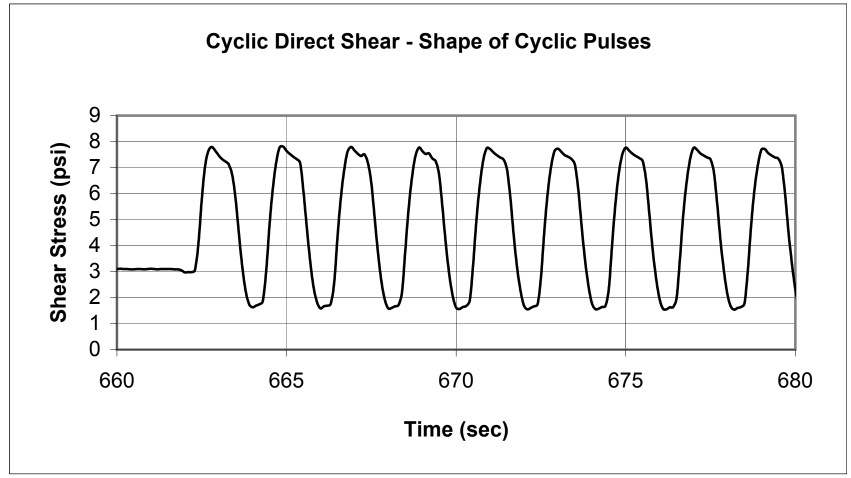
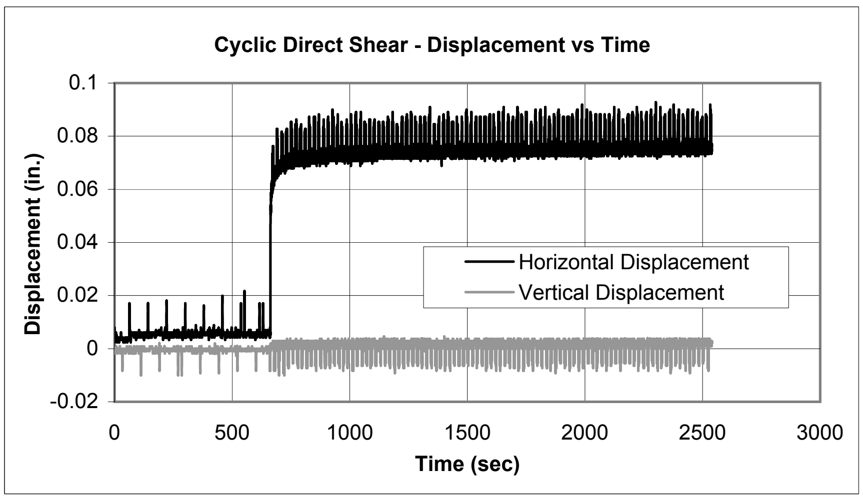
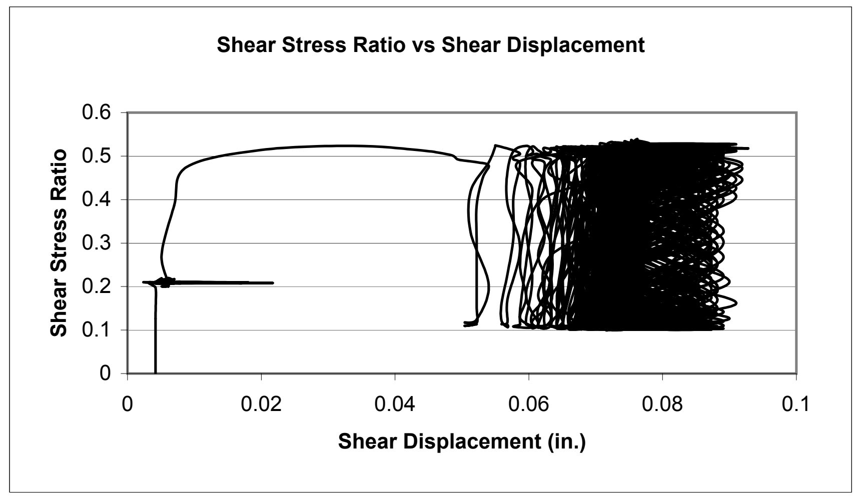
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

Time allowed for dissipation of pore pressures after static load application	10	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.52
48	7.68	1106	0.55	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.21
19.5	3.12	449	0.22	

No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)
10
8
6
4
2
0
-2
0
500
1000
1500
2000
2500
3000

Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)
9
8
7
6
5
4
3
2
1
0
660
665
670
675
680

Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement
0.1
0.08
0.06
0.04
0.02
0
-0.02
0
500
1000
1500
2000
2500
3000

Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.02
0.04
0.06
0.08
0.1

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Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	4/29/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-042905-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

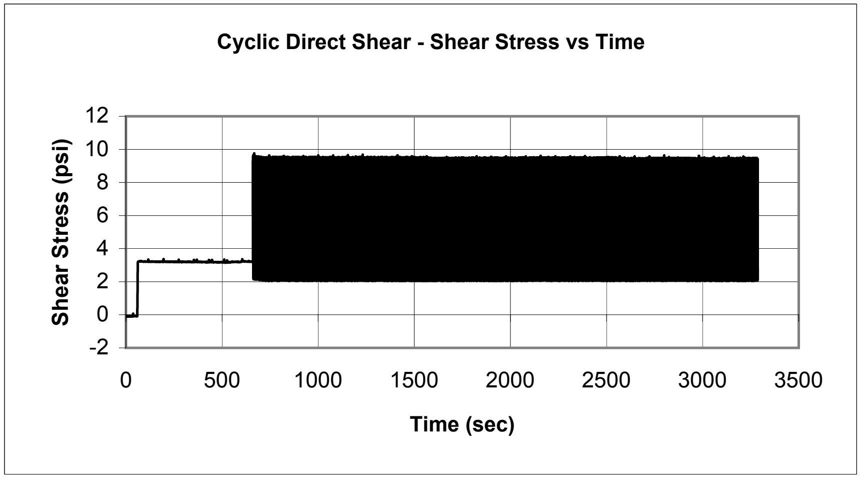
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

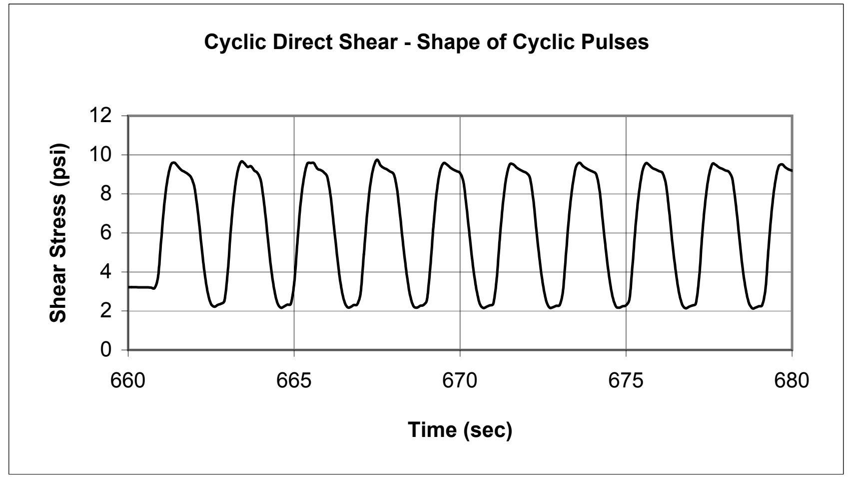
Time allowed for dissipation of pore pressures after static load application	10	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

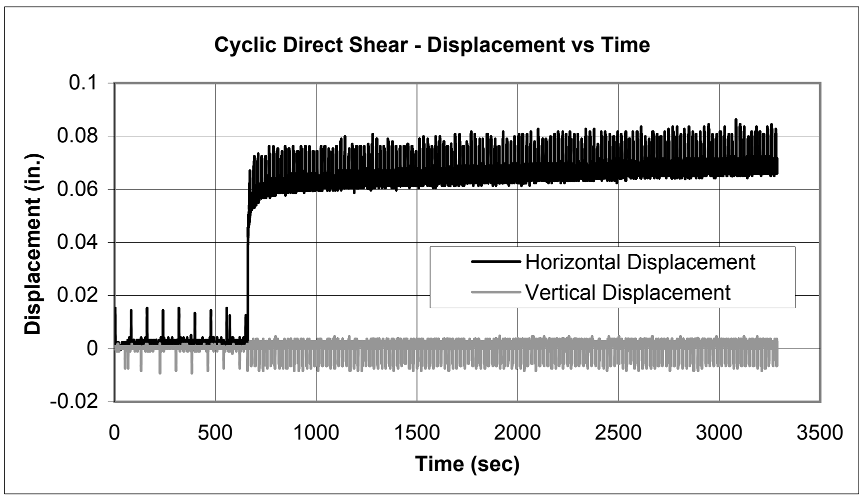
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.64
60	9.60	1382	0.69	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.22
20.1	3.22	463	0.23	

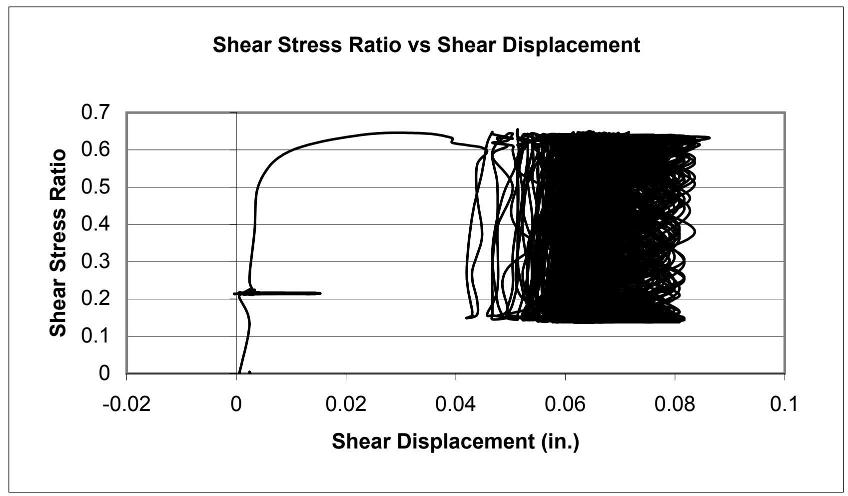
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)
12
10
8
6
4
2
0
-2
0
500
1000
1500
2000
2500
3000
3500


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)
12
10
8
6
4
2
0
660
665
670
675
680


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement
0.1
0.08
0.06
0.04
0.02
0
-0.02
0
500
1000
1500
2000
2500
3000
3500


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.02
0
0.02
0.04
0.06
0.08
0.1

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Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces
Location:	W.C. English Laboratory — Test Date: 6/21/2005
Sample ID:	Rancho Solano Clay #1 — Tested By: Derek Martowska
Classification:	Brown Fat Clay (CH)
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-062105-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

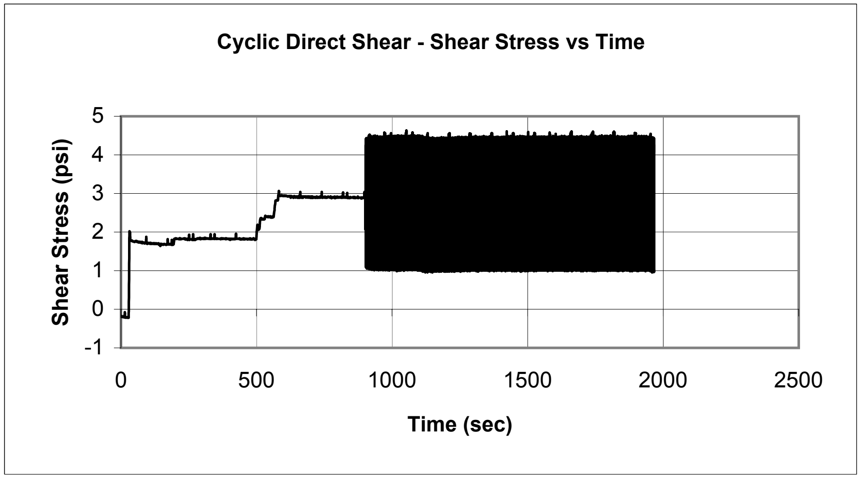
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

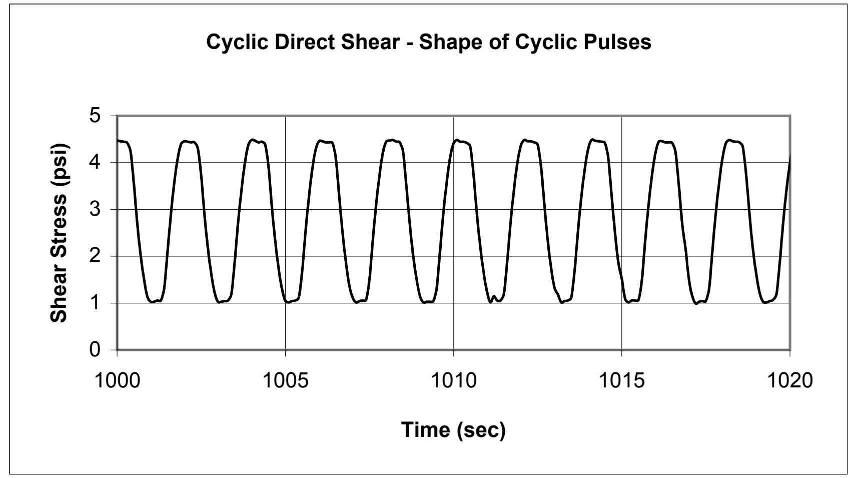
Time allowed for dissipation of pore pressures after static load application	5	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

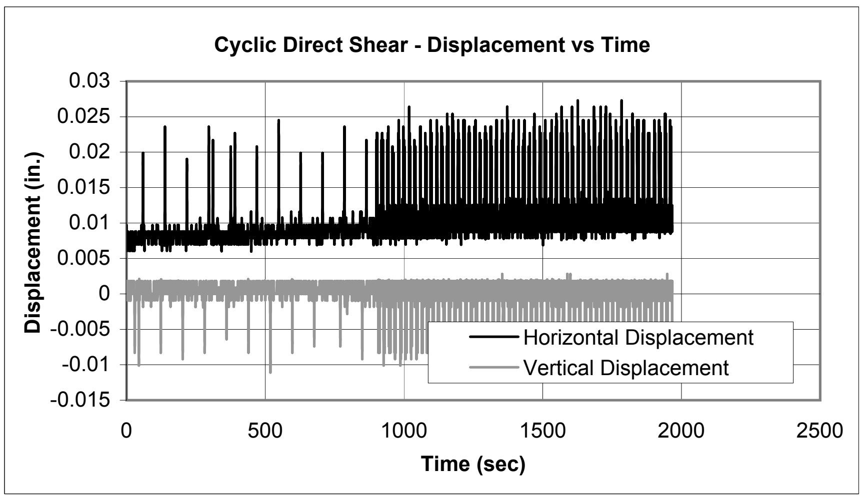
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.30
27.6	4.42	636	0.32	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.19
18.1	2.90	417	0.21	

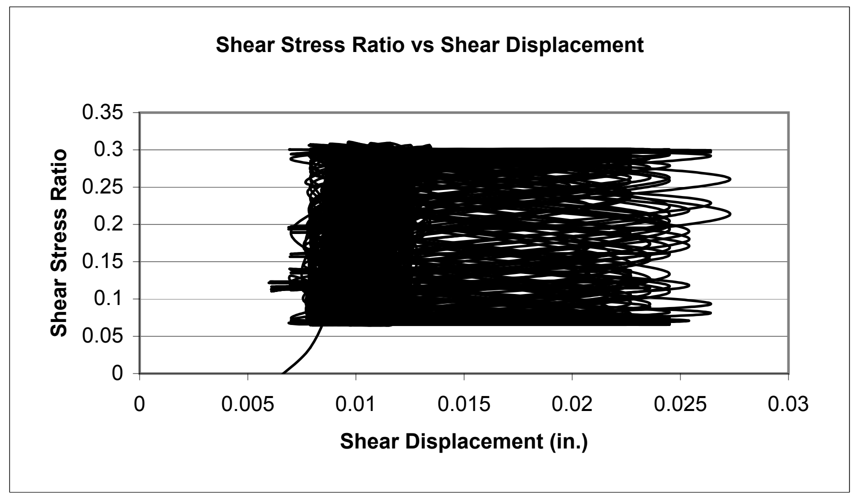
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)
5
4
3
2
1
0
-1
0
500
1000
1500
2000
2500


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)
5
4
3
2
1
0
1000
1005
1010
1015
1020


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement
0.03
0.025
0.02
0.015
0.01
0.005
0
-0.005
-0.01
-0.015
0
500
1000
1500
2000
2500


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
0
0.005
0.01
0.015
0.02
0.025
0.03

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Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	6/27/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-062705-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

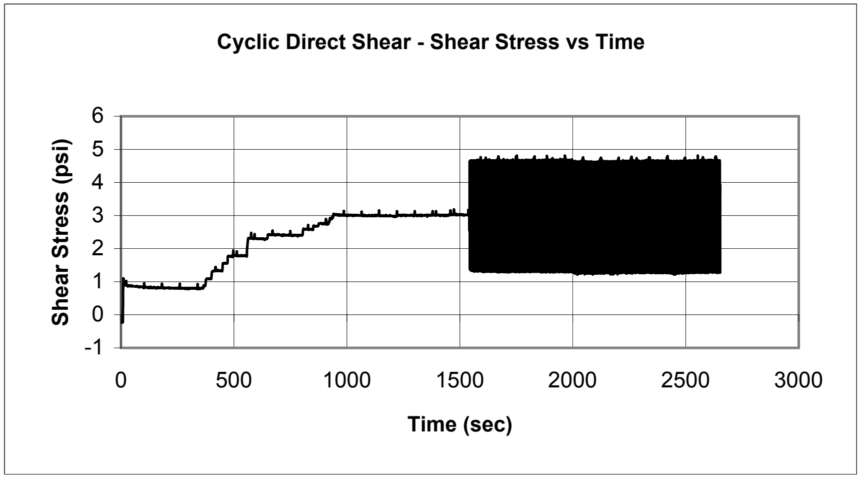
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

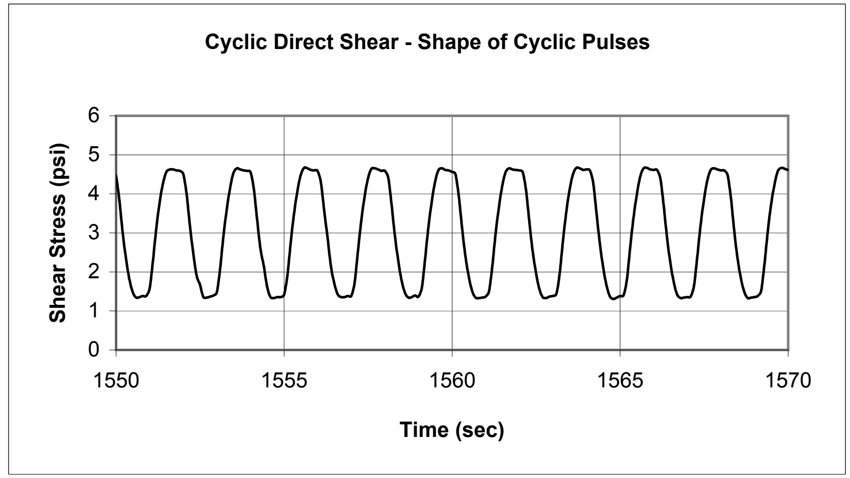
Time allowed for dissipation of pore pressures after static load application	10	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

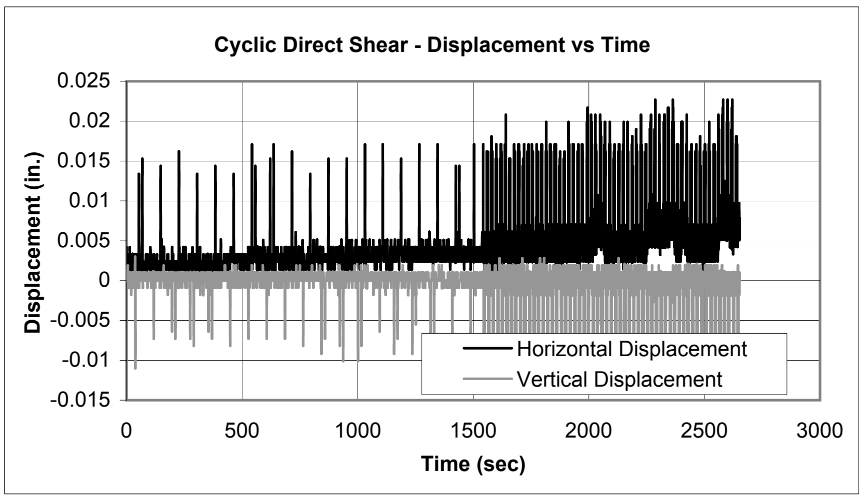
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.31
29	4.64	668	0.33	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.20
18.9	3.02	435	0.22	

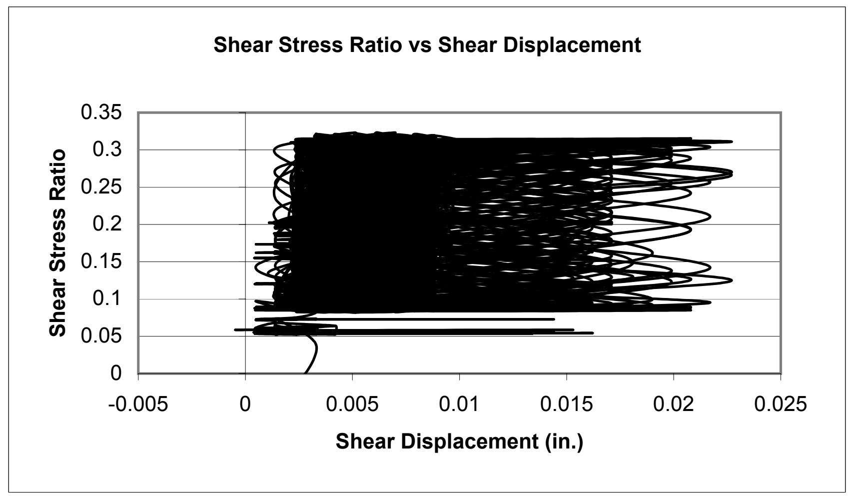
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)
6
5
4
3
2
1
0
-1
0
500
1000
1500
2000
2500
3000


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)
6
5
4
3
2
1
0
1550
1555
1560
1565
1570


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
0.025
0.02
0.015
0.01
0.005
0
-0.005
-0.01
-0.015
0
500
1000
1500
2000
2500
3000
Horizontal Displacement
Vertical Displacement


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.005
0
0.005
0.01
0.015
0.02
0.025

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Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	6/28/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-062805-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

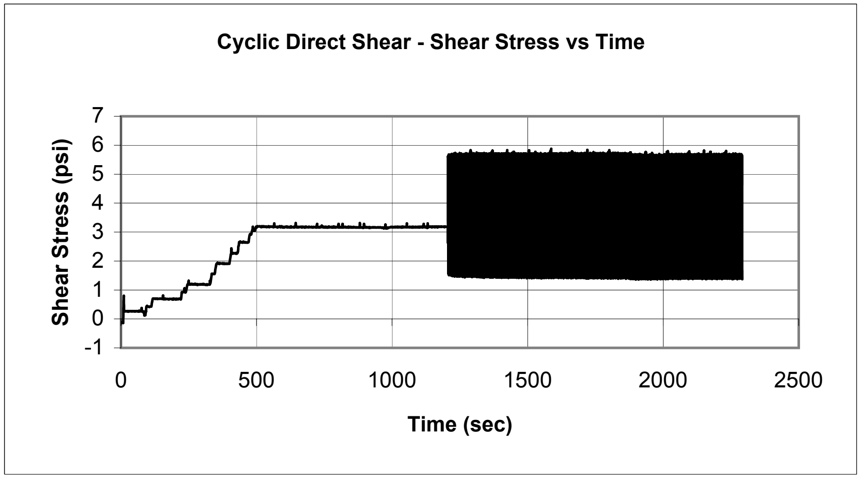
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

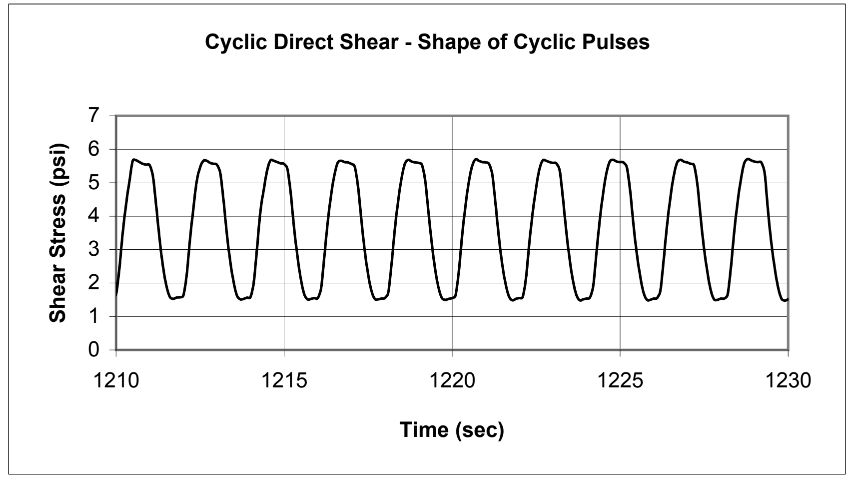
Time allowed for dissipation of pore pressures after static load application	12	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

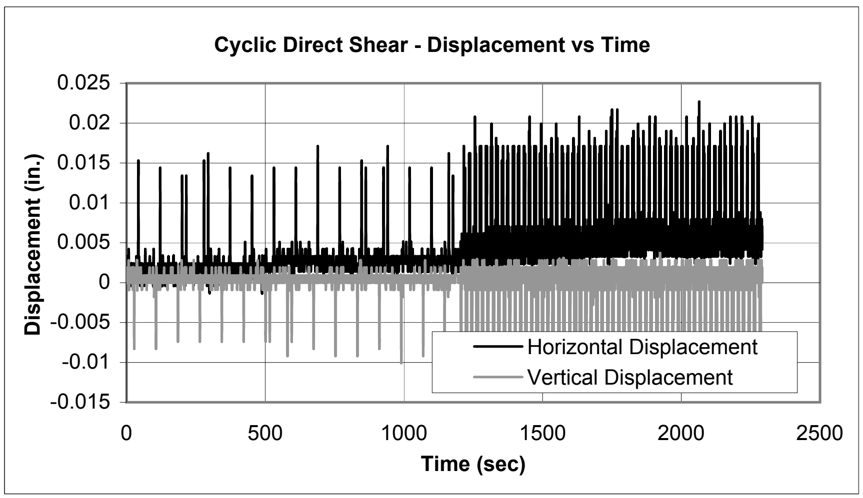
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.38
35.2	5.63	811	0.41	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.21
19.9	3.18	458	0.23	

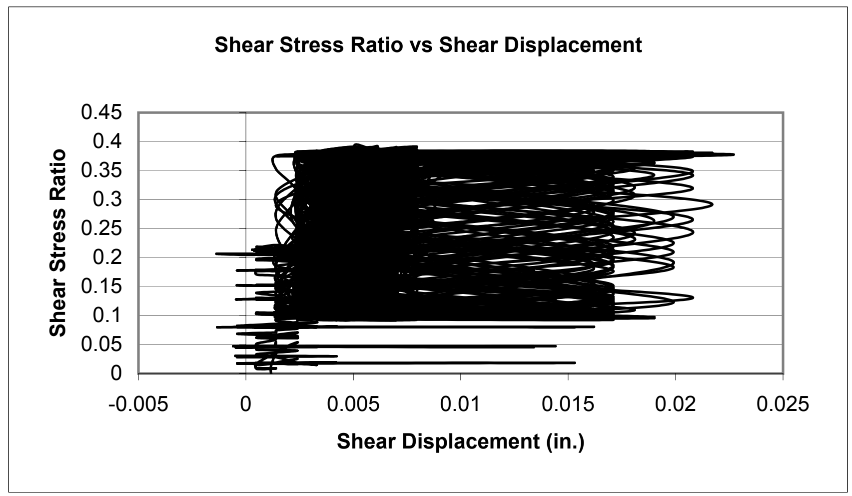
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)
7
6
5
4
3
2
1
0
-1
0
500
1000
1500
2000
2500


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)
7
6
5
4
3
2
1
0
1210
1215
1220
1225
1230


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement
0.025
0.02
0.015
0.01
0.005
0
-0.005
-0.01
-0.015
0
500
1000
1500
2000
2500


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)
0.45
0.4
0.35
0.3
0.25
0.2
0.15
0.1
0.05
0
-0.005
0
0.005
0.01
0.015
0.02
0.025

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	9/1/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-090105-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

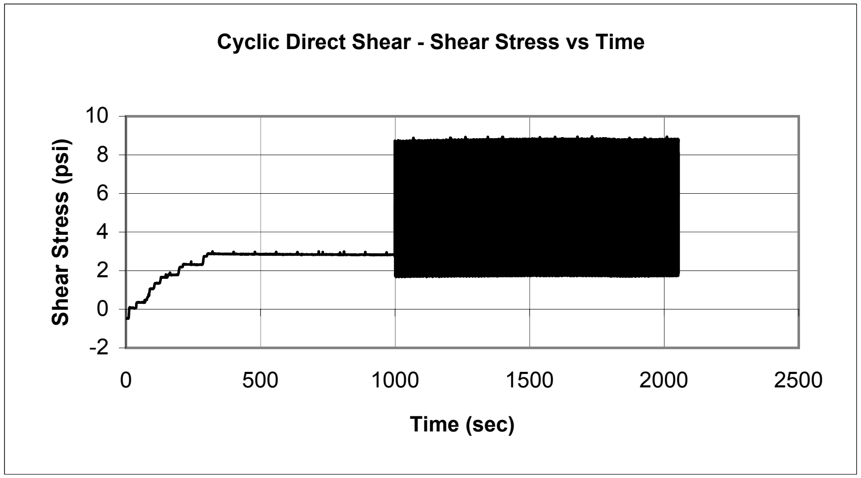
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

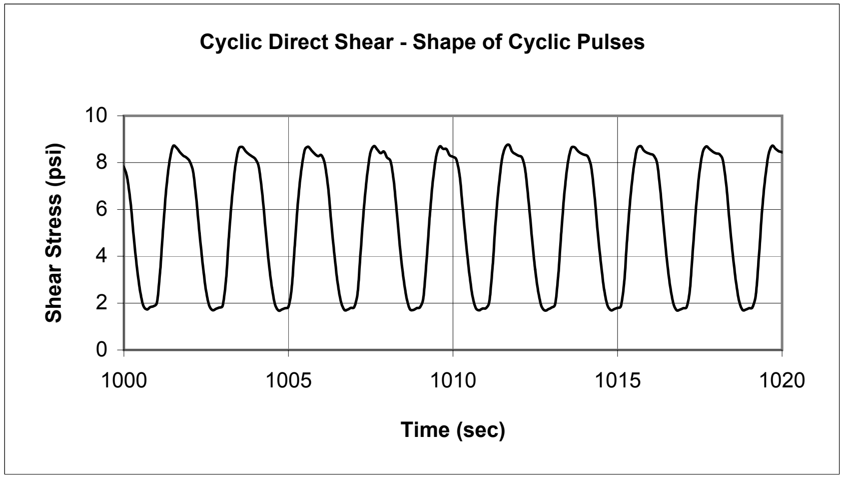
Time allowed for dissipation of pore pressures after static load application	11	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

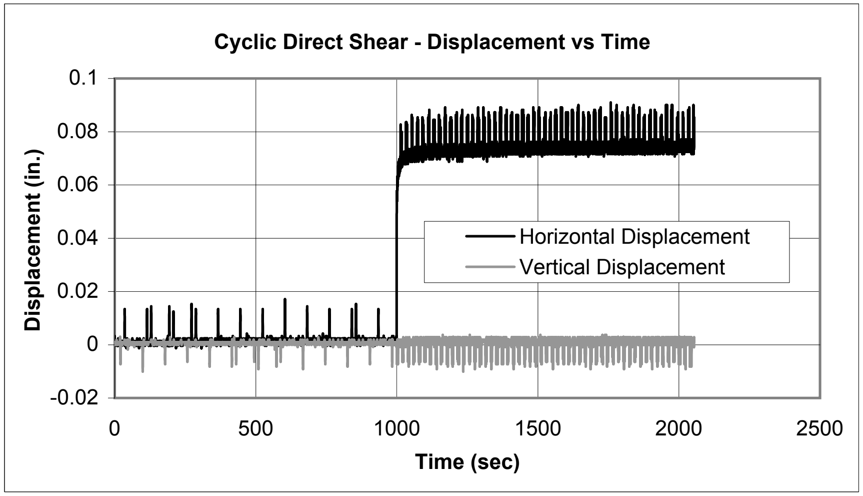
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.58
54.4	8.70	1253	0.63	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.19
17.7	2.83	408	0.20	

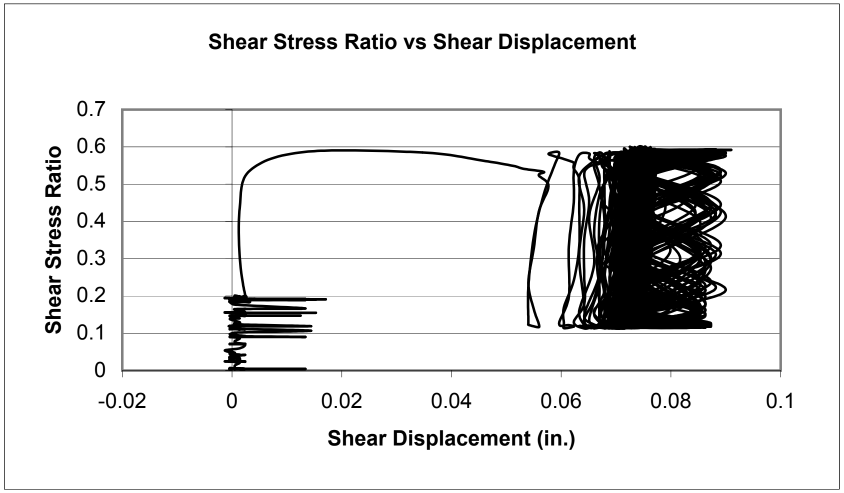
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)

Virginia Polytechnic Institute and State University

Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	9/27/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-092705-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

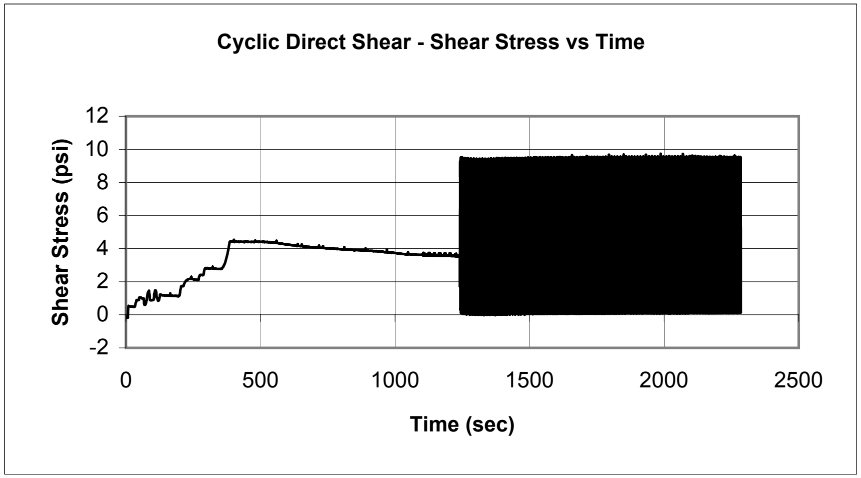
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

Time allowed for dissipation of pore pressures after static load application	14	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

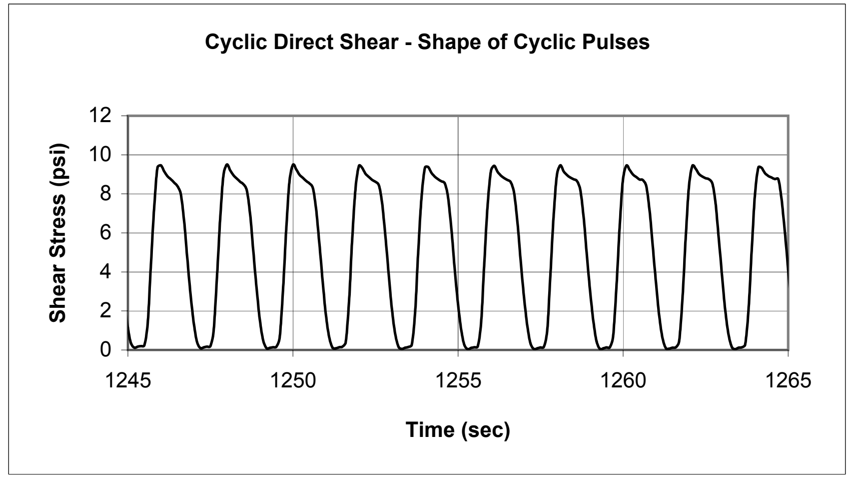
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.63
59	9.44	1359	0.68	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.24
22.1	3.54	509	0.25	

No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

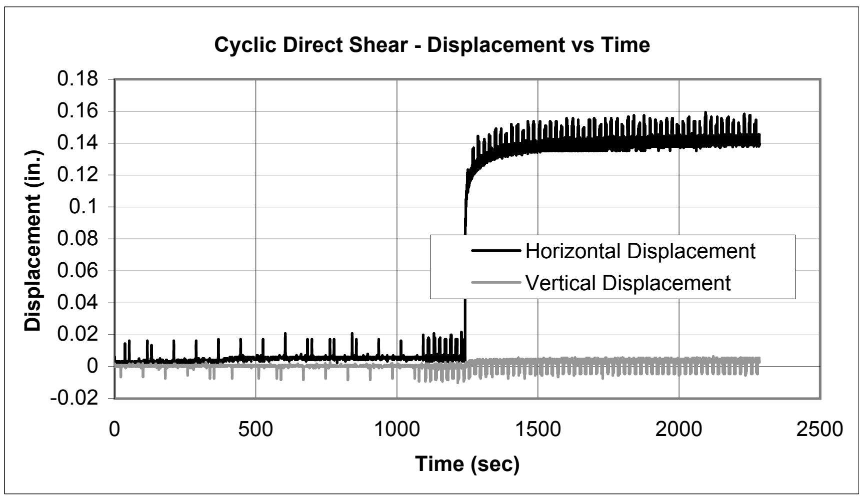
Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength



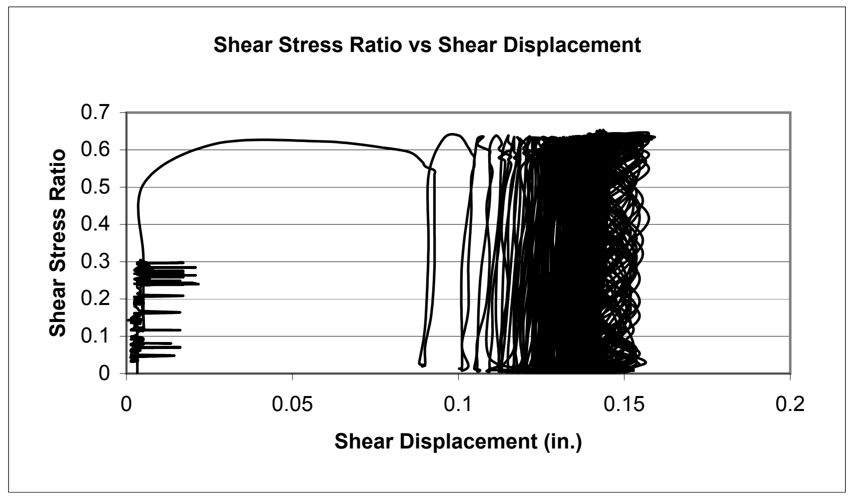
Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
12
10
8
6
4
2
0
-2
0
500
1000
1500
2000
2500
Time (sec)



Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
12
10
8
6
4
2
0
1245
1250
1255
1260
1265
Time (sec)



Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
-0.02
0
500
1000
1500
2000
2500
Time (sec)
Horizontal Displacement
Vertical Displacement



Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.05
0.1
0.15
0.2
Shear Displacement (in.)

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Geotechnical Engineering Laboratory

Cyclic Direct Shear Data Sheet

Project:	Determining the Cyclic Shear Strength of Slickensided Slip Surfaces		
Location:	W.C. English Laboratory	Test Date:	9/29/2005
Sample ID:	Rancho Solano Clay #1	Tested By:	Derek Martowska
Classification:	Brown Fat Clay (CH)		
Test Device:	Cyclic Direct Shear Device (modification of existing simple shear device)		

Sample Preconsolidation			
Applied Air Load	Normal Stress		
(psi)	(psi)	(psf)	(tsf)
6.0	17.78	2560	1.28
10.5	31.11	4480	2.24
20.0	59.26	8533	4.27
33.8	100.00	14400	7.20

Test No.		D2-092905-1	
Initial Specimen Thickness		0.5	in.
Square Box Width		2.5	in.
Presheared	No	-	in./min
Preformed slickensided plane?		Yes	
Failure Surface Location		Middle	

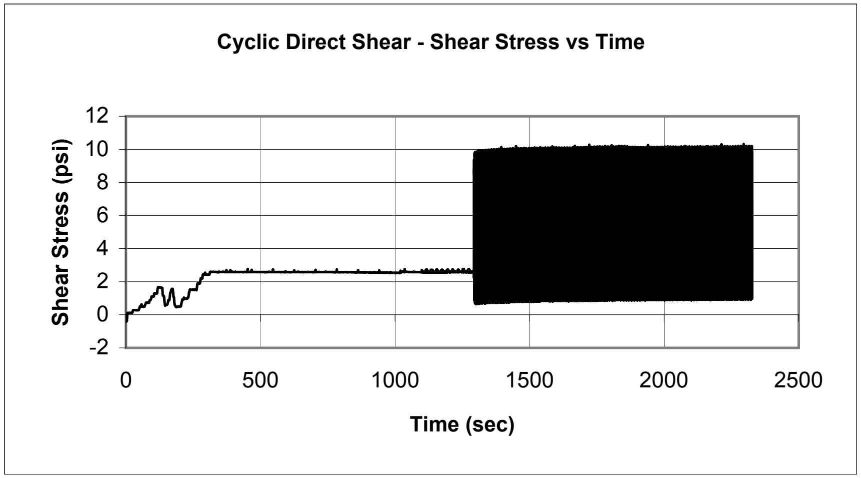
Normal Load		
Air Load	4.0	psi
Normal Stress		
(psi)	(psf)	(tsf)
14.89	2144	1.07

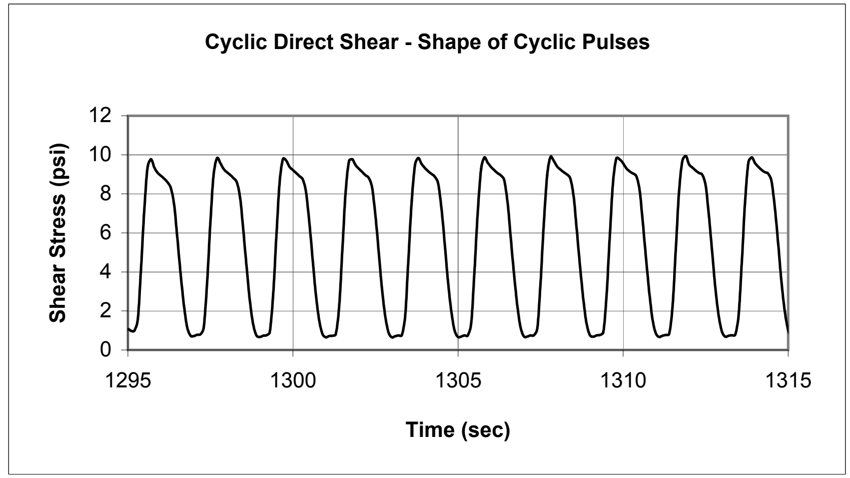
Time allowed for dissipation of pore pressures after static load application	11	min
Cyclic pulse frequency	0.5	Hz
Cyclic pulse period	2	sec

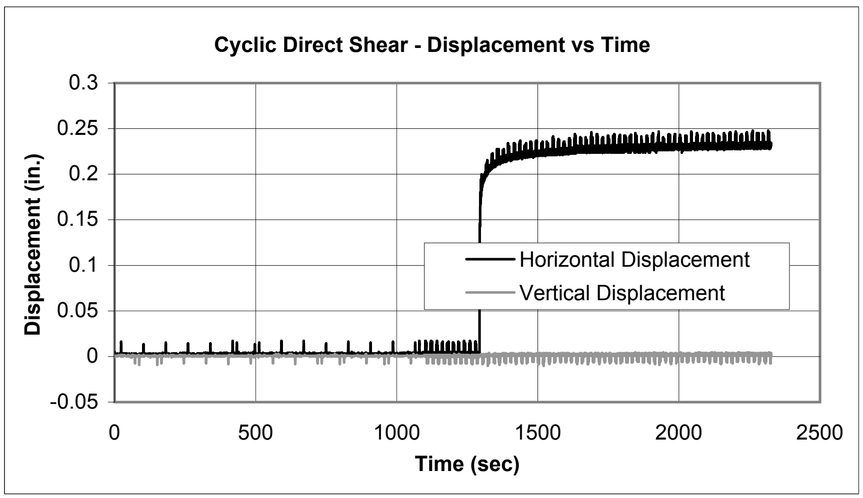
T_{peak}	τ_{peak}			τ_{peak}/σ'_{fc}
(lb)	(psi)	(psf)	(tsf)	0.66
61	9.76	1405	0.70	
T_{static}	τ_{static}			$\tau_{static}/\sigma'_{fc}$
(lb)	(psi)	(psf)	(tsf)	0.17
16.2	2.59	373	0.19	

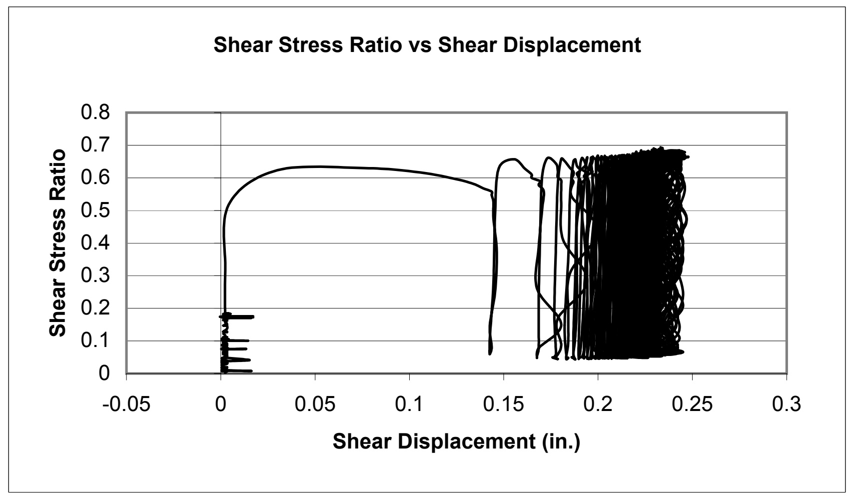
No. of cycles tested?	500
Reason for stopping test?	Test limited to 500 cycles

Notes: Specimen remolded at LL
Specimen pushed through #40 sieve
Monotonic static shear load and backpressure applied gradually to ~70% theoretical static shear strength


Cyclic Direct Shear - Shear Stress vs Time
Shear Stress (psi)
Time (sec)


Cyclic Direct Shear - Shape of Cyclic Pulses
Shear Stress (psi)
Time (sec)


Cyclic Direct Shear - Displacement vs Time
Displacement (in.)
Time (sec)
Horizontal Displacement
Vertical Displacement


Shear Stress Ratio vs Shear Displacement
Shear Stress Ratio
Shear Displacement (in.)

APPENDIX C

CENTRIFUGE MODEL

SHOP DRAWINGS

This appendix contains detailed shop drawings that were used to construct the centrifuge model for test CLM02. The purpose of this appendix is to communicate important model construction details that may be of interest to other centrifuge researchers. Further details about the model construction process can be found in Meehan et al. (2005a) and Meehan et al. (2005b), which are available on the UC Davis NEES website (http://nees.ucdavis.edu/). The following figures are contained in this appendix:

- Figure C-1. This figure is a plan view of the design concept that was used in test CLM02. It shows the concrete bases in the rigid container, the sliding block models, the static loading system, and the points of water application and drainage from the model.

- Figure C-2. This figure is a side view of the design concept that was used in test CLM02. It shows the same design elements as Figure C-1.

- Figure C-3. This figure shows a detailed side view of one of the sliding block specimens.

- Figure C-4. This figure shows a detailed plan view of the upper steel plate. The weir system and kaolinite injection locations are shown.

- Figure C-5. This figure shows a detailed plan view of the lower steel plate.

- Figure C-6. This figure shows a plan view of the consolidation mold that was used to create the stiff clay specimens for the sliding block model.

- Figure C-7. This figure shows a side view of the consolidation mold that was used to create the stiff clay specimens for the sliding block model. The upper figure shows an outer view of the consolidation mold. The lower figure shows how the detailed consolidation layer system fit within the consolidation mold.

- Figure C-8. This figure shows a detailed side view of the layered system that was used to consolidate the sliding block models.

- Figure C-9. This figure shows instrument sizes and dimensions.

- Figure C-10. This figure shows the locations of the instruments that were used on each of the sliding block models.

- Figure C-11. This figure shows overall instrument locations and instrument numbers.

Figure C-1. Overall Concept for Centrifuge Test CLM02 – Plan View.

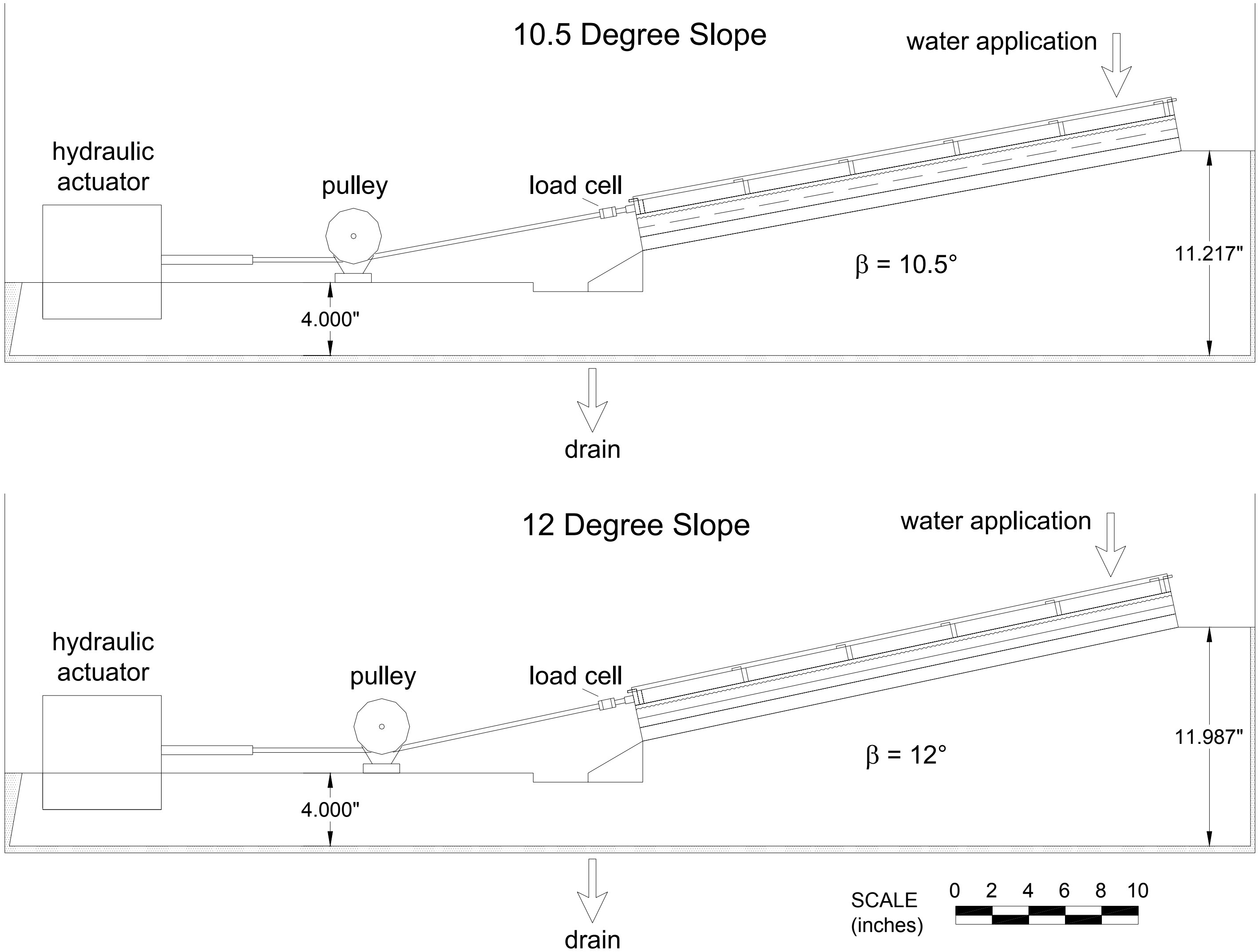


Figure C-2. Overall Concept for Centrifuge Test CLM02 – Side View.

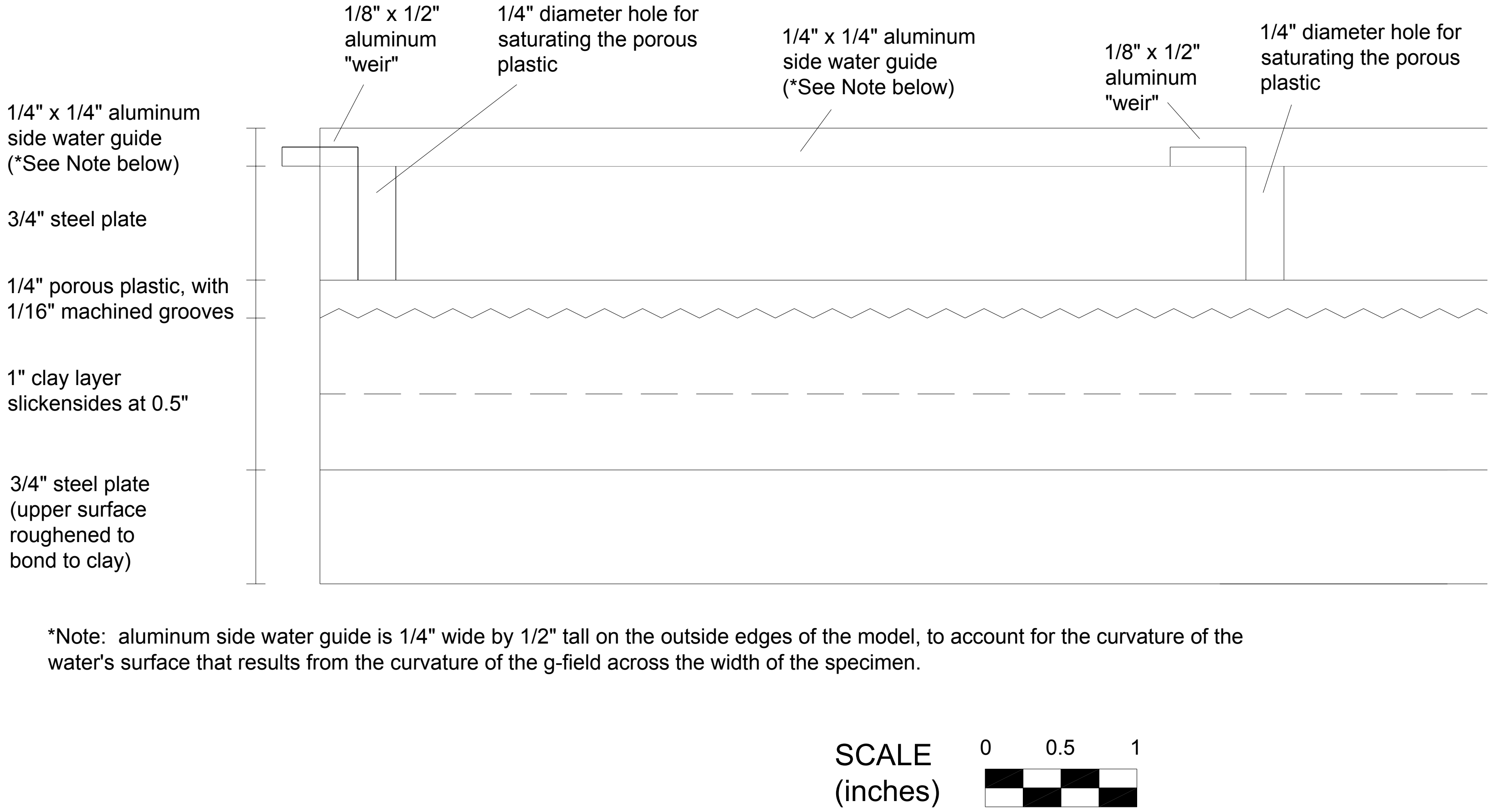


Figure C-3. Detailed Side View of Specimen During Test.

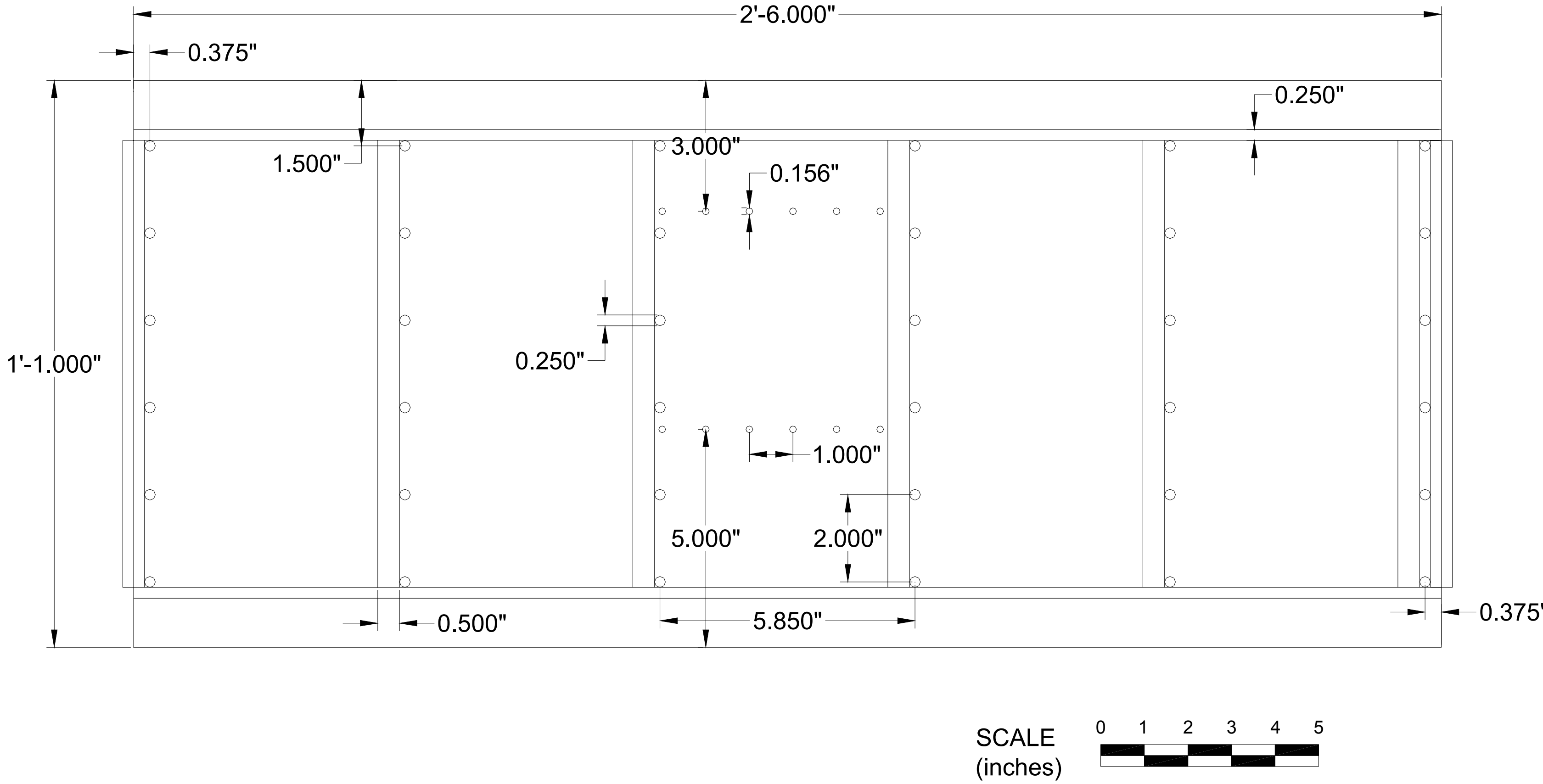


Figure C-4. Detailed View of Upper Steel Plate.

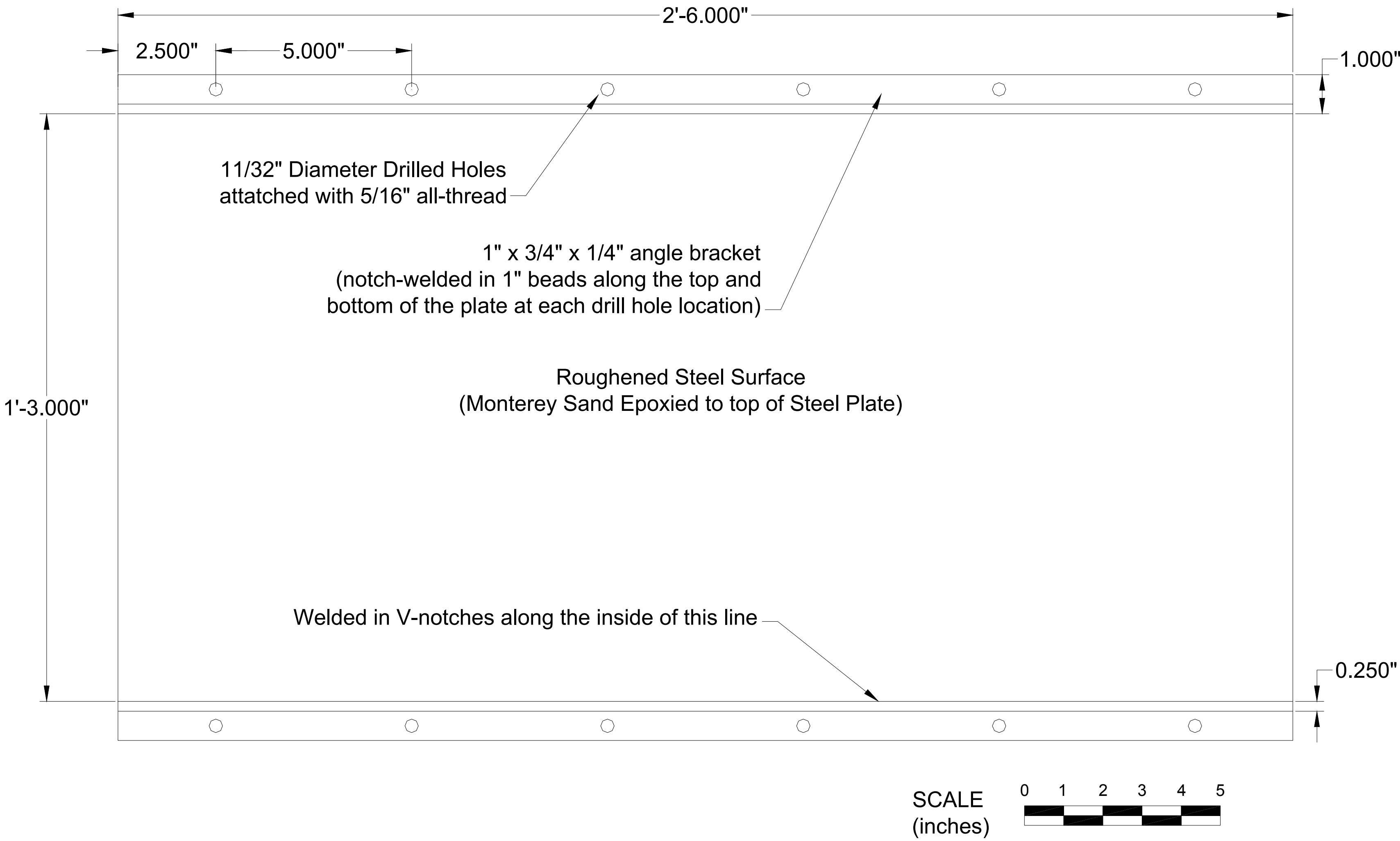


Figure C-5. Detailed View of Lower Steel Plate.

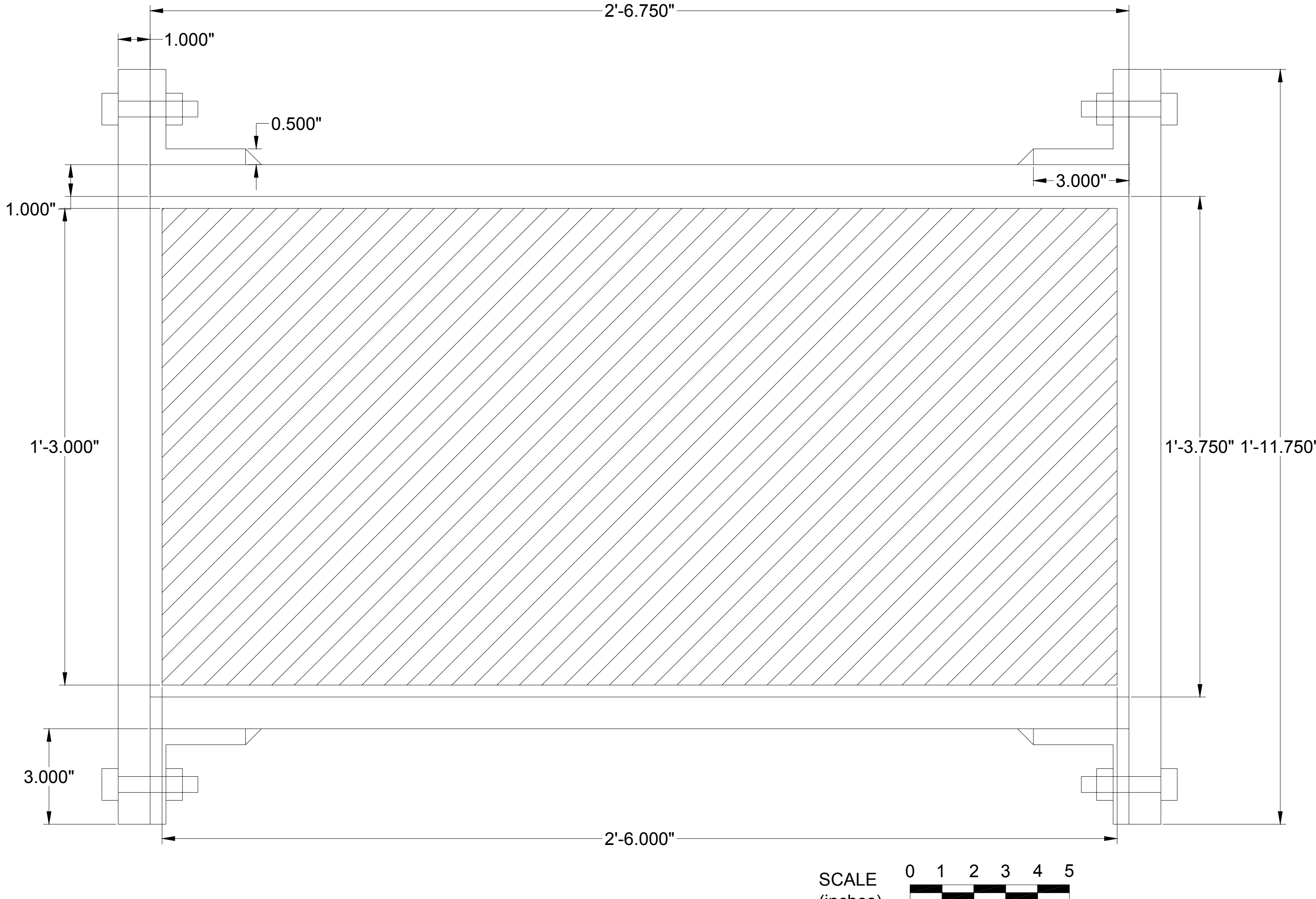


Figure C-6. Consolidation Mold, Plan View.

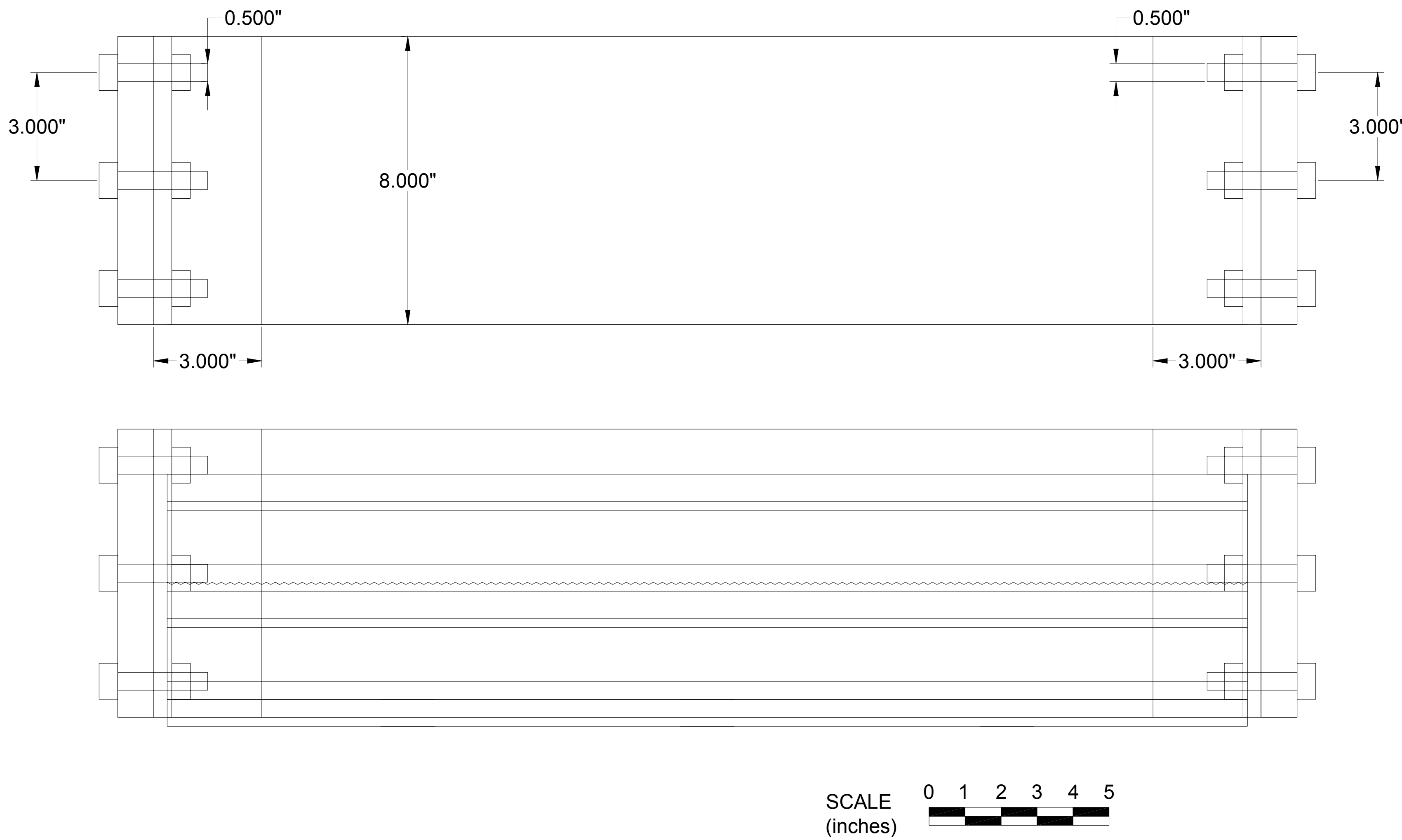


Figure C-7. Consolidation Mold, Side View.

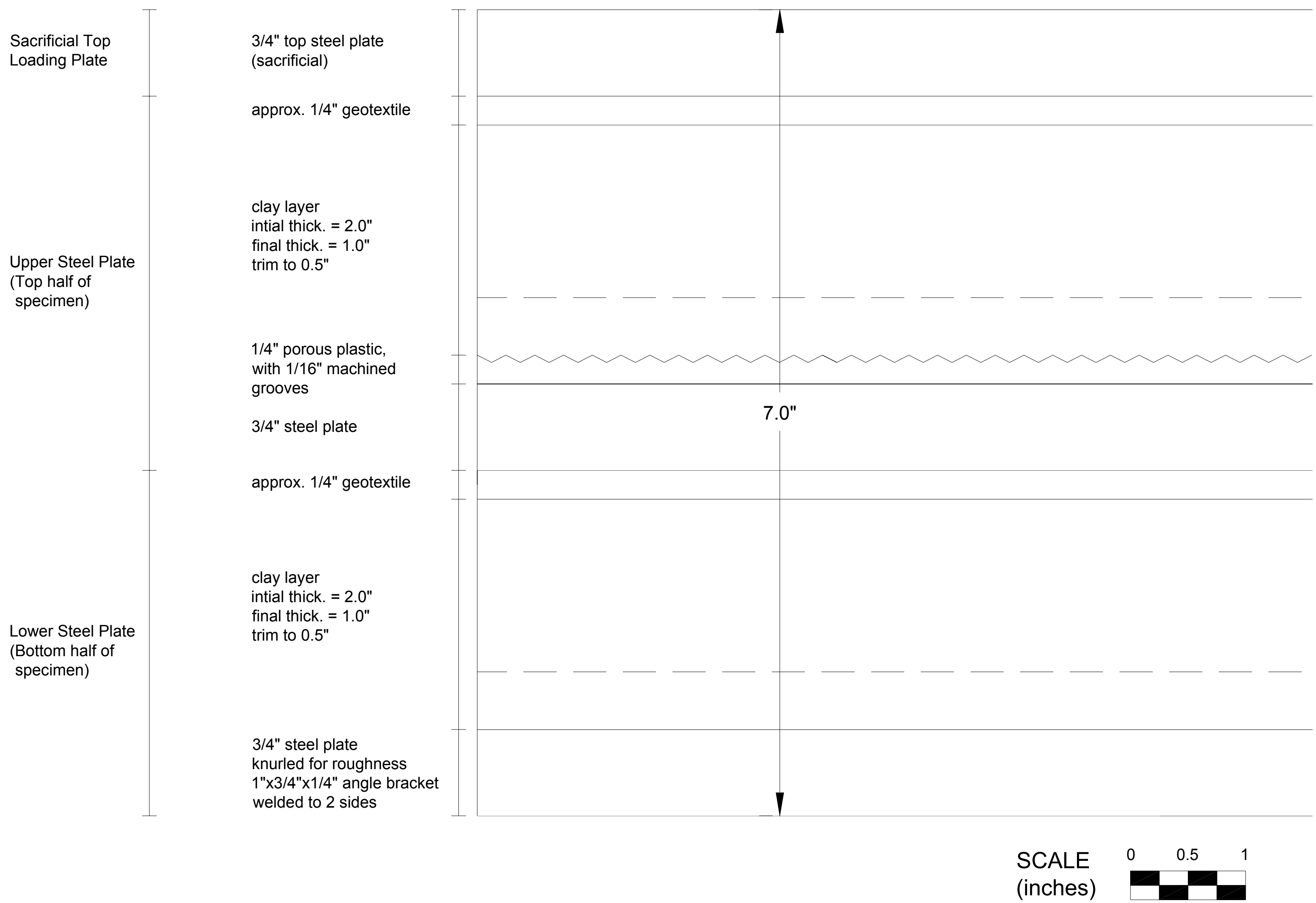
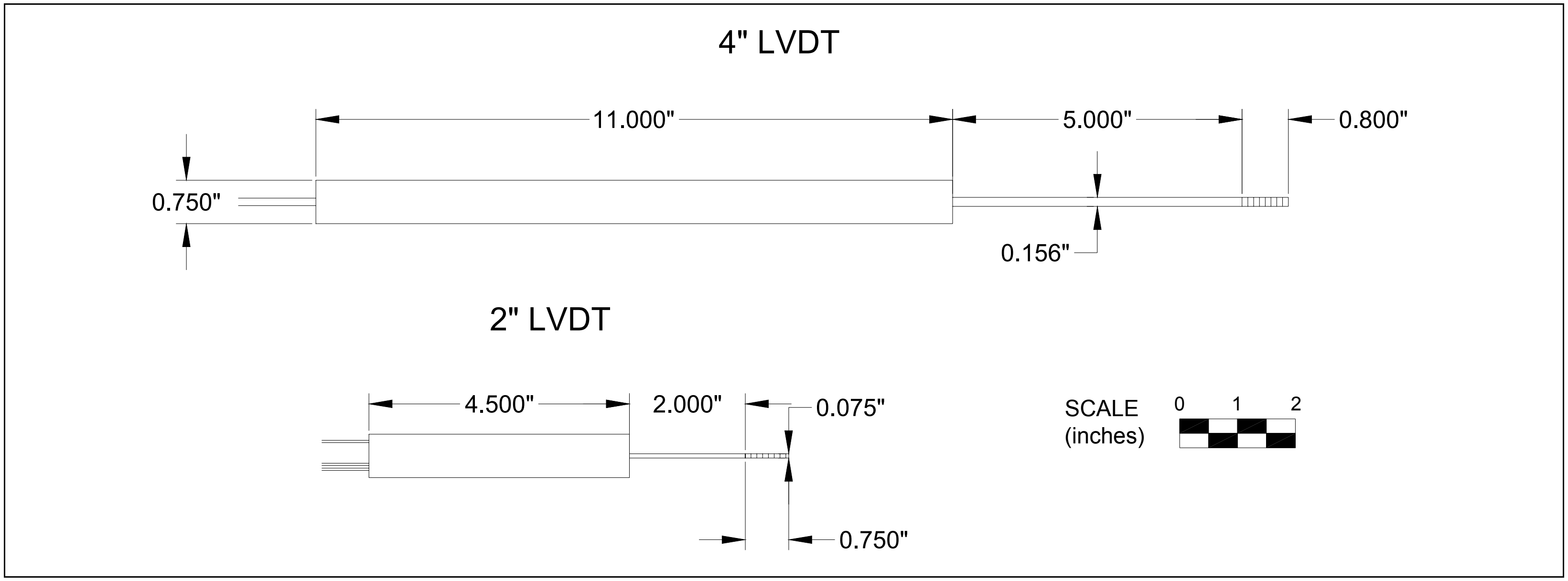


Figure C-8. Consolidation Layer System, Detail View.



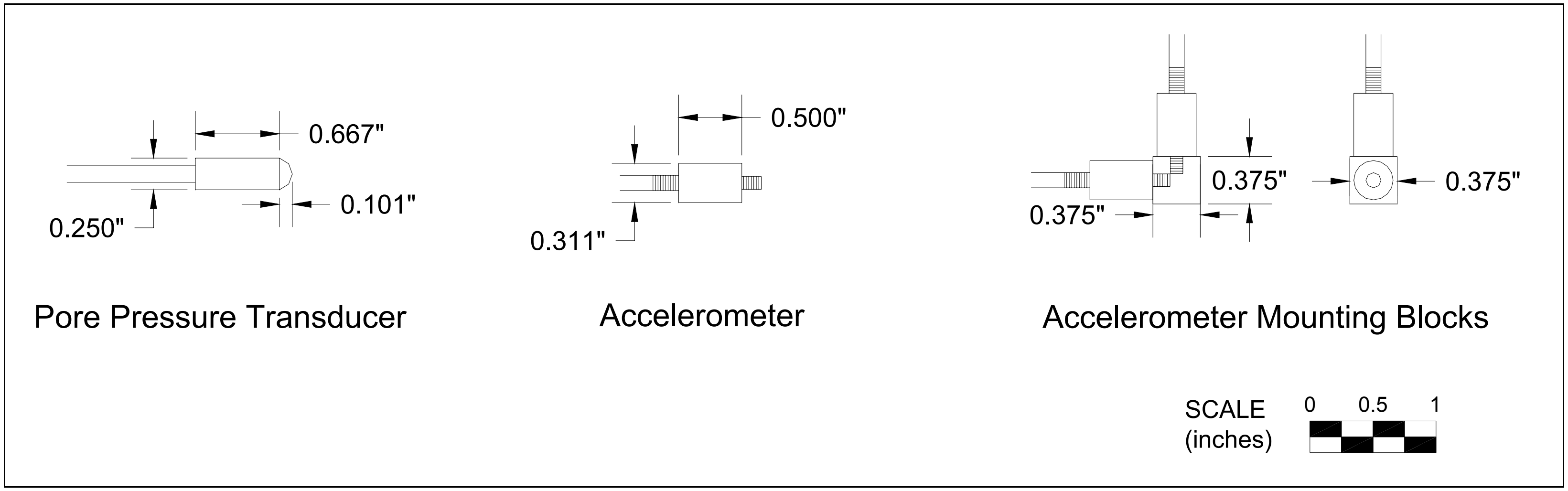


Figure C-9. Instrument Sizes and Dimensions.

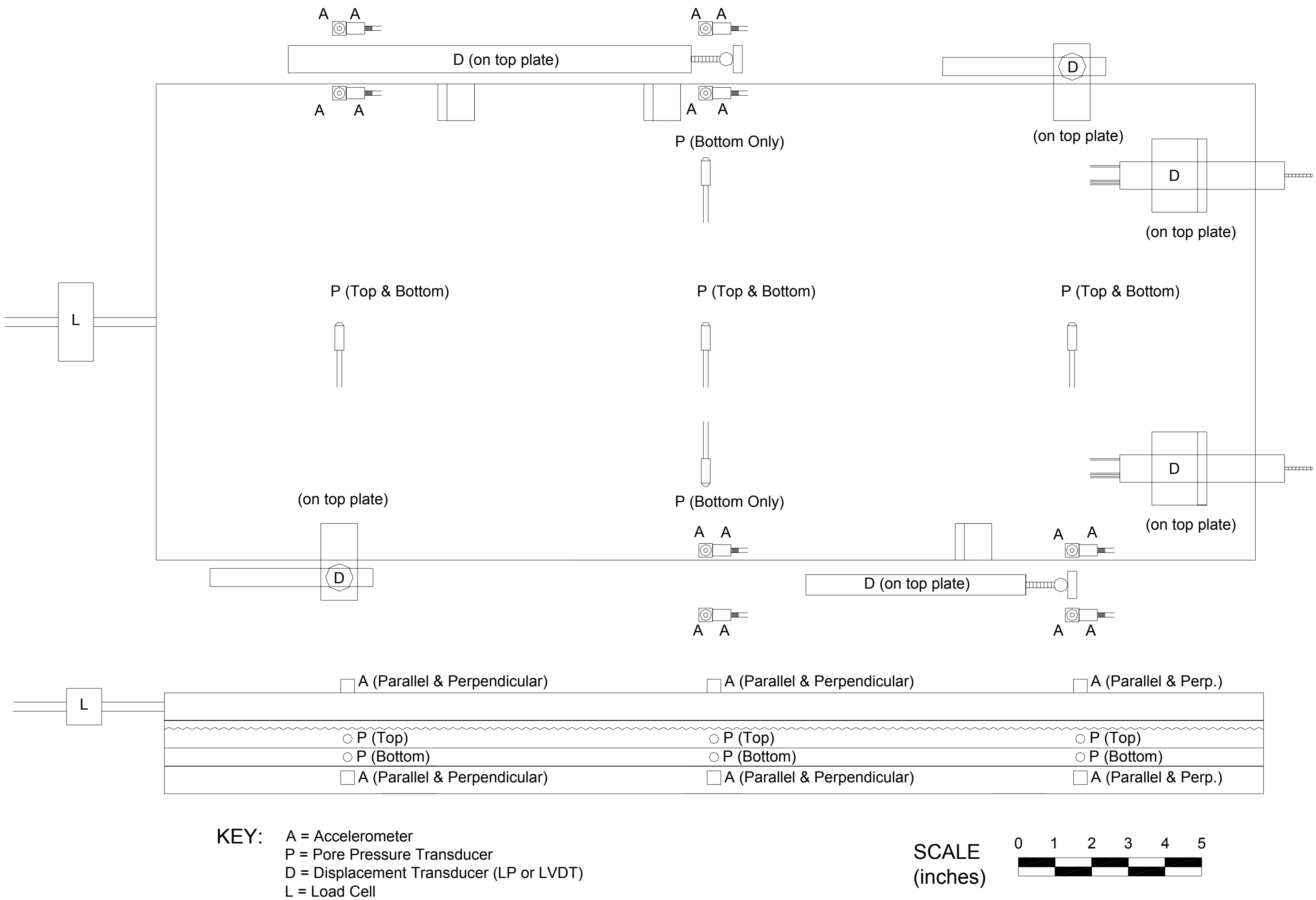


Figure C-10. Location of Instruments on Each Sliding Block Model.

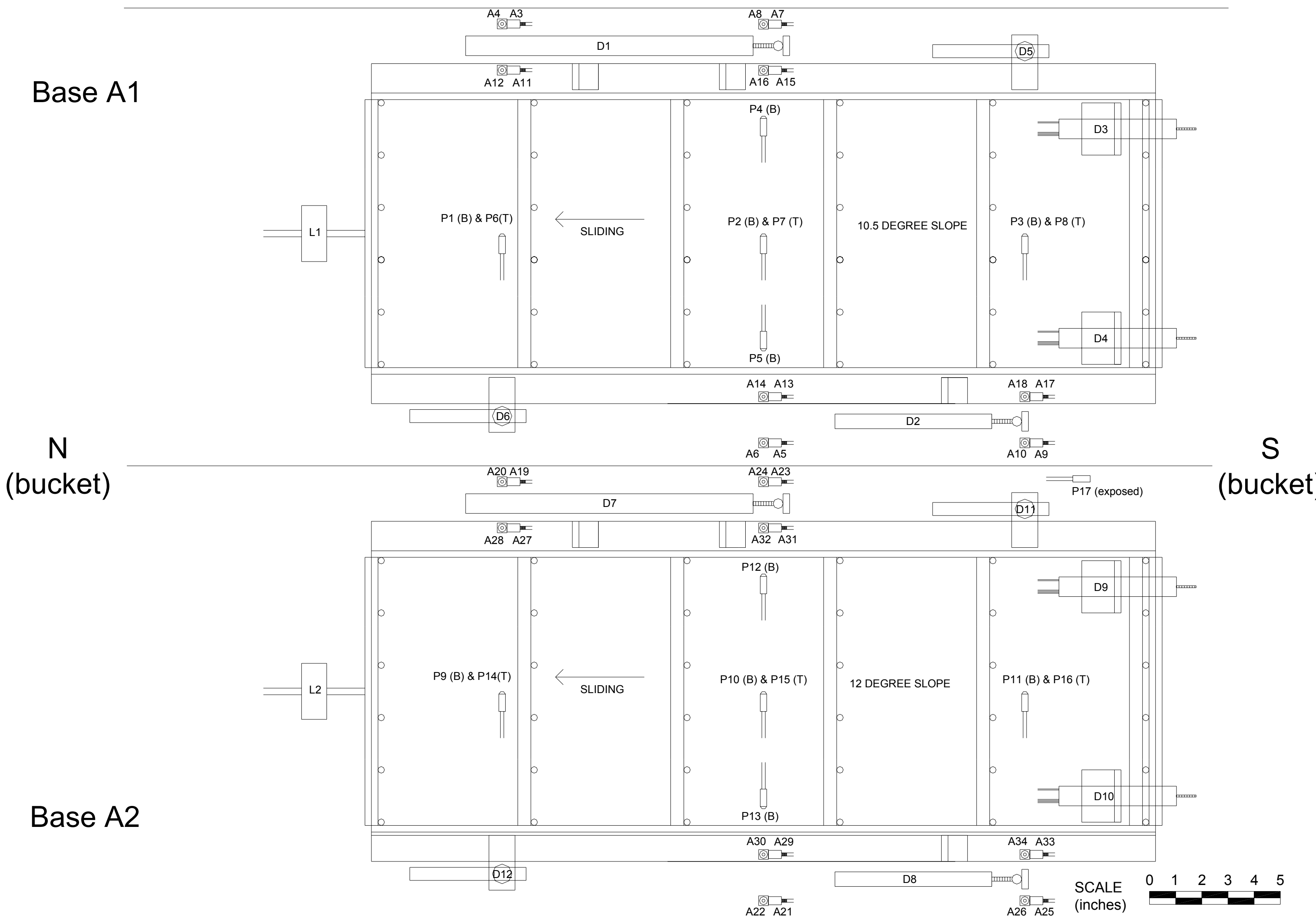


Figure C-11. Instrument Locations.