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 dissertation. 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.

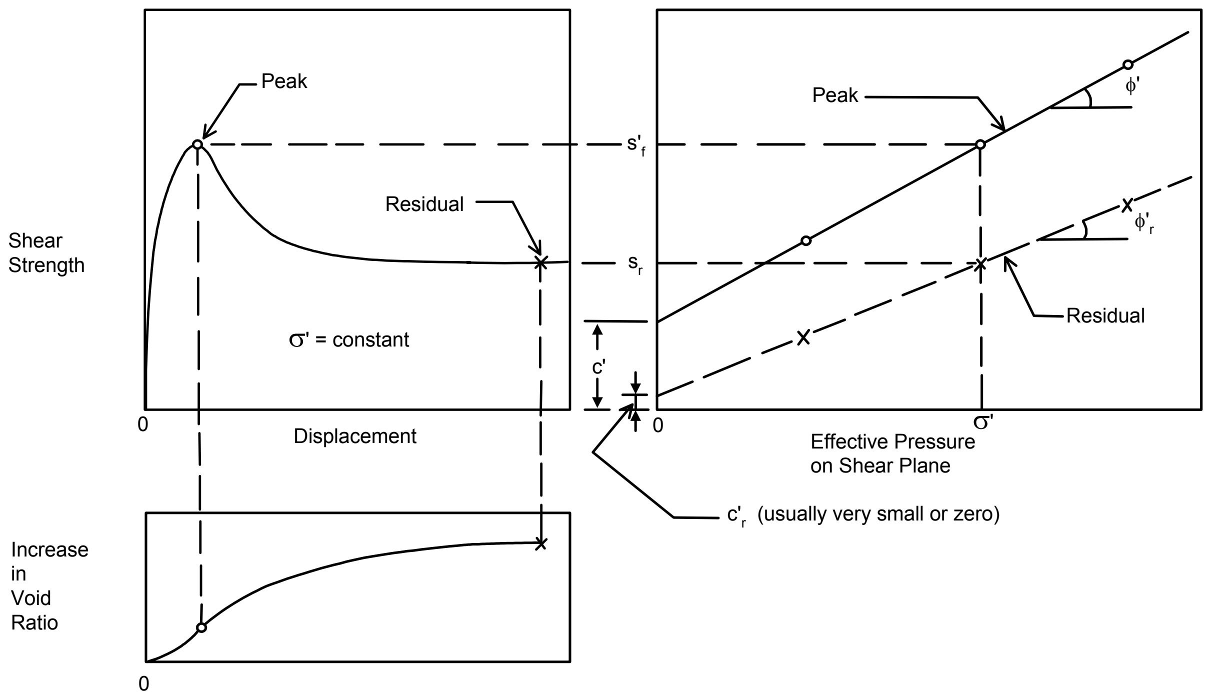

Figure 2-1. Shear characteristics of overconsolidated clay (Skempton, 1964).

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.

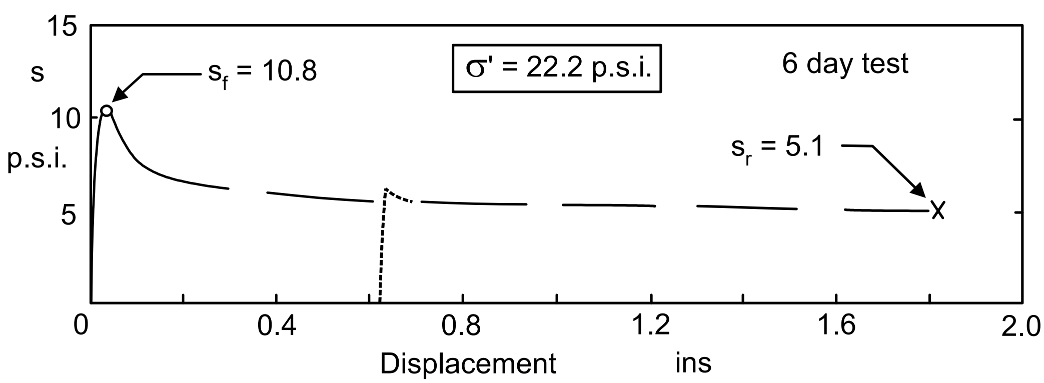

Figure 2-2. Effect of reversal in direct shear tests (Skempton, 1964).

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.

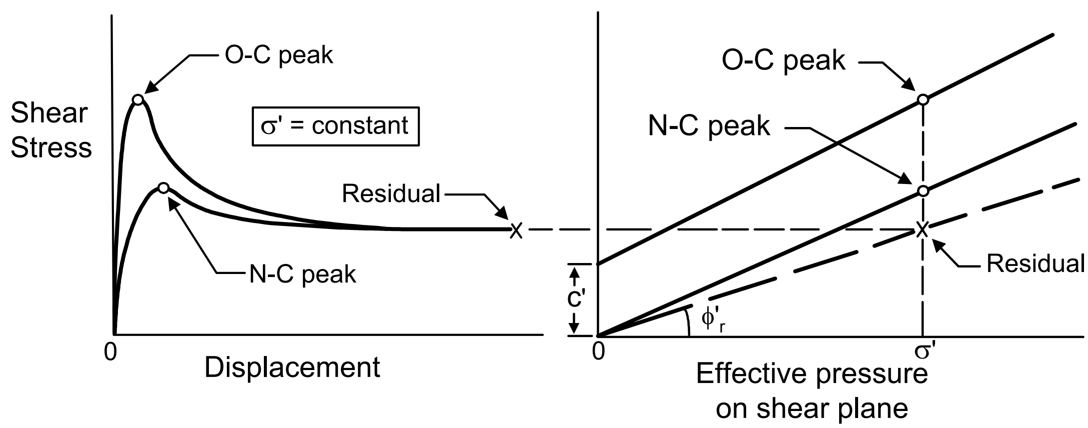

Figure 2-3. Simplified relation between normally and over-consolidated clay (Skempton, 1964).

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.

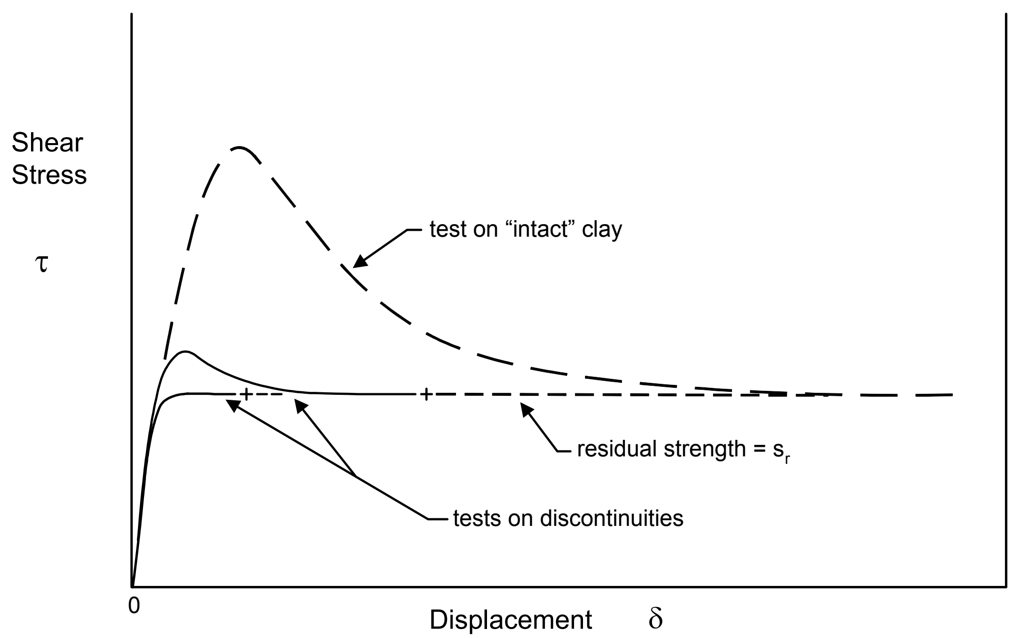

Figure 2-4. Stress-displacement curves from direct shear box tests on a discontinuity and on intact clay (after Skempton and Petley, 1967).

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.

Skempton (1985):

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 ±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.

Slow Shearing of Slickensided Surfaces – Triaxial

Skempton (1964):

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.

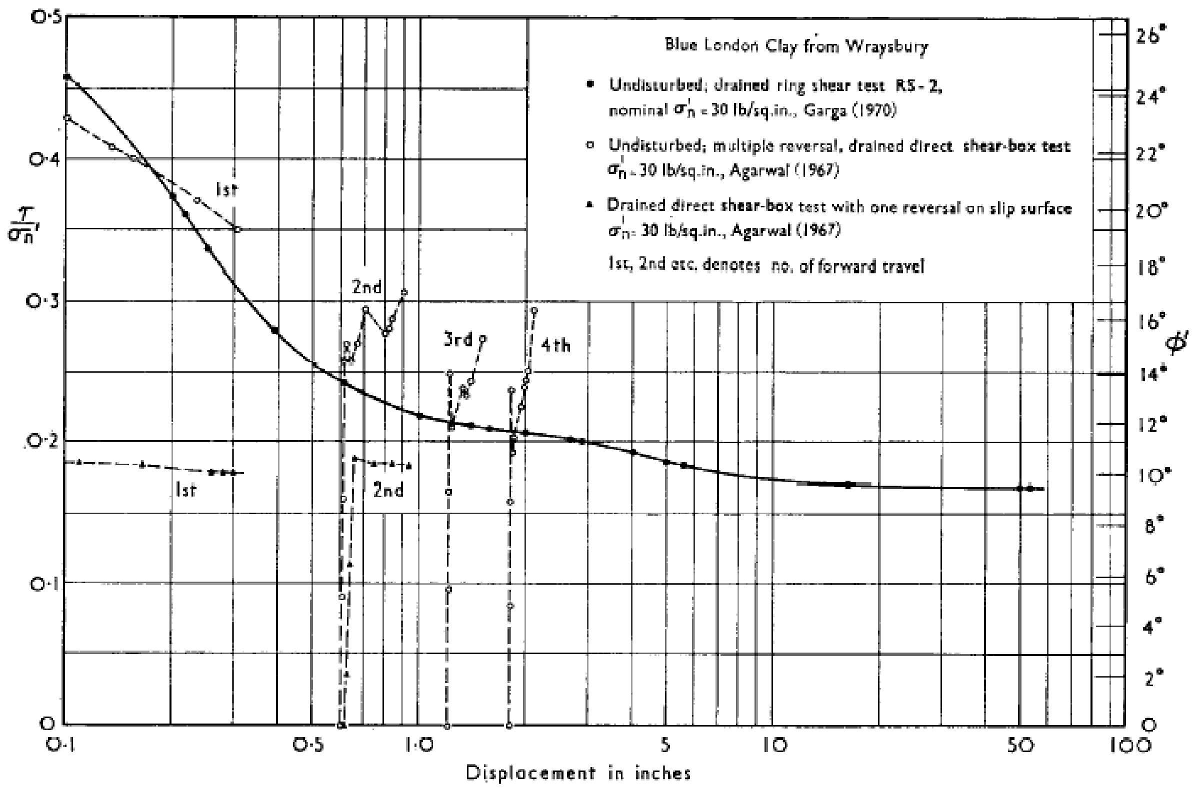

Figure 2-5. Drained ring shear and multiple reversal direct shear test results for blue London Clay (Bishop et al., 1971).

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.

Lupini et al. (1981):

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 dissertation.

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.

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

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.

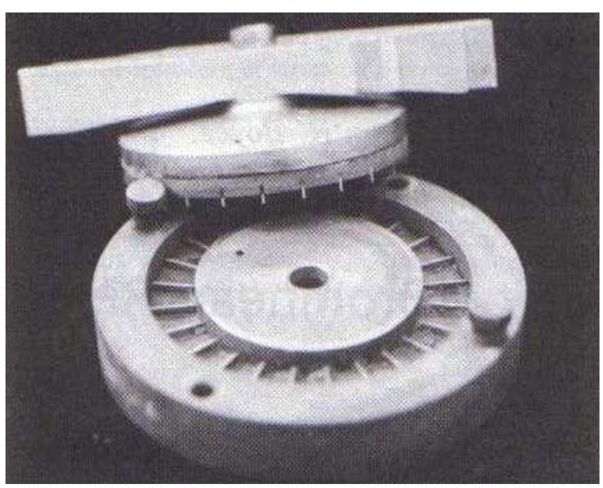

Figure 2-6. Modified specimen container for the Bromhead ring shear device (Anayi et al., 1989).

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).

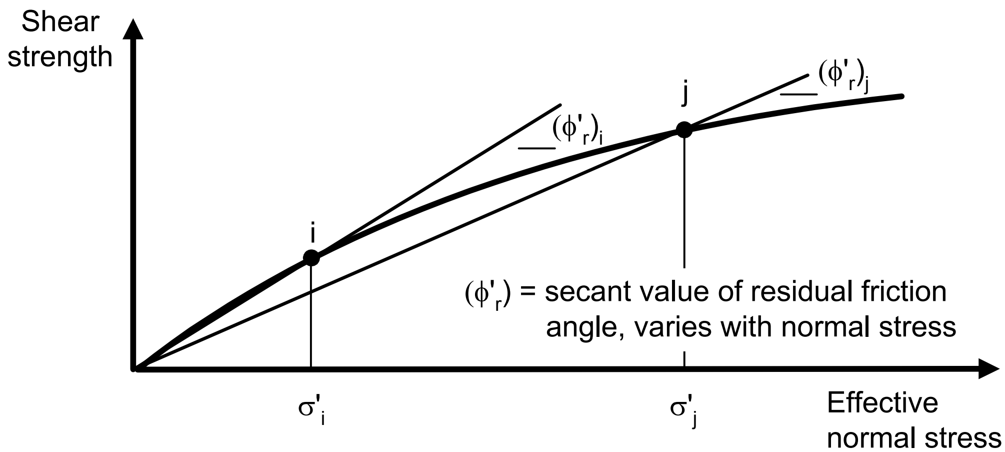

Figure 2-7. Shear strength envelope for a slickensided rupture surface.

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.

<u>Tiwari et al. (2005), and others:</u>

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.

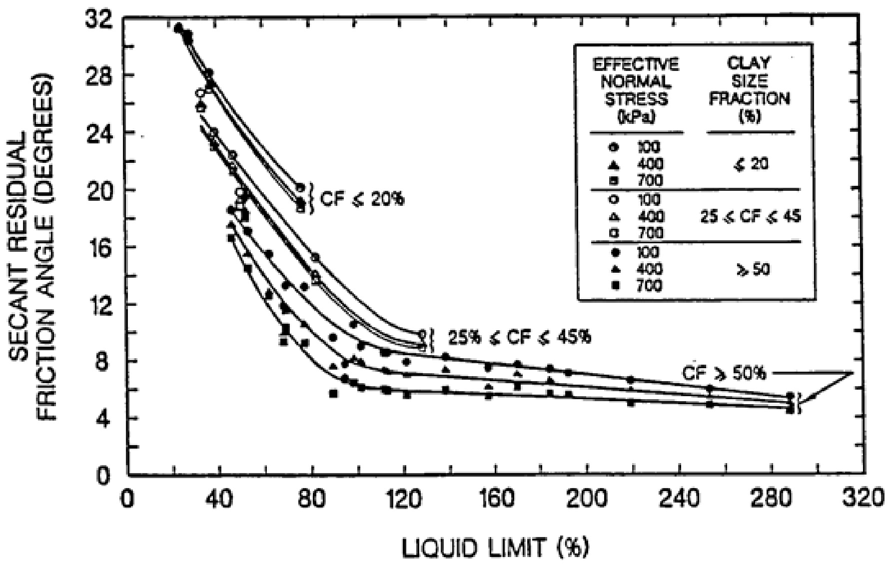

Figure 2-8. Correlation among residual friction angle, Liquid Limit, percent clay size, and effective normal pressure (from Stark and Eid, 1994).

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 2º.

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.

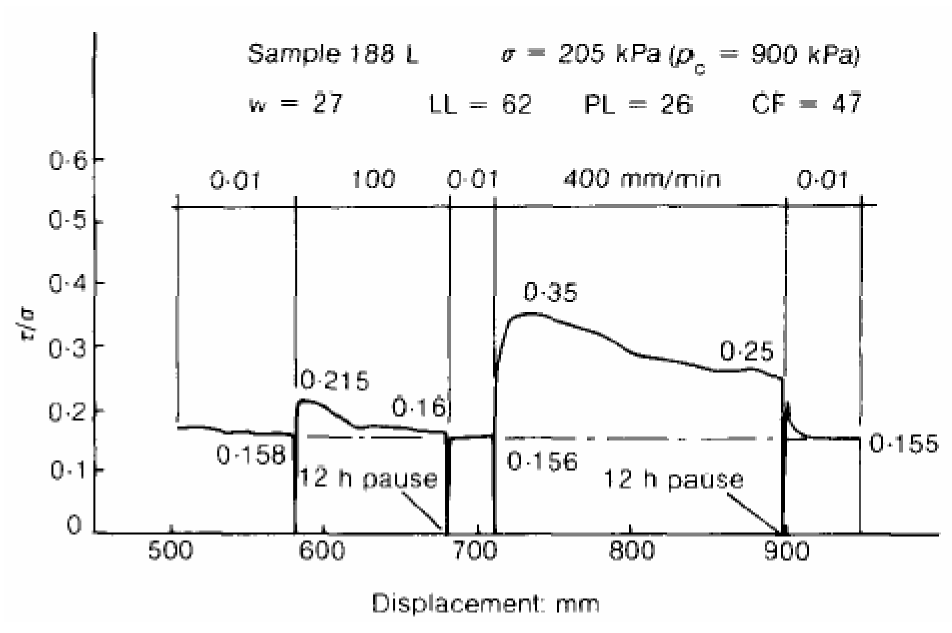

Figure 2-9. The stress-displacement response measured for a Kalabagh Dam clay specimen (Skempton, 1985).

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."

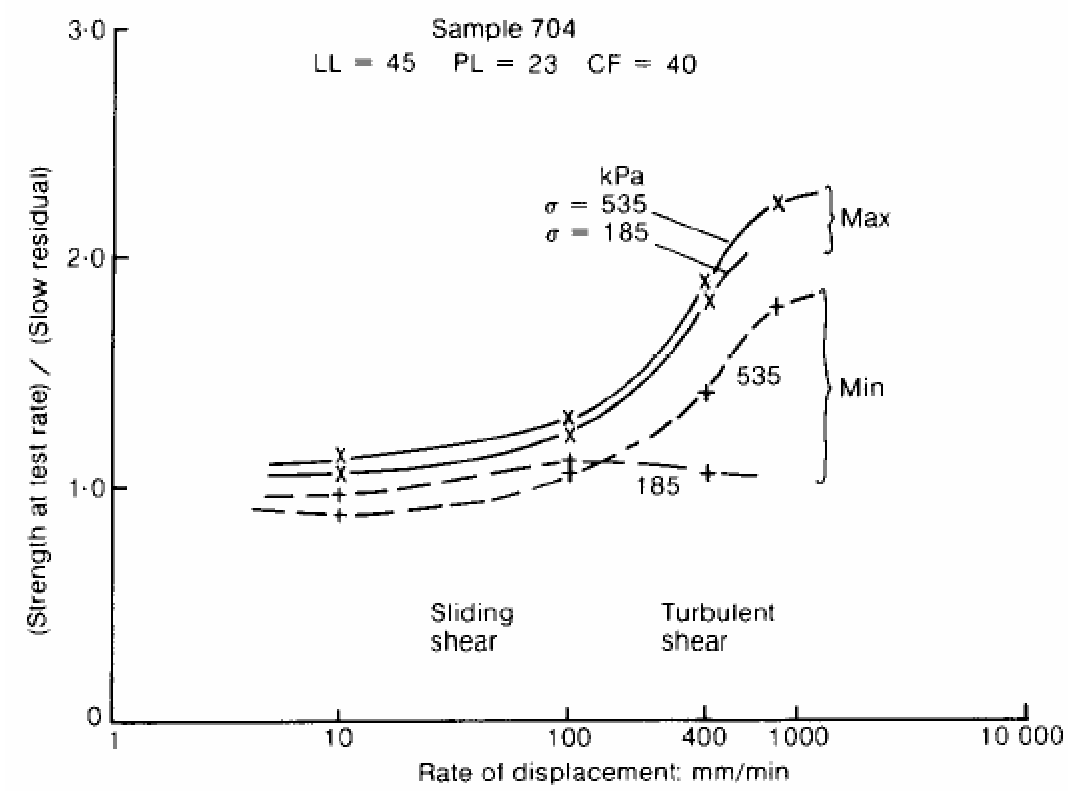

Figure 2-10. Rapid loading strengths measured for Kalabagh Dam clay (Skempton, 1985).

<u>Lemos et al. (1985) and others:</u>

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 <u>drained residual</u> 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).

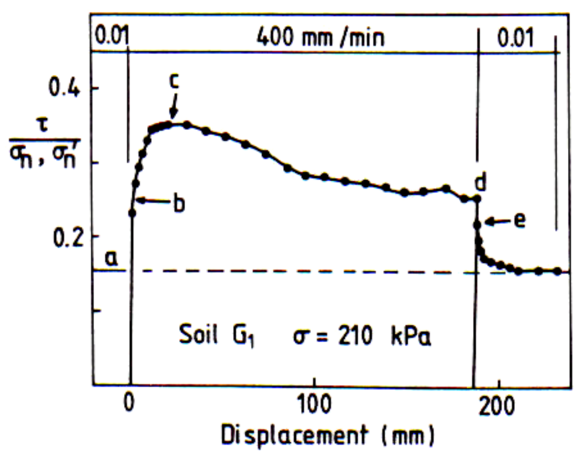

Figure 2-11. Typical results from rapid ring shear tests conducted along existing slickensided surfaces (from Lemos et al., 1985).

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.

<u>Cyclic Testing of Slickensided Surfaces</u>

<u>Yoshimine et al. (1999):</u>

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.

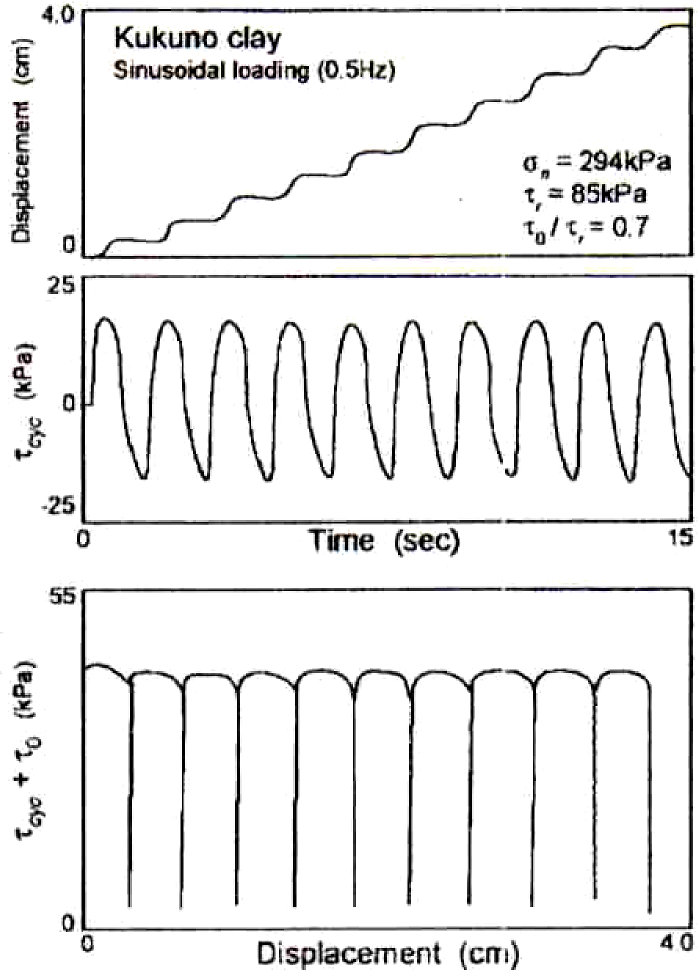

Figure 2-12. Constant amplitude sinusoidal loading of pre-sheared Kukuno clay (Yoshimine et al., 1999).

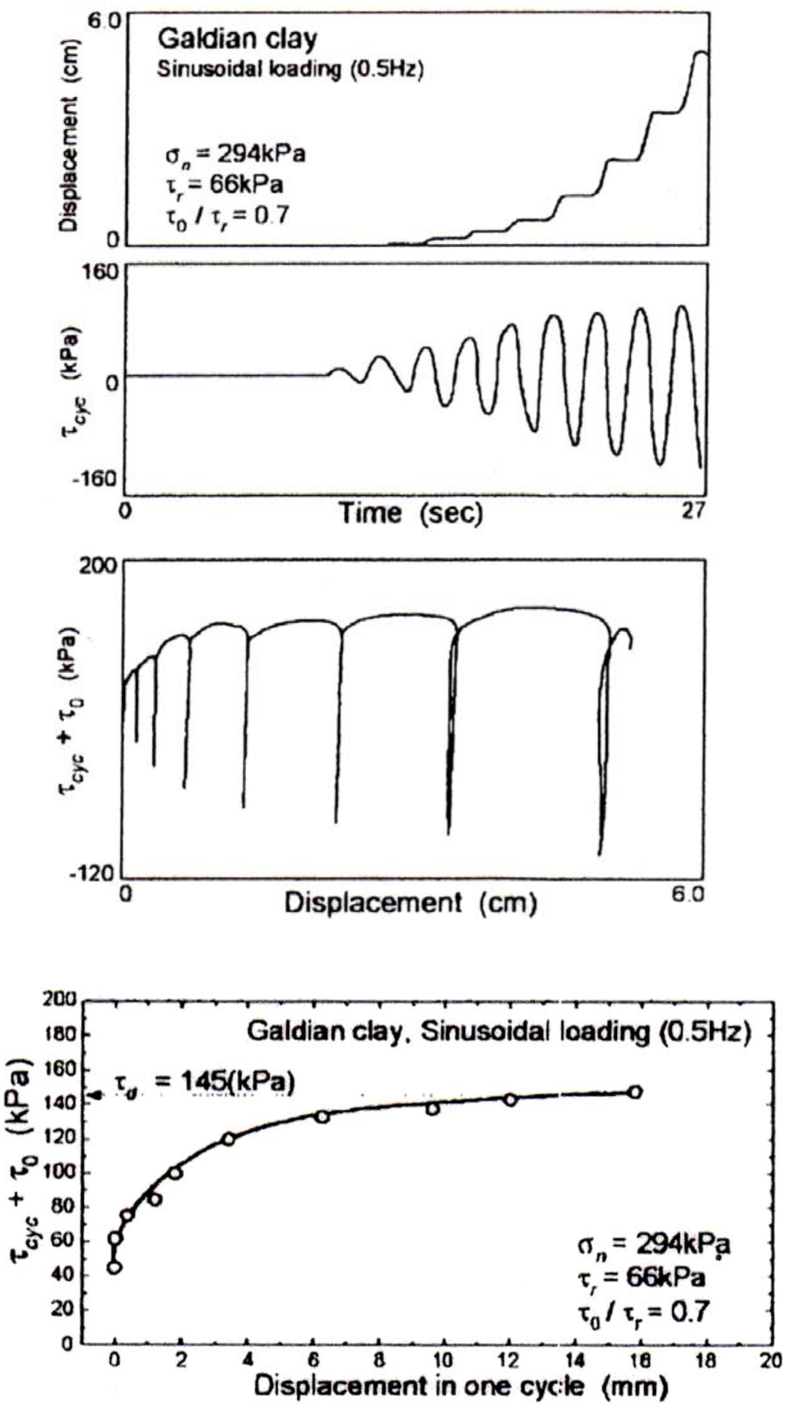

Figure 2-13. Increasing amplitude sinusoidal loading of pre-sheared Galdian clay (Yoshimine et al., 1999).

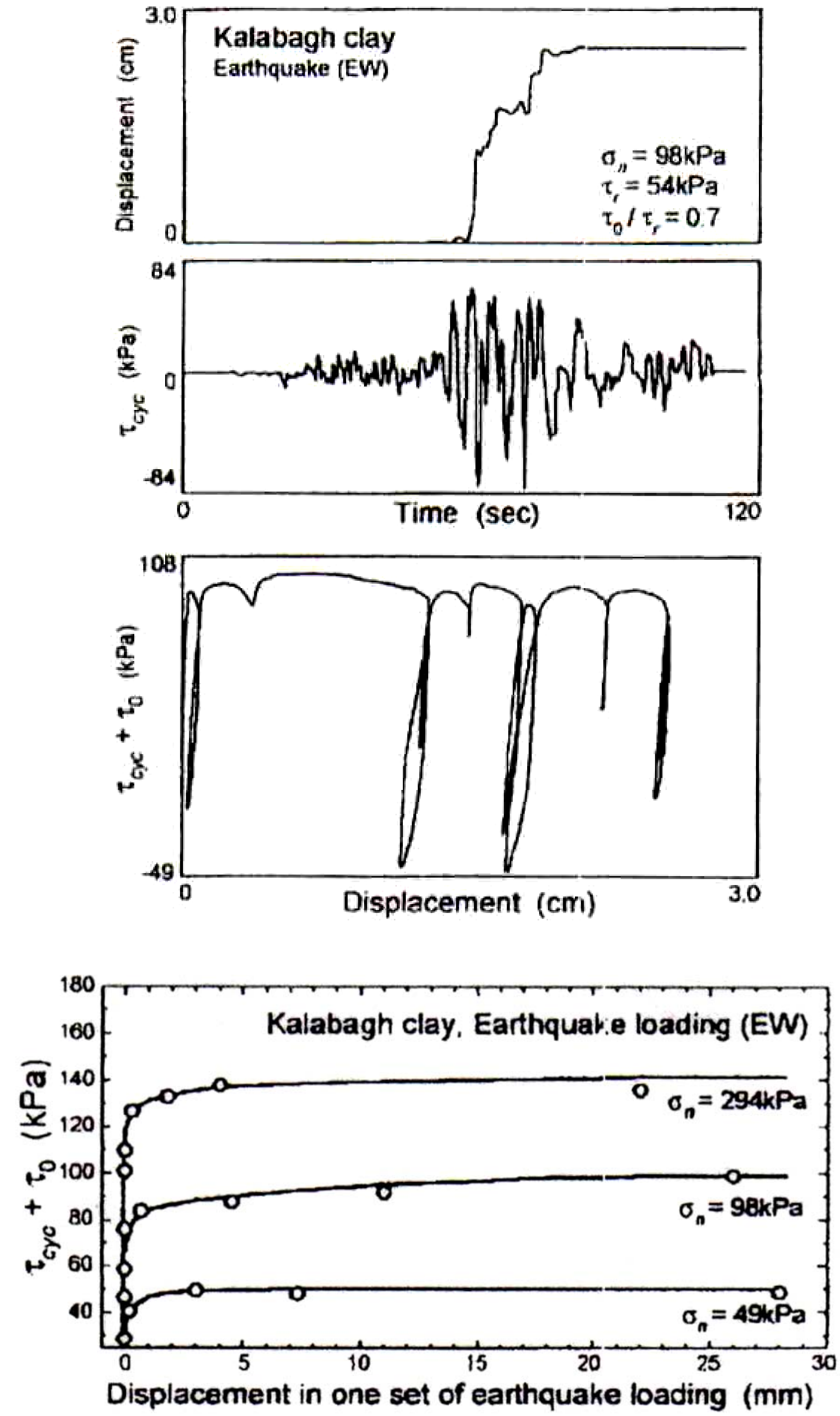

Figure 2-14. Simulated earthquake loading of pre-sheared Kalabagh clay (Yoshimine et al., 1999).

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).

Seismic Slope Stability Analyses

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).

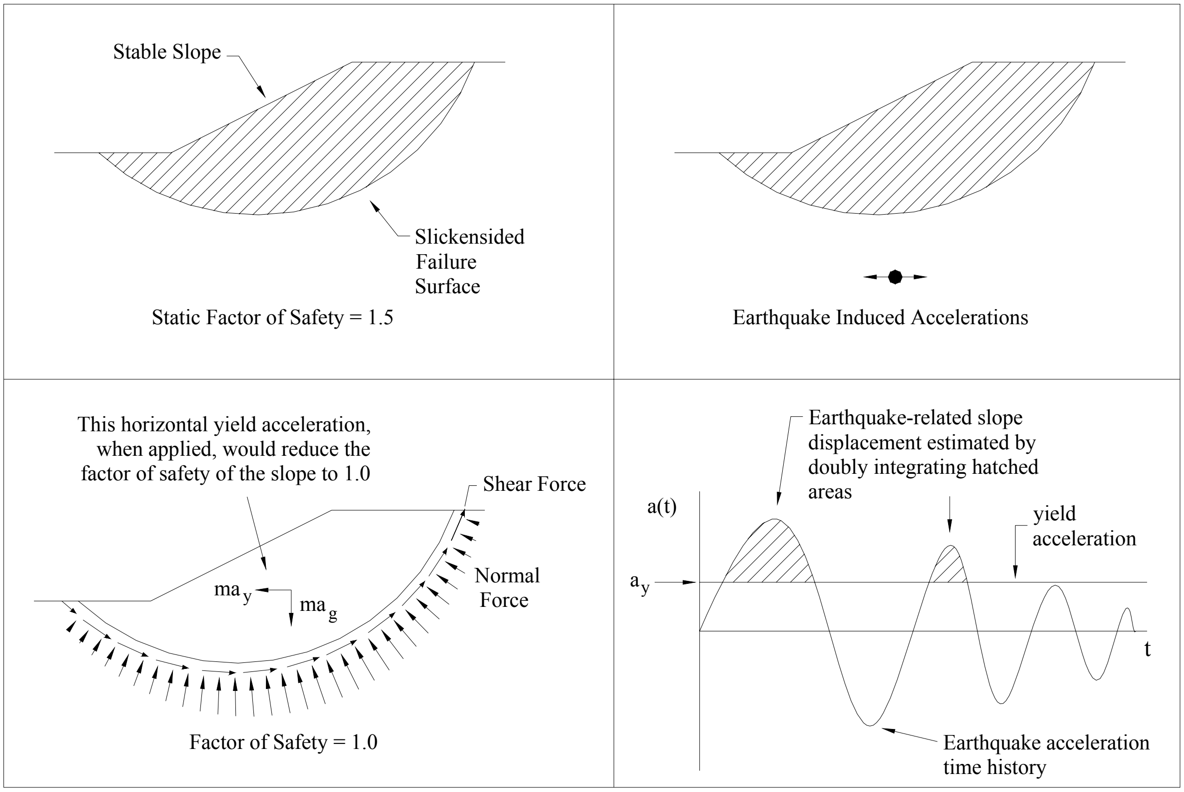

Figure 2-15. Newmark's method for calculating earthquake-induced slope displacements (Newmark, 1965).

<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."

<u>Pradel et al. (2005):</u>

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.