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Seismic Stability of Hillslopes in Greater Cincinnati Area

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SYNOPSIS: Slope stability problems involving colluvia on shale bedrock are common in the Greater Cincinnati Area. The behavior of these hillslopes during earthquakes is, however, not known. This gives cause to speculation whether they are vulnerable to earthquake vibrations. In order to verify the response of these slopes to earthquake induced ground motions, representative composite clay-shale samples obtained from a typical colluvial hillside in Cincinnati were tested in a cyclic direct shear apparatus which was attached to a load cell from the MTS piston actuator. The samples were initially consolidated under a selected normal pressure and then sheared until residual strength was developed. Pulsating strains were then superimposed simulating seismic excitation and the response was recorded by digital equipment. The results from the laboratory experiments were used in the analysis of "infinite colluvial slopes" for a wide variety of assumed conditions.

INTRODUCTION

The Greater Cincinnati Area, located in Southwestern Ohio, is a major metropolitan area with many slide-prone hillslopes (Fig. 1). The instability problems associated with the hillsides are rather common and long-standing in this region. Road closures, construction delays, and destruction of property are but a few examples of typical damage caused by landslides involving large expenditures of time and money (Fleming and Taylor, 1980).

The most damaging landslides in the Greater Cincinnati Area occur in bedrock-derived colluvium that may vary in thickness from a few centimeters to several meters. In general, the shale in the bedrock sequence disintegrates when in contact with moisture and colluvial soils are formed (Fig. 2). The overlying colluvium derived from this weathering process slides down the slope which causes slickenside at the contact surface with the bedrock (Fig. 3a). These types of slides are slow moving (Fig. 3b), and may be triggered by improperly constructed developments, heavy rains, and vibrations, as indicated in Fig. 2.

Although, the slope-movement problems seem inordinately complex, the basic causes for their development under static conditions is fairly well understood, and remedial measures have routinely been taken to stabilize them. The one factor that has not received any recognition regarding the hillslopes of this area in the past is the effect of earthquakes (Zoghi, 1988). It is well known that one of the major causes of damage during earthquakes is earth slope failures (Finn, 1966; Seed, 1967 and 1968). Therefore, the possibility of catastrophic consequences of seismically-induced landslides in this region is of major concern, and it was the determining factor for undertaking of this study. The principal objective of this research program was then to find the effects of earthquakes on colluvial slopes and, specifically, to determine whether the exerted dynamic forces would trigger landslide movements.

TOPOGRAPHY AND GEOLOGY

According to geologists (Fleming, et al, 1981), "the topography of the area is characterized by a rolling upland surface, hillslopes along the Ohio River and major tributaries, and flood plains and terraces of the rivers. Maximum relief in the area is about 160 m. The uplands are generally covered by glacial deposits, mostly till of Illinoian age. Bedrock, consisting of thin-bedded limestone and shale, underlies the glacial deposits at various depths in the upland areas and is exposed on some hillslopes where erosion has removed the glacial cover."

The geologic characteristics of the Greater Cincinnati Area is best described in two parts (Fleming, 1975): (1) the bedrock of the area and the colluvium material derived from the bedrock, and (2) the glacial deposits of the area and the soils derived from them. The materials derived from the bedrock and the glacial deposits are unstable and present the basis for the engineering geology problems. Although landslides do occur in both units, the colluvium related slope stability problems are more common and will be considered herein.

It is well known that bedrock-related landsliding in the Greater Cincinnati Area takes place almost entirely in the colluvium derived from the Kope Formation, or in artificial fill built from the same colluvium. Except in a few instances, where it extends to colluvium overlying the Fairview Formation. The natural slopes in the area are characterized by 14% (8^o) to 44% (24^o) slopes on Kope Formation, and 44% (24^o) to 55% (28^o) slopes on the Fairview Formation. The majority of landslides take place between the elevations of 168 m and 216 m (Fleming, 1975).

The mineral composition of the colluvium of the Kope Formation and its parent bedrock is essentially similar and consists of about 60-90% illite, 10-40% kaolinite and less than 10% chlorite and montmorillonite. The average index properties of the colluvium are 45% Liquid Limit, and 25% Plasticity Index.

The contact zone between the colluvium and the bedrock is somewhat transitional over a few centimeters of thickness. In fact, the exposed failure surface of a slide exhibits a single, shiny, paper-thin surface containing grooves and furrows with an amplitude of up to 0.5 cm aligned in the direction of movement (Fleming, et al, 1981).

Furthermore, there is evidence that the sliding of the colluvial cover over the bedrock involves the state of minimum resistance known as the "residual strength" (Skempton, 1964 and 1985). This is generally associated with progressive movement and the residual strength is markedly lower than the peak strength. In the state of progressive movement, slopes may be most susceptible to earthquake shaking because of the pre-existence of an active or "quiescent" landslide disposition, which are at the state of minimum resistance to movement (Cotecchia, 1987 and Vaughan, et al, 1985). The objective of this research project was to study the links between seismic action, progressive movement, and the initial stress conditions within colluvial slopes.

CHARACTERISTICS OF EARTHQUAKE GROUND MOTION

Generally, the behavior of slopes during earthquakes depends upon the nature of the ground shaking, the geometry of the slopes and their material composition (Cotecchia, 1987; Maugeri and Motta, 1985). The detailed discussion on earthquakes in Ohio is beyond the scope of this paper; however, an excellent review has been presented by Bradley and Bennett (1965). A few remarks considered essential to the present research study, are nevertheless included herein to shed light on the implications of earthquake problems with regard to the instability of colluvial slopes and the solutions that may result from this investigation.

Although the cause of earthquakes in Ohio is not precisely known, they probably are of the interplate type and are clustered mainly in the vicinity of the western Ohio town of Anna, in Shelby county. The majority of earthquakes in Ohio have been of minor intensity, causing little or no damage. The most noteworthy of these were the two that occurred on March 7 and 9, 1937 with intensities of VIII and VII respectively. Considerable damage was done to various buildings in Anna and nearby communities by these two quakes. The state of Ohio is not a seismically active area compared to other parts of the country such as California. The seismic map of the United States, however, designates the southwestern region of Ohio as moderate wi respect to potential damages due to earthquake This latter designation is primarily due earthquakes originating outside the state, su as in Kentucky and Missouri, and felt in Ohi It is known that east of the Rocky Mountains the United States, the quakes have a much larg "felt" area than in the far west (Algermisse 1983 and Bolt, 1978).

The clay-shale samples of this investigatic were subjected to the appropriate number c cycles corresponding to earthquakes of differer magnitudes following the recommendations made k Seed and Idriss (1982). Strain-controlled typ of testing at the frequency of 2 Hertz wa adopted for all specimens. It is believed tha this frequency is representative of most earth quake vibrations (Thiers, Seed, 1968). Further more, sinusoidal load-trace shape was employe throughout this investigation.

SAMPLE COLLECTION AND PREPARATION

The sampling, testing and analysis of colluvia slopes pose a combination of perplexing prob lems. D'Appolonia, et al (1967), affirm thi complexity by stating: "...the slickensides upo which movement has taken place in the past ar hard to locate and difficult to sample. If th intact residual soil is tested, rather than th slickensides that are present in all colluvia slopes, the strength of the soil and hence th factor of safety of the slope can be seriousl overestimated. Even if the slickensides ar sampled and tested, the question of wha strength will be mobilized at failure - th peak, the residual or in between - must still b resolved." Nevertheless the problem is no intractable and the following procedure wa adopted in collecting and testing the colluvium bedrock samples in order to accomplish th objectives of this study.

A trench was dug using a backhoe at one of th typical hillslopes in the Greater Cincinnat Area (Fig. 1). The colluvium material was uncov ered and the slickenside surface was exposed After a number of trials, it was realized tha good quality undisturbed block samples of shal could be obtained by simply using a hammer and chisel. It is worth noting that the intact shale samples were collected from below the potential failure surface due to a slakin problem associated with the slickensides. It was believed that this polished surface could be created artificially on the shale specimens by trimming the top surface smooth in the labora tory. The block samples obtained were wrappe in aluminum foil and enclosed in zip-lock plastic bags, transferred to a larger two ply plastic bags and were shipped immediately to the moist room in the laboratory.

In the laboratory, prior to testing, the block shale samples were reduced to smaller round specimens with diameters little larger than 6 cm using the band-saw. Then, a sharp knife was employed to trim the excess so that the specimens could fit snugly inside the shear device. The excess shale from the trimming process was ground and then passed through the number 40 sieve. This served as the colluvium or soil) portion of the combined prism.

ince the main objective of this investigation as to verify the characteristics of bonding at he colluvium-bedrock-interface, the soil obained from the trimming process, as explained bove, was smeared over the relatively unisturbed sample of intact shale following the rocedures set forth by Kenney (1967) and Noble 1973). The combination of the clay-shale ample was placed inside the cyclic direct shear ox (to be described in next section) such that he plane of interface lined up with the split cross the middle of the box. The sample was hen subjected to either several repeated eversals of shearing in order to achieve the tate of minimum resistance (residual strength), r was simply displaced a predetermined amount n one direction before superimposing the cyclic oading.

PPARATUS AND TEST PROCEDURES

he investigation of soil behavior both under tatic and dynamic conditions poses several ifficulties, one of which is the design of a uitable apparatus with the capability of reroducing field conditions in the laboratory. onsiderable advances in the instrumentation and esting ability, including tests of slope odels, have been made in recent years (Prakash, 981). A thorough investigation of the availble dynamic testing equipment reveals that each ossesses its own unique features, advantages, nd limitations (Bhatia, et al, 1985a).

ince during an earthquake the soil deformation s primarily attributed to the upward propaation of shear waves from underlying bedrock, he cyclic simple shear apparatus seems to imulate more closely the actual in-situ condiions (Arango and Seed, 1974). This criterion, oupled with the concept of residual strength nd involvement of clay-shale composite samples, ecessitated the use of a cyclic direct shear evice which could impose a predetermined plane f failure at the clay-shale interface.

he apparatus employed in this investigation was ssentially similar to the conventional static irect shear device, except for a few modifiations which were made so as to accommodate ynamic applications in addition to the ordinary tatic operations. The equipment consisted of n aluminum box that was split horizontally cross its middle. A Bellofram pneumatic load ell was used to apply the normal load to the oil sample. This was accomplished through the se of a unique framing arrangement as illustrated in Fig. 4.

The frame consisted of a rectangular aluminum plate, 5 cm wide by 16 cm long by 2 cm thick, and two 1 cm diameter threaded posts. There were two oversized holes at the far ends of the plate and another hole (1 cm in diameter) right in the middle. The posts passed through the end holes of the plate and were screwed into the top part of the shear box. A set of nuts were used to adjust the location of the plate and a cap which sat directly on top of the sample. The plunger of the load cell was threaded which made it possible to be screwed into the cap from one end, whereas the special fitting at the other end of the load cell passed through the middle hole of the plate and linked into a hose from the air compressor.

The bottom half of the shear box included an extended piece of plate on each side (in the shape of the outstanding leg of an angle) to make provisions for bolting (or clamping) the lower half of the box down to a platform. The top half of the shear box was attached to an MTS electrohydraulic closed-loop actuator by means of a yoke. Thus, during actual testing (Fig. 5a), the lower half of the box remained stationary while the upper portion was free to move in either direction. The shear force was measured by a load cell, and the deformation by an LVDT (Linear Variable Differential Transducer). The test signals from these instruments were amplified and recorded on a disk using a Nicole digital oscilloscope containing two disk drives (Fig. 5b).

Among others, the underlying factor that led to the decision in adopting this particular equipment for this project was its ability of imposing a predetermined plane of shear failure. As the upper half of the box was displaced relative to the lower half, the sample tend to shear on a plane across the split of the box. This, of course, was of great value in testing the samples of colluvial slopes since the combination of the shale-clay prism could be trimmed and positioned inside the shear box in such a way that the potential plane of weakness (obviously at the interface of the two materials) coincided with the split across the box.

It is known that saturated soil element subject to seismic loading tends to develop excess porewater pressure due to the progressive rearrangement of the soil particles during each successive cycle of loading (Bhatia, et al, 1985b). In a drained test, this process causes a large decrease in volume. However, in an undrained condition, the intergranular stresses of the soil decrease significantly while maintaining a constant volume (France and Sangrey, 1977). It is a fact that the duration of an earthquake is much too short for the drainage to take place in an element of soil (Seed and Chan, 1966, Lambe and Whitman, 1968). Thus, the undrained testing is prevalent under repeated loading induced by an earthquake. Consequently, the samples tested in this research were enclosed in a rubber membrane to prevent drainage, and an attempt was made to monitor the excess pore-water pressure during the cyclic part of the test. There were two through-holes available in the cap of the shear box which permitted the drainage of the sample under the consolidation period of the experiment. Also, by means of special fittings, two piezometers were inserted into the sample through the above holes for pore-water pressure measurements. The tip of one piezometer was positioned at the interface of the clay and shale, while the other was located right above the clay prism. The monitoring was made possible by means of highly sensitive porepressure transducers. The signals from these instruments were amplified and recorded on the other disk of the Nicole digital oscilloscope.

Although, extreme care was exercised in all the stages of planning and design to ensure accurate test results, the elimination of a few mechanical deficiencies associated with this apparatus was not possible. For example, similar to cyclic simple shear devices, wall friction, sample disturbance due to the stress concentrations near the boundaries, uniformity in the distribution of shearing stress over the failure surface, and the change in the area of the sample as the test progressed comprised some of the shortcomings of the cyclic direct shear apparatus.

LABORATORY EXPERIMENTAL PROGRAM

The sampling and testing procedure adopted in this study was fully explained in the preceding section. In this part, the results of the direct shear tests will be presented in the following order: (1) static tests on clay-shale samples; (2) cyclic tests superimposed on samples subjected to initial sustained static stresses representing various states of slope stability, expressed in terms of their factor of safety; (3) static tests on the samples previously tested in step (2).

(1) <u>The Static Direct Shear Test Program</u>

A wide variety of static direct shear experiments were conducted on the composite clay-shale samples in order to establish a trend in their stress-strain characteristics and to check the influence of the many parameters on the outcome of test results. It is important to note that the samples were consolidated under various levels of normal load prior to the application of any shear tests.

A typical shear stress-strain diagram is shown in Fig. 6. It may be observed from this figure that the stress-strain relationship remained essentially linear up to a strain value of about 1% and a stress level that was near 30% of peak strength. Beyond that, the rate of change of strain increased rapidly with higher stresses. The peak shear stress intensity (shear-strength) of the soil sample was reached at about 10% of strain. Further increases in strain corresponded to an abrupt reduction of shear stress reaching a low residual value and eventually failure.

It is deduced from the above observations, and those made in the previous sections, that a strong bond existed between the clay and the underlying shale prism and even after the post-peak drop-off, the composite sample still exhibited some strength (introduced as residual strength in the previous section). It is also apparent from this figure that the peak stress level is very much pronounced, which is the characteristic of an overconsolidated clay.

(2) <u>The Cyclic Direct Shear Test Program</u>

It is recognized that seismic shocks tend to increase the shear stresses within a soil mass and decrease its shear strength, with a potential for failure or excessive deformation (Chaney, 1985; Ellis and Hartman, 1967). The pre-earthquake condition of a typical soil element in a slope is depicted in Fig. 7a. The sample is in a state of initial shear stre corresponding to the state of stability of t slope, usually defined in terms of a "factor safety." During an earthquake, the pulsati shear stresses are superimposed on the initi pre-earthquake stresses (Fig. 7b).

In general, the composite samples of the cla shale in this investigation were subjected three distinctively different combinations pre-earthquake" and "during earthquake" sheari stresses. One group of these samples was su jected to cyclic stresses with no initial she stresses present, so as to identify t characteristics of the samples from so formations with level ground surface condition. The second group of samples, which comprised t majority of the tests, were subjected to vario intensities of pre-earthquake shear stress before the cyclic earthquake shear stresses we superimposed. The third group, consisted samples that were first sheared slowly in bo directions several times to a state of residu strength, to which the seismic shear stress were then applied. As explained previousl this mode of stressing closely represents t state of stress and strength typical colluvial slopes. The results from this latt type of testing will be discussed herein.

Since a strain-controlled testing approach wa used in this investigation, the amplitudes deformation remained constant for a given tes while the load response was recorded. One su record is demonstrated in Fig. 8. As the numb of cycles increased, a gradual decrease in th intensity of peak shear stress was observed.

Fig. 8 exhibits the initial segment of stre and strain records for a typical sample with r initial static shear stress, representing leve ground conditions and used as a reference. Th behavior of other samples, subjected to vario initial and cyclic loading conditions (explaine in foregoing paragraphs) was different, de pending upon the sustained initial stati conditions imposed on the samples. For ir stance, Fig. 9 shows the load response of sample subjected to a pre-earthquake shear de formation.

It can be seen that, unlike the above stres pattern, the cyclic stress variation is n longer symmetrical with respect to the initia reference static stress level line denoting th inelastic characteristics of the clay to cycli loading (Arango and Seed, 1974).

As for the samples representing the slopes a the state of residual strength (reactivate slides), they were first subjected to severa slow back and forth motion prior to cycli applications. The stress-strain history diagra and the corresponding normalized cyclic shea stress graph are shown in Figs. 10a and 10b respectively for a typical specimen.

It is apparent that the dynamic shear stres mobilized during the first quarter cycle wa relatively low and only a minute reduction i the value of the normalized cyclic shear stres is observed for the very first few cycles o repeated loading. Finally, the residual cycli shear stress was achieved after about 20 cycles i.e., the magnitude of mobilized cyclic shea tress remained constant thereafter for the emainder of the test.

t is postulated that the progressive breaking f adhesive bonds between the upper clayey aterials and the lower shale prism, as well as he gradual face-to-face arrangements upon the irst few slow shearing reversals were the nderlying factors for the behavior of the above amples during the superimposed repeated oading. Accordingly, the composite clay-shale amples at the state of residual strength offer onsiderably lower resistance during the arthquake ground shaking than similar samples rom slopes possessing higher initial factors of afety against instability.

3) <u>Results of Static Tests on Clay-Shale Samples Previously Subjected to Cyclic Loading</u>

ll the clay-shale specimens of this study were ubjected to post-earthquake undrained static hear loading in order to investigate the ffects of preceding undrained cyclic loading on heir undrained static strength. A typical post-eismic shear stress-strain diagram is shown in ig. 11. An examination of this figure in omparison with the corresponding pre-seismic tatic shear test (Fig. 6) illustrates that the pecimen did not exhibit a pronounced peak value ny more and there was no significant post-peak rop off.

During this stage of experiment, all the samples were subjected to about one and one-half cm deformation. Most of the samples did not exhibit significant drop in shear strength beyond the peak value as indicated in Fig. 11. The specimens that were first sheared to the state of residual strength prior to the application of cyclic straining and subsequent static deformation did, however, exhibit a further drop beyond the peak shear strength. These latter specimens attained complete failure during the post-seismic static shear testing.

In summary, the above observations signify the fact that even if failure is not reached during the cyclic loading, the clay-shale samples will suffer considerable deformation. Also, the bond at the interface will deteriorate. Consequently, the undrained cyclic loading will cause a significant reduction in the undrained static shear strength.

CONCLUSIONS AND RECOMMENDATIONS

The static shear test results on clay-shale samples demonstrated pronounced peak shearing stresses which dropped to the minimum "residual" strengths upon further shearing. It can be concluded that a relatively strong bond exists at the interface of clay and shale for the first-time slides (i.e., the hillslopes with an operational strength in excess of the residual strength). However, this bond tends to deteriorate somewhat once their residual strength has been activated, representing the conditions of progressive failure (or reactivated slides).

The majority of the samples in this investigation exhibited considerable reduction in strength subsequent to pulsating strain applications. The progressive breaking of adhesive bonds between the clay and shale, as well as the gradual reorientation of platy clay particles into parallel face-to-face arrangements upon cyclic straining appears to be the underlying cause for the reduction in strength. It is of paramount importance to consider this drop in strength in seismic slope stability analysis.

The study undertaken herein has provided a better understanding of some of the perplexing problems encountered in seismically-induced colluvial hillslopes in Cincinnati Area. To further the knowledge of the consequences of the landslide hazard of these slopes, the following recommendations are made.

The majority of tests conducted herein represent thin colluvial slopes. In future studies, thicker colluvia over shale also require attention. This may be accomplished by increasing the magnitude of normal stress on the laboratory test samples.

Furthermore, considerable effort was expended in this investigation to measure the pore water pressure both at the interface of the clay-shale prism and at the top of the clay. It is inferred that the migration of pore water through the clay material to the top surface of the impervious shale would reduce the existing bond resistance and could contribute to the overall failure.

In spite of all the careful preparation, no pore water pressure was recorded throughout the experimental program. The underlying cause for the lack of excess pore water pressure development is not evident. It is a well known fact that, among other factors, the cyclic mobility and the generation of excess pore pressure are closely associated with the overconsolidation ratio, number of cycles, frequency of loading, initial shear stress, as well as the test method. Alternative pore pressure measuring techniques should be considered in future studies in order to clarify the outcome of this investigation.

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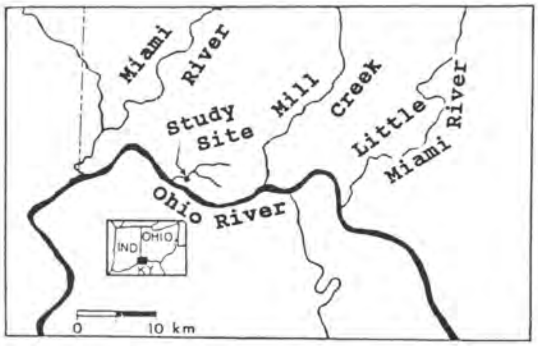

Figure 1. Location of the Study Site (Fleming, 1975).

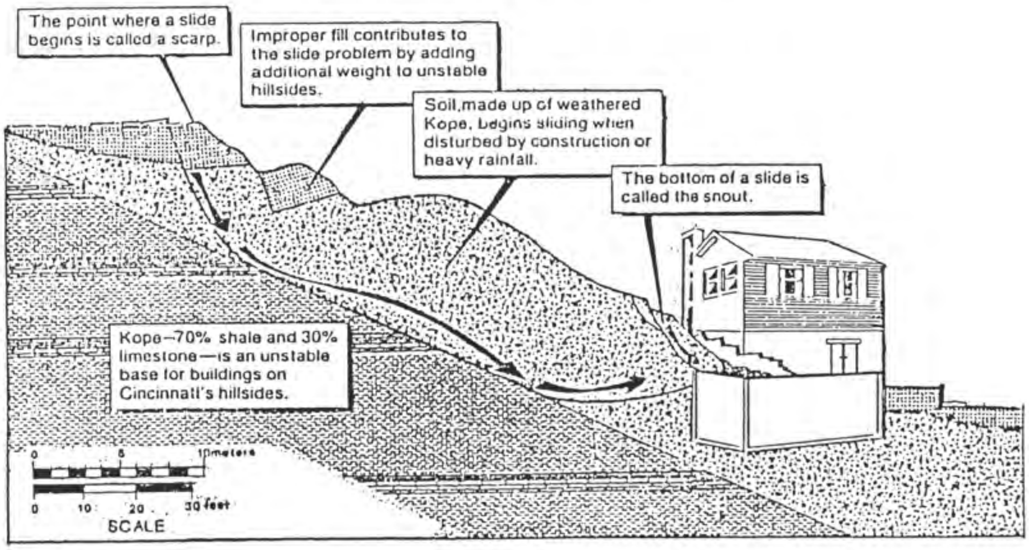

Figure 2. Cross-Section of a Typical Landslide in Colluvial Material on the Kope Formation (Wetenkamp, 1986).

Figure 3a. Exposed Slickenside Surface.

Figure 3b. Creep at Toe of a Sensitive Slope.

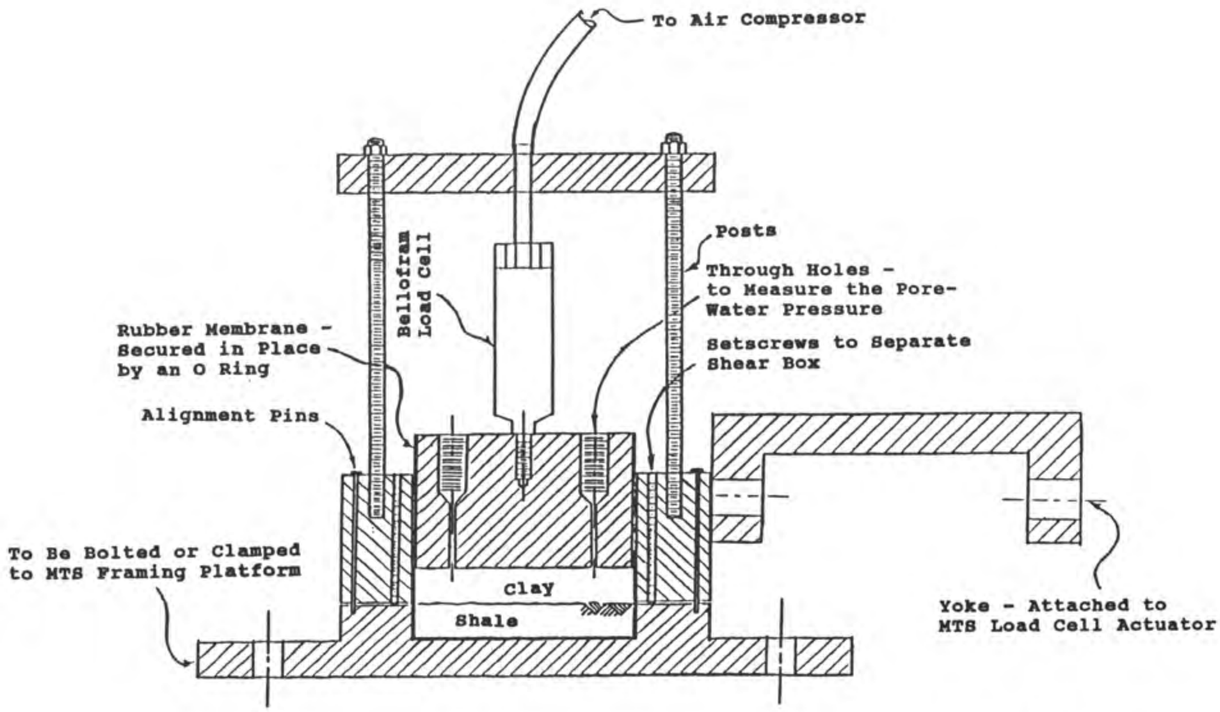

Figure 4. Cyclic Direct Shear Apparatus. Not to Scale

Figure 5a. Typical Test Set-up.

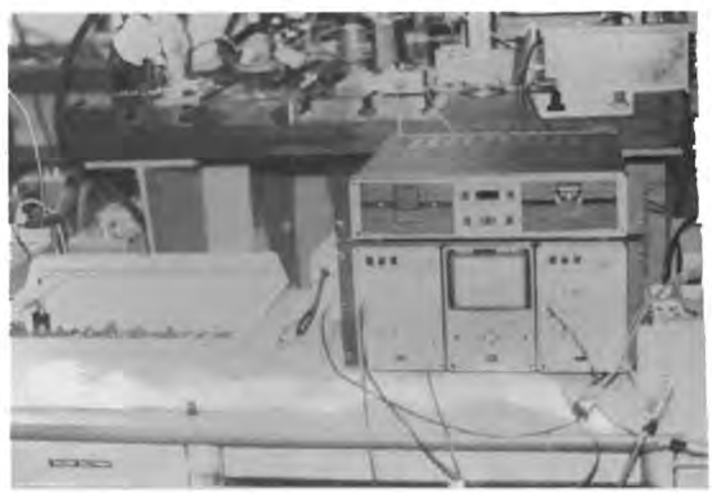

Figure 5b. Recording and Plotting Instruments.

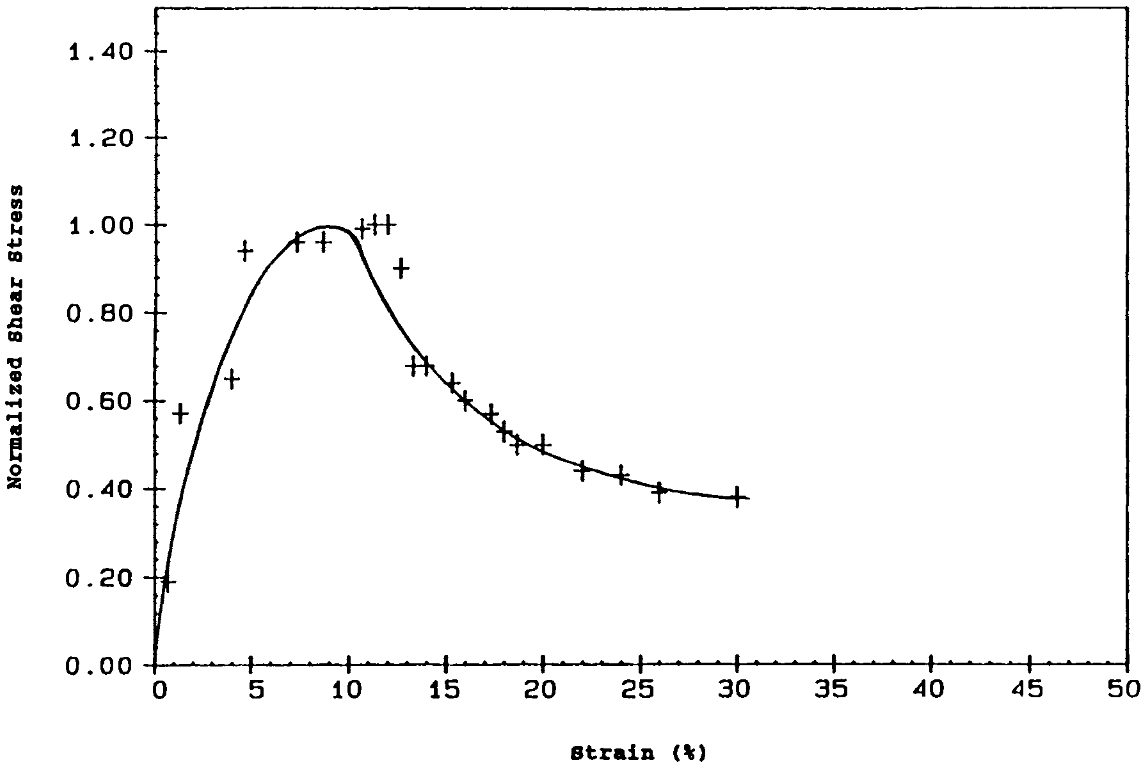

Figure 6. Typical Static Direct Shear Test.

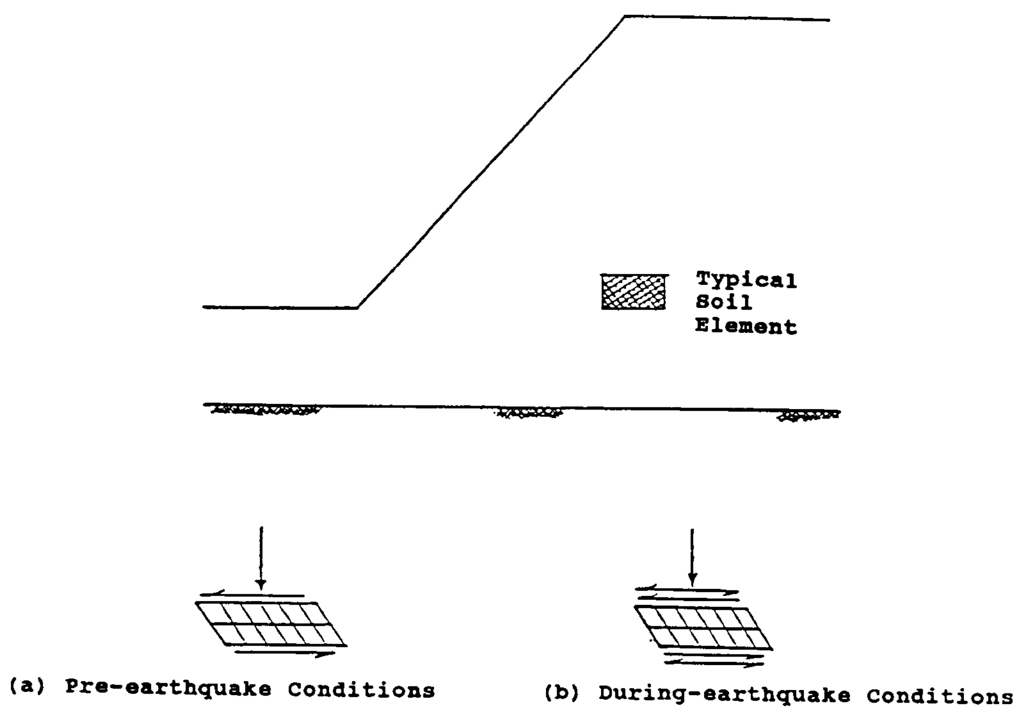

Figure 7. Field and Laboratory Cyclic Loading of Soil Element (Arango, et al, 1974).

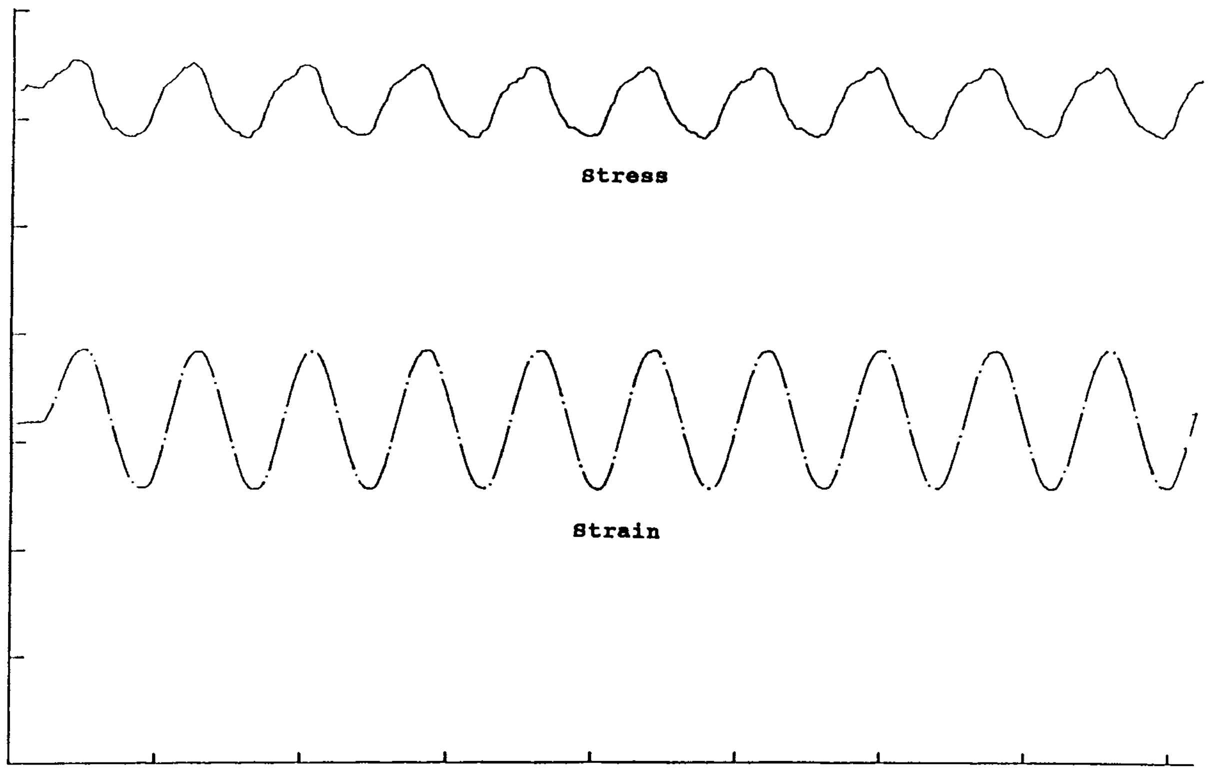

Figure 8. Typical Stress and Strain Time History Diagrams with No Initial Sustained Shear Stress (or Strain) Present (Using Strain-Controlled Test).

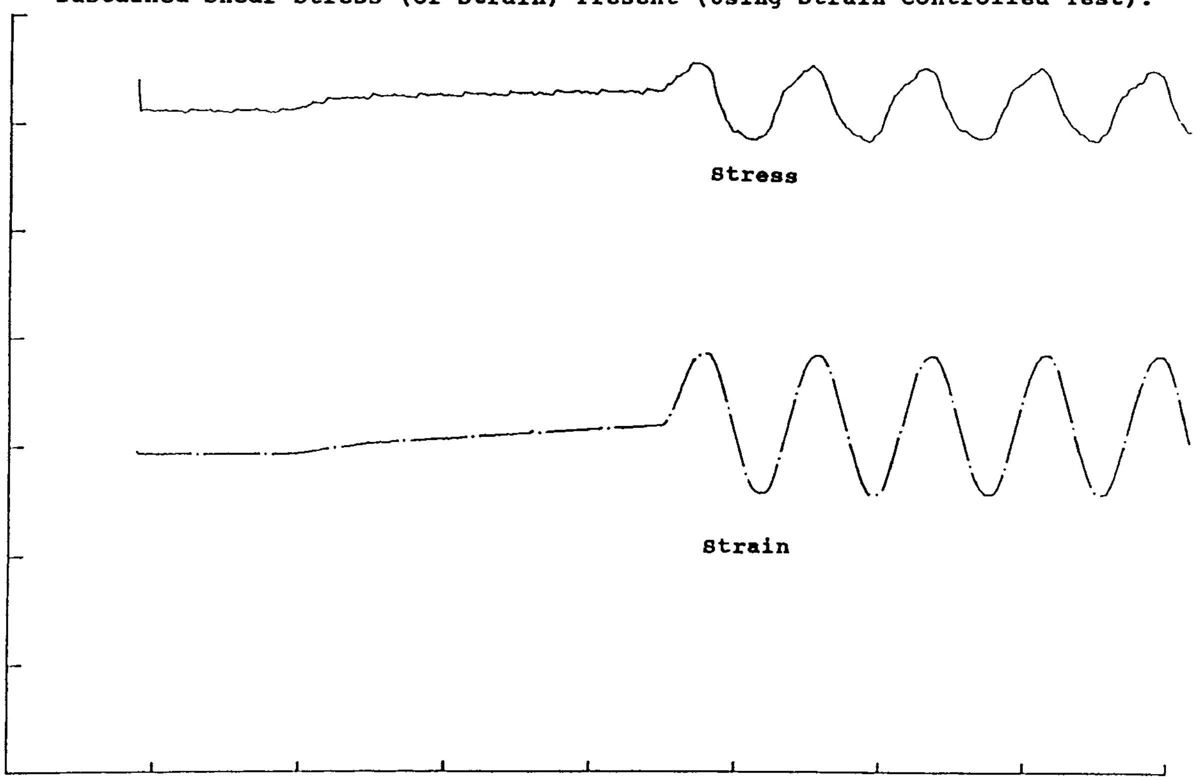

Figure 9. Typical Stress and Strain Time History Diagrams with Initial Sustained Shear Stress (or Strain) Present (Using Strain-Controlled Test).

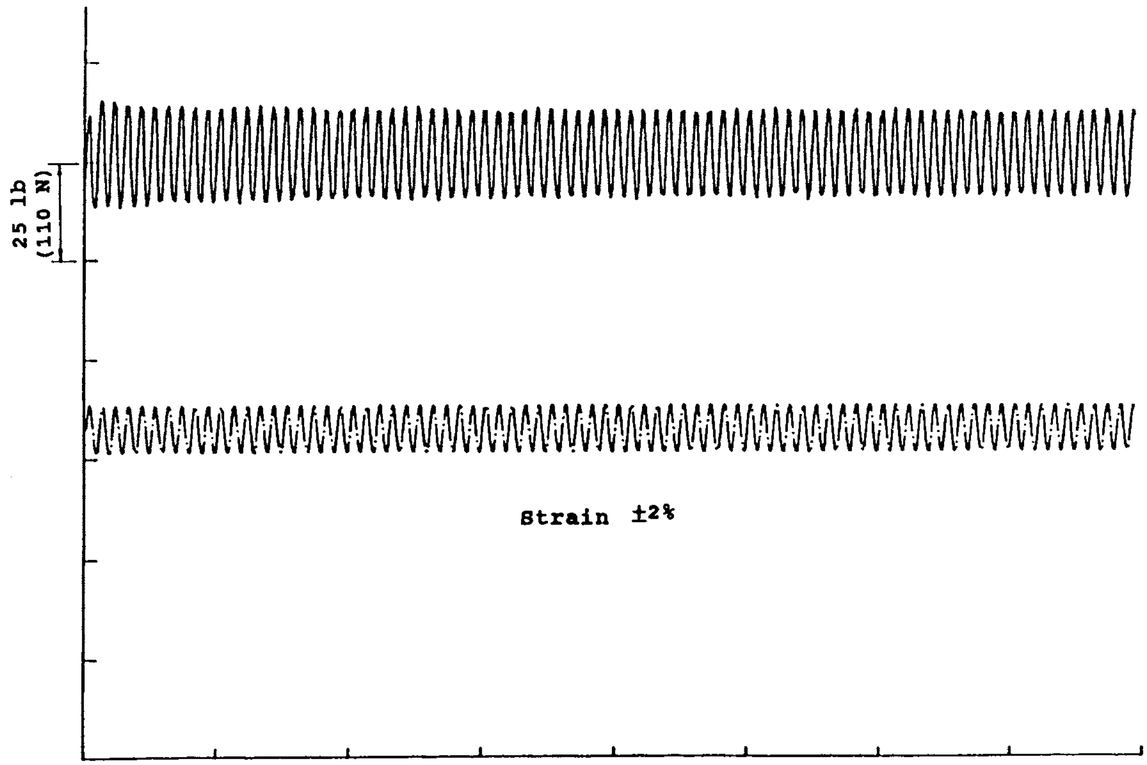

Figure 10a. Typical Stress and Strain Time History Diagrams (for a Sample Representing a Slope at the State of Residual Strength).

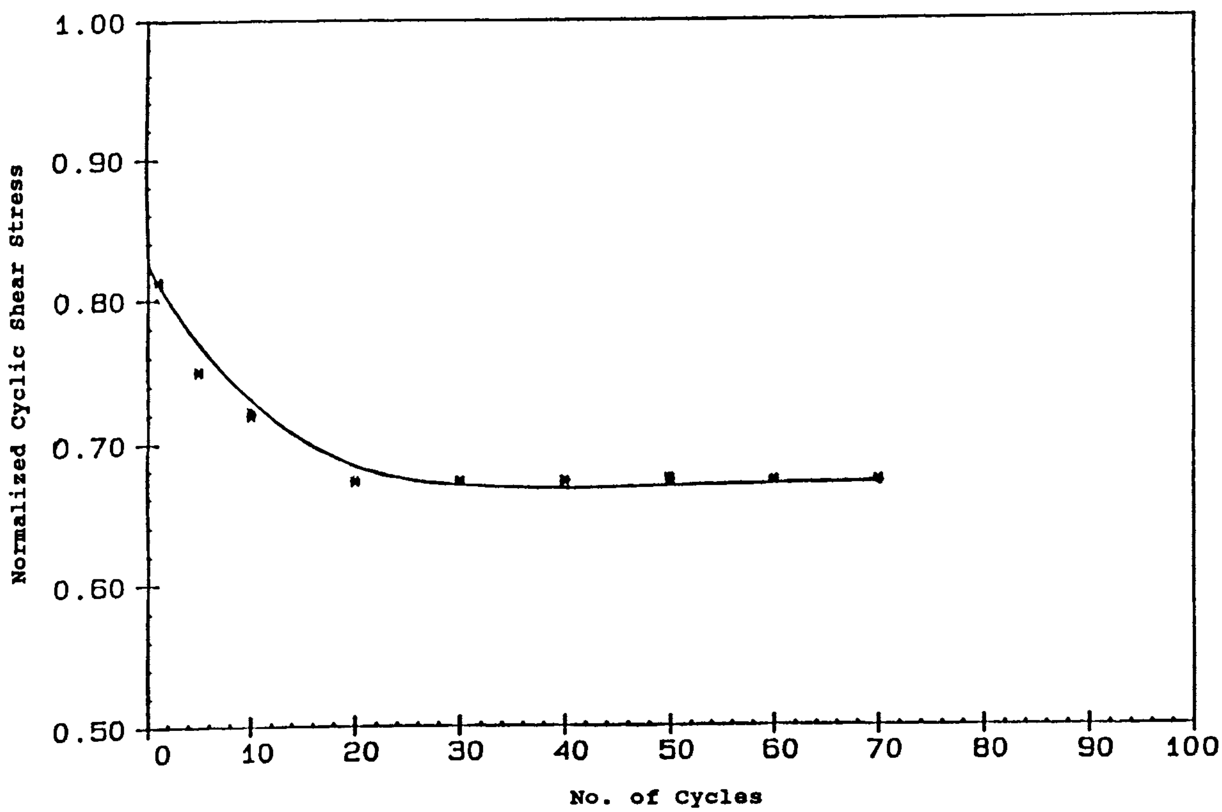

Figure 10b. Normalized Cyclic Shear Stress vs. No. of Cycles.

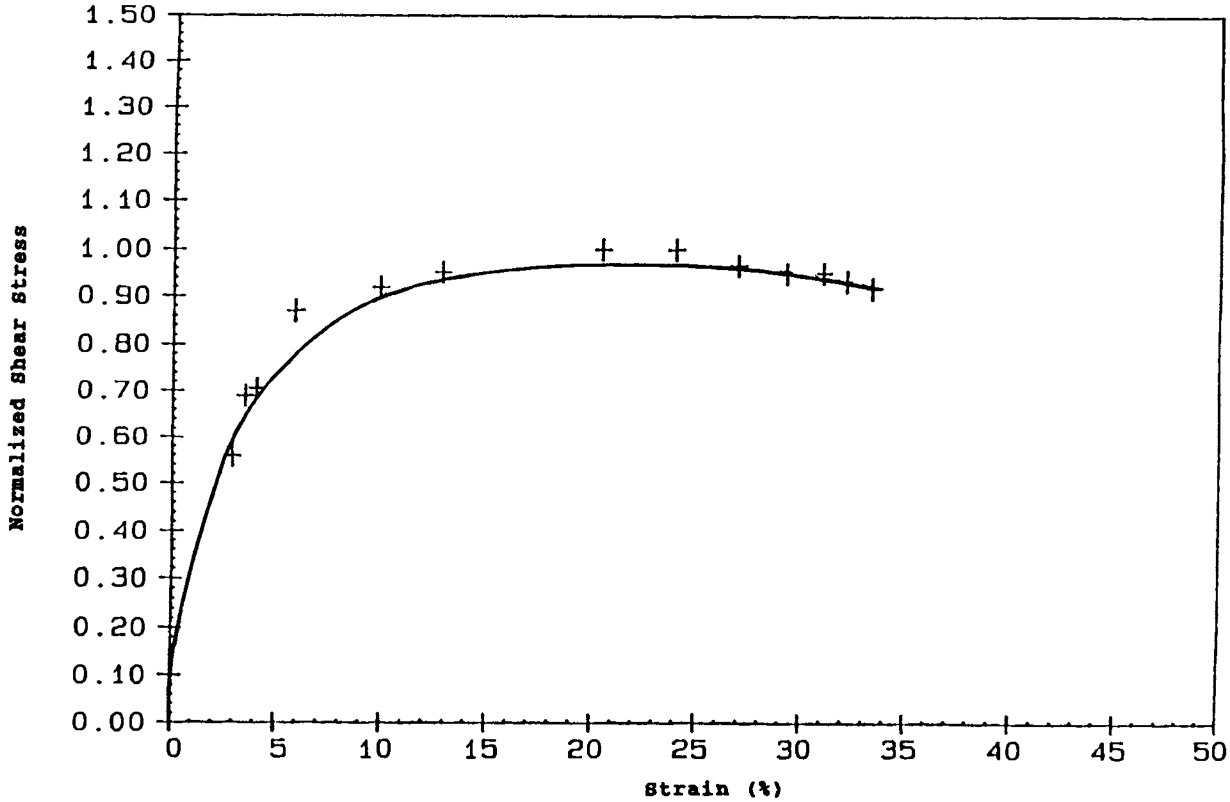

Figure 11. Typical Post-earthquake Static Direct Shear Test.