DIRECT SHEAR TESTING

Prepared by Dr. Roy E. Olson on Fall 1989

Modified by Jiunnren Lai on Spring 2004

INTRODUCTION

The direct shear device is used to determine failure envelopes for soils. The device is not suitable for determination of stress-strain properties of soils. Various aspects of the device and resulting data have been discussed previously and will not be repeated.

Most of this chapter will be written with the assumption that the tests are of the fully drained type. A later section will cover undrained testing.

STANDARD SPECIFICATIONS

Direct shear testing is covered in ASTM standard D-3080, "Standard Method for Direct Shear Test on Soils under Consolidated Drained Conditions".

GENERAL CONFIGURATION OF THE DIRECT SHEAR DEVICE

A cross section through a typical direct shear device is shown in Fig. 1. The soil sample has a porous stone at the top and bottom to allow free drainage (impervious plates could be used for "undrained" tests). Above the upper stone is a metal loading cap which is, in turn, subjected to the normal force. The sample and rings are mounted in a tank which is typically filled with water, but will be empty if tests are being performed on "unsoaked" soil. Several device configurations are in use but, typically, the shearing force is applied to the outside of the tank, which is on rollers (Fig. 1). The tank transmits the load to the lower porous stone and hence to the lower part of the sample. Load is then transferred through the shearing surface to the top part of the system and hence to a load measuring device.

Most laboratory direct shear tests are performed on samples with widths of the order of two to three inches. However, samples up to a meter in width have been used for soils containing gravel-sized particles (Nonveiller, 1954; Schultze, 1957; Bishop, __).

OVERALL REVIEW OF TESTING PROCEDURE

Introduction

The overall testing procedure will be reviewed first, without covering details, and then each aspect of the test will be considered separately. It is assumed that the test is to be fully drained and the sample is undisturbed and cohesive, and is in a sampling tube. Minor modifications cover other cases.

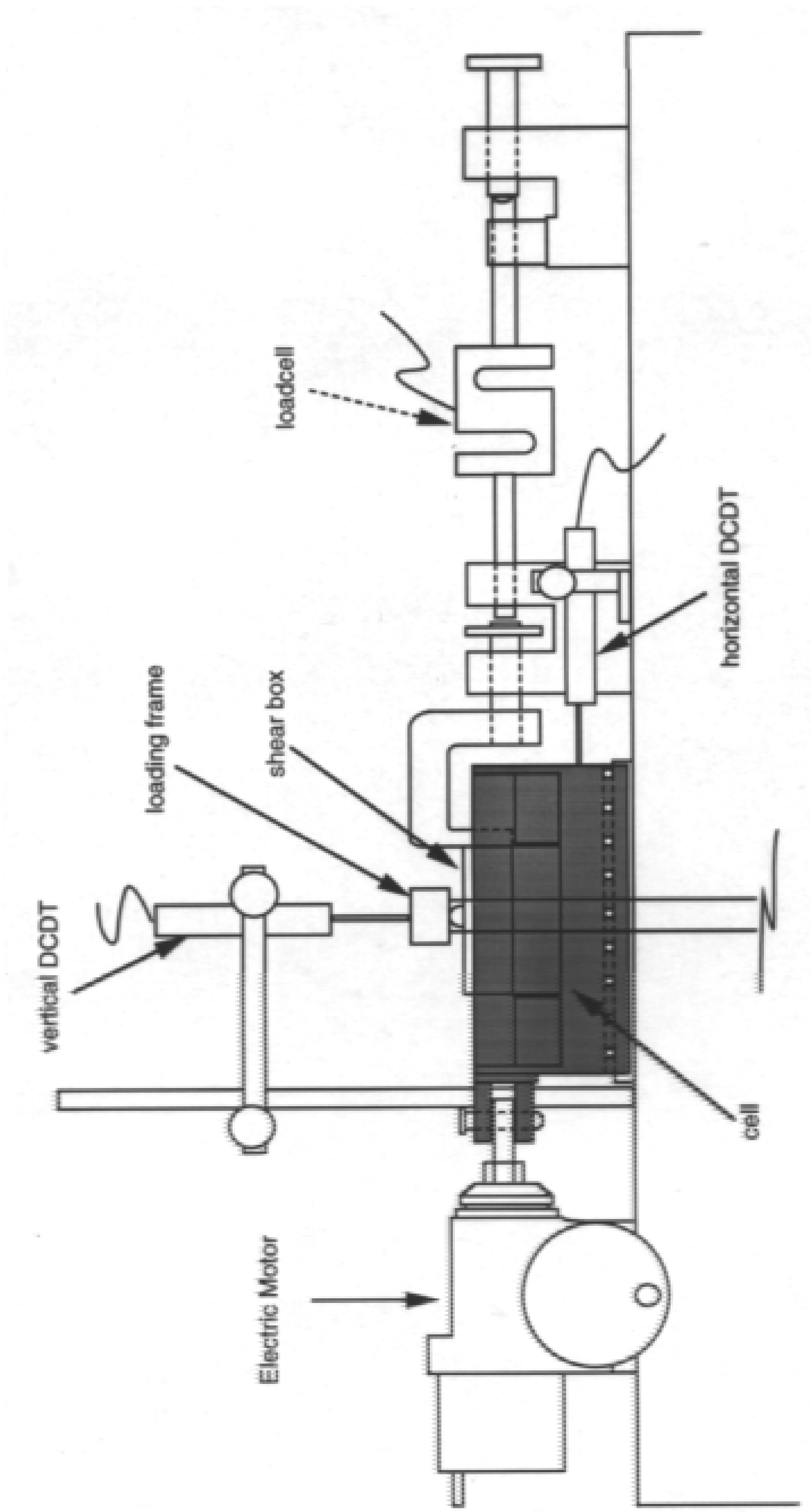

Fig. 1 Schematic Drawing of Direct Shear Apparatus

Trimming the Sample

The soil sample is extruded from the sampling tube. The extruded sample must typically be trimmed to fit into the shear box. The soil cannot conveniently be trimmed directly into most direct shear devices because the shear box is typically too large and heavy to be handled conveniently. Instead, a special trimming ring is used. The trimming ring has a height that is standard for that laboratory. If a thinner sample is desired, then after the soil has been trimmed into the ring and one face has been trimmed, a spacer plate is used on the surface just trimmed, to push the soil up into the ring an appropriate distance, and then the other face is trimmed (Fig. 2).

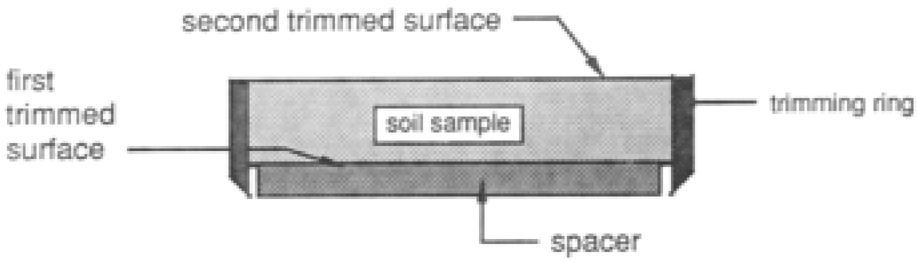

Fig. 2 Trimming a Specimen to a Height Less Than That of the Trimming Ring

The trimmings can be used to obtain an initial water content but they tend to dry out so fast that such water contents usually turn out to be significantly too low. It is better to weigh the soil in the trimming ring, subtract out the known weight of the ring, and then dry the sample after the test, being sure not to lose any of the sample.

Apparatus Assembly

The shear box is then assembled with the top and the bottom halves of the box screwed (or otherwise rigidly attached) together. The inside of the shear box is typically lightly greased to minimize side friction, just as for consolidation tests. The lower porous stone is placed in the shear box. Sometimes spacer disks are placed below this stone to adjust the elevation of its top to accommodate soil samples of different thicknesses. The trimming ring is then carefully aligned with the top of the shear box. Sometimes the trimming ring and top shear box have been machined so the ring fits into the shallow slot in the top of the shear box, to provide proper alignment. The sample is then slowly extruded into the shear box by pressing on its top surface, typically using the top porous stone or a suitable disk. The upper porous stone and loading cap are placed in the shear box, and the system to apply the normal-toad is brought into place and a small normal load (seating load) is applied.

Consolidation Stage

A dial indicator, or other suitable device for measuring the change in thickness of the sample, is quickly mounted and a zero reading taken. A consolidation pressure is then added to the top of the sample using the load-application system of the apparatus (typically a lever arm or a pneumatic system). The consolidation stage proceeds as for a standard incremental one-dimensional consolidation test. Loads are typically applied with a load increment ratio of one, to minimize problems with soil extrusion. Readings of settlement or expansion are taken as a function of time to allow appropriate calculation of consolidation coefficients and to ensure that the sample has come to equilibrium prior to the start of shear.

Preparation for the Shearing Stage

During the consolidation stage, the upper and lower halves of the shear box have been tightly screwed together (Fig. 3) to prevent the soil from extruding out from between the boxes. Typically, only two locking screws are used. Prior to shearing the sample, the upper half of the box is typically raised to provide a small separation between the boxes and ensure that the shearing and normal stresses are actually transmitted through the soil rather than from box to box. The boxes are usually separated before the final shearing stage by removing the locking screws, and then using screws that are threaded through the top box but not the bottom box, to lift the upper box (Fig. 3).

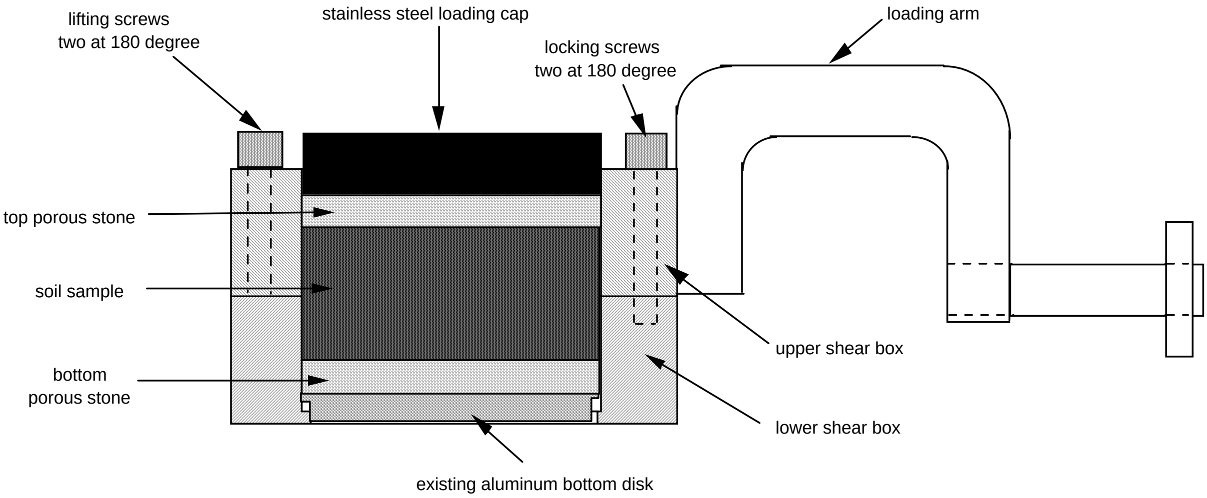

Fig. 3 Assembly Drawing of Direct Shear Box

For normally consolidated samples, of the order of 5-8% of the applied load has been transferred into side shear in the, upper half of the box so lifting the box momentarily reverses the side shear and causes a small amount of sample disturbance, thus causing a small amount of additional time-dependent consolidation. Prior to starting the shearing stage, the screws used to lift the top box must be withdrawn so the full applied stress plus the weight of the upper half of the shear box, acts on the soil in the potential failure zone, and another stage of a small amount of consolidation begins. If the top half of the shear box is heavy, and not counterbalanced, and the sample is soft, a significant amount of additional consolidation may result. To minimize the time during this stage of the test, the top half of the box is usually raised and then it is released at once; and then readings of sample thickness are continued until the sample has come back to equilibrium.

Shearing Stage

The shearing stage is usually performed at a constant rate of deformation. Methods of selecting the deformation rate will be discussed subsequently. A rate is selected and the shearing stage begin. Readings are taken of horizontal displacement, vertical movement of the top cap, and shearing force, as a function of time. Stress conditions in the sample become increasingly uncertain as deformation continues so the test is usually stopped at a horizontal deflection of about 0.25 inch even if the shearing stress has not reached a peak value.

Dismantling Stage

When the test is over, the shearing stress is reduced to zero. Equipment for measuring deformations is removed. The normal load is then reduced to zero as quickly as possible and the apparatus dismantled. The soil sample starts to rebound as soon as the normal load begins to decrease so the dismantling stage must be quite rapid if there is any desire to measure the water content at the failure stage. Once the apparatus has been dismantled, the two halves of the shear box are separated. Often, a water content sample is taken from the shear zone, and then the water content sample and the rest of the specimen are dried to obtain a final dry weight (and thus an initial water content).

Data Reduction

The data are reduced by calculating the normal and shearing stresses, plotting a curve of shearing force vs. horizontal movement, and perhaps plotting change in sample thickness vs horizontal movement. The failure condition is plotted in a Coulomb diagram.

There are numerous options in the above procedure, e.g., it may be desired to determine the residual strength as well as, or in place of, the peak strength. These options will be discussed in later sections.

SAMPLE SHAPE AND SIZE

Direct shear samples are shaped like flat disks or flat rectangular prisms. The size of the sample is partially controlled by the grain size of the soil. For clays, silts, and sands, the sample width is typically two to four inches. For soils with larger particle sizes, the width increases with widths of six to twelve inches usual and widths up to one meter in use.

For testing undisturbed samples, the size and shape tend to be controlled by sampling considerations. For samples taken with thin-walled tube samplers, the direct shear sample is typically circular and either has the same diameter as the inside of the sampling tube, or, preferably, is smaller by an amount that allows for trimming away the disturbed outer zone. When hand carved samples are used, the size and shape are more at the option of the engineer.

In direct shear testing, the desire is that the sample be subjected to shearing stress on its upper and lower surfaces, with the rings used only for lateral support, not for application of shearing force. Such a loading is promoted by using a sample that is significantly wider than it is thick, thus minimizing edge effects and promoting the effects of shear on the flat faces. A width of at least 2.5 times the thickness is usually used, but greater ratios are desired when convenient. In addition, load transfer on the faces is promoted if the porous stones at the top and bottom can grip the soil. The stones may have small grooves cut into them to promote load transfer. More commonly, a thin metal plate, called a *gripper* is placed between the stones and the sample. The metal plate is grooved and has holes cut in it to allow water to flow into the stones. Such metal plates inevitably impede drainage. It is usually assumed that the soil is disturbed to a depth at least twice the depth of the grooves so the sample thickness after consolidation is set to at least four times the groove height plus perhaps 1/8 inch.

In testing clays and silts, the friction angle between the soil and a normal porous stone should be more than the friction angle of soil on soil. Thus, some engineers believe that no special gripping mechanism is required. In direct shear testing, no filter paper is used between the stone and the soil because it would provide a plane of weakness. Special care must therefore be taken to keep the porous stones clean so they will provide adequate drainage.

If the width-to-thickness ratio is large enough, there is minimal need for any rings to support the sample. Casagrande and Hirschfeld (1960) performed tests on 6.3-inch diameter by about 0.1-inch thick, remolded samples of Boston Blue clay by just smearing the soil between two porous stones, applying a normal stress between the stones, and shearing the sample to failure, all with no supporting rings. They found the same friction angle as for regular direct shear tests (31.9 degrees), but his special tests involved a great reduction in testing time because of the small drainage distance. Such a procedure cannot be used with "undisturbed" samples but these data do indicate the benefits of using as thin a specimen as possible.

The main use of direct shear tests in our laboratory is for drained tests on undisturbed samples of stiff clay. Samples are taken using 3-inch diameter thin-walled tubes and are trimmed to 2.5-inch diameter for the direct shear tests. Sample thicknesses are of the order of 0.5 inch. If gripper plates are used, they have groove heights of only 0.05 inch.

LOADING MECHANISM

The normal load is typically applied through a simple hanger system. For typical applications, the maximum normal stresses in the field are likely to be no more than about 6000 psf. For a 2.5-inch diameter sample, this normal stress can be achieved with a dead load of only 205 pounds. For larger-sized samples or higher stress levels, a lever-arm system can be used provided that the top of the sample remains essentially stationary during the test. The lever-arm systems are usually fixed to the main support frame for the device so movement of the top cap leads to development of undefined lateral loads being applied to the top cap by the lever-arm system. Hydraulic or pneumatic loading systems are also in use. They are convenient when the applied loads are computer controlled but their use complicates the apparatus and increases its cost.

The lateral load is typically applied using a screw jack which is activated using an electric motor and a variable-speed transmission. The transmission is usually geared to apply about 0.25 inch (typical deformation to cause failure) in a time ranging from five minutes to a week. The load transmitted to the top of the sample is measured using a load cell which may be as simple as a proving ring but preferably is an electronic load cell. The electronic load cells compress less than do proving rings and also provide a signal that can be hand recorded, or can be recorded in a data acquisition system for automatic data reduction.

The cost of the apparatus is reduced greatly if a constant-load system is used. Weights are simply hung on a platform and readings of horizontal deformation are taken until it is clear that essentially all pore pressure dissipation under that shearing force has taken place and then additional loading is applied until failure results. The initial shearing load is usually elected as 10% - 25% of the estimated failure load, depending on the number of points desired for the load-deformation curve. As the failure load is approached, the size of the load increment is reduced. Although the cost of the apparatus is much less than for the jack system, the testing time is more uncertain. If the engineer makes an inaccurate estimate of the failure load, the sample may fail prematurely under undrained conditions (for the last load increment), or a series of excessively small loads may be applied, leading to unreasonably long testing times.

AREA CORRECTIONS

As the test progresses, the area of soil-to-soil contact diminishes. For a square sample with horizontal dimensions of BxB, subjected to a horizontal displacement of Δh, the actual area of contact is $B(B-\Delta h)$. The corrected area, A, can be expressed in terms of the original area, A_0=BxB, using an area correction factor, F:

$$A = A_0 F \tag{18.1}$$

where:

$$F = 1 - \frac{\Delta h}{B} \tag{18.2}$$

For a circular sample, displacement leads to contact through the hatched area shown in Fig. 4. For a sample of diameter D, the original area is given by:

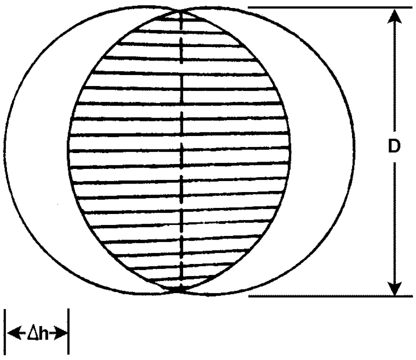

Fig. 4 Contact Area of the Sample

$$A_0 = \frac{\pi}{4} D^2 \tag{18.3}$$

The correction factor for area becomes:

$$F = \frac{2}{\pi}\left\{\cos^{-1}\left(\frac{\Delta h}{D}\right) - \left(\frac{\Delta h}{D}\right)\sqrt{1-\left(\frac{\Delta h}{D}\right)^2}\right\} \tag{18.4}$$

where the arc-cosine is in radians. Numerical values of the correction factor are as follows:

$\Delta h/D$	0	0.02	0.04	0.06	0.08	0.10	0.12	0.14	0.16	0.18	0.20
F	1.000	0.975	0.949	0.924	0.898	0.873	0.848	0.822	0.797	0.772	0.747

The normal force is typically constant during a direct shear test so the reduction in area leads to a gradual increase in normal stress. The shearing force is found by dividing the applied force by the corrected area A. The stress path, then, is not a vertical line but is actually a line that curves to the right, as shown in Fig. 5. In drained direct shear tests, the failure envelope often has only a small intercept. If the envelope goes through the origin, the area correction just moves the failure point out along the same failure envelope. Because there will then be no change in the friction angle, many engineers simply ignore the area correction. If the direct shear device were to be used for a Q test on a saturated soil, where the failure envelope was horizontal, ignoring F would lead to an apparent shearing strength equal to the actual strength times F.

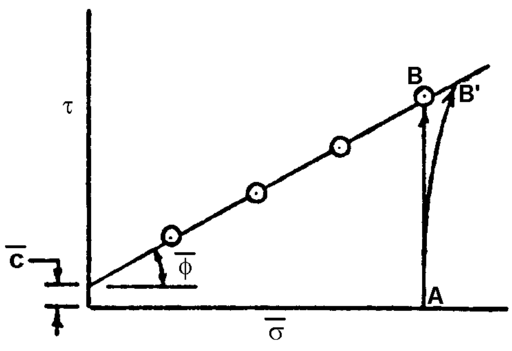

Fig. 5 Stress Path from Direct Shear Tests

Once horizontal deformation begins, the soil-on-soil area begins to reduce, but a soil-on-shear-box contact begins at both the leading and trailing edges of the sample. To minimize

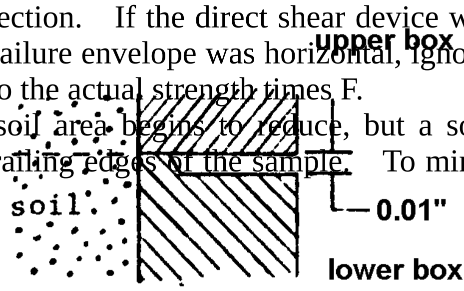

Fig. 6 Cross Section Through the Edge of a Direct Shear Box

the shearing stress between the soil and the opposing box, the surface of the box is lightly greased, usually with a silicone grease, and the surface of the box may be relieved by about 0.01 inch except near the inside edge (Fig. 6).

Horizontal displacements also lead to problems with the stress state on the sample. At the beginning of the shearing stage, the normal stresses on the vertical and horizontal surfaces of the sample are assumed to be uniform, with no "shear on these boundaries. The resultant forces all pass through the center of the sample and the sample is clearly in equilibrium. As soon as a horizontal displacement occurs, the forces on the top and bottom surfaces tend to move out of alignment (Fig. 8), producing an overturning moment that cannot exist in the real test (if it did, the sample would begin rotating in space). It is probable that the normal stresses on the horizontal surfaces of the sample become nonuniform in such a way that the resultant forces on the top and bottom remain equal and have the same line of action. The changing normal stresses should cause an extraneous s set of water pressure gradients in the sample.

As soon as a shearing stress is applied to the horizontal faces of the specimen, there must also be shearing stresses developed at the vertical boundary (Fig. 7). However, the inside of the box was greased to prevent any shearing stresses at these boundaries during the consolidation stage, and this lubrication presumably carries over to the shearing stage. If no shearing stresses can exist at the vertical soil/box boundary, then none can exist on the horizontal surface either, and consequently the shearing stresses must also be non-uniform on the horizontal faces.

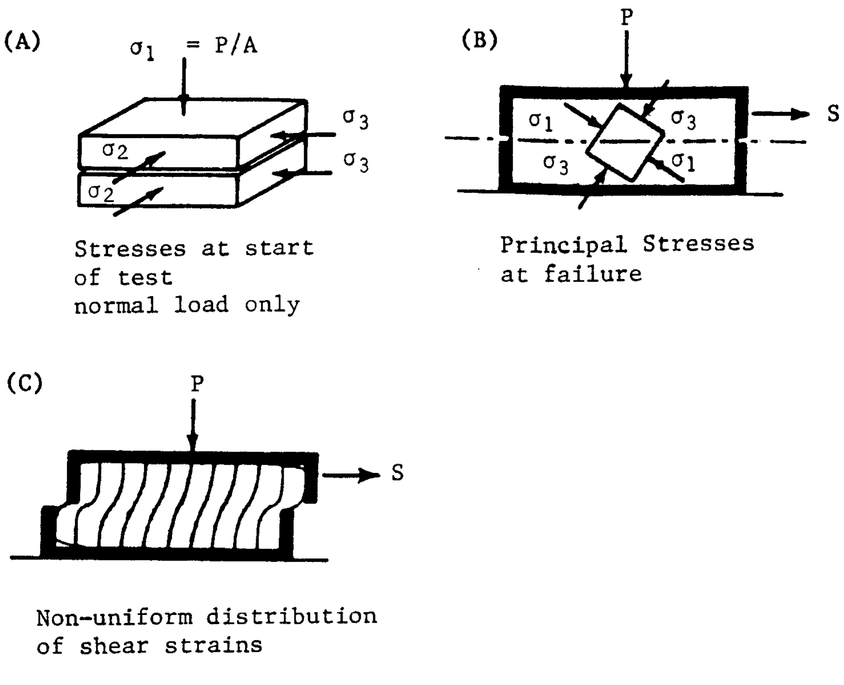

Fig. 7 Stress and Shearing Strain That Develop in the Direct Shear Test (Sowers, 1964)

RATES OF DEFORMATION

When constant-rate-of-deformation tests are performed, the engineer must select an appropriate rate of deformation. The selection may be based on theory or experience. The theoretical approach will be reviewed first.

A theoretical solution was developed by Gibson and Henkel (1954). They assumed that shearing a sample to failure under undrained conditions would lead to a uniform rate of increase in pore water pressure throughout the sample up to the moment of failure (Fig. 8), after which the pore water pressure would remain constant. For a "drained" test, they assumed that any increment of deformation produces the same increase in pore water pressure as did the same increment in an undrained test, but that the pore water pressures now dissipate in accord with Terzaghi's theory of one-dimensional consolidation. The solution is an infinite Fourier series. It is convenient to define a degree of consolidation at failure, U_f, using:

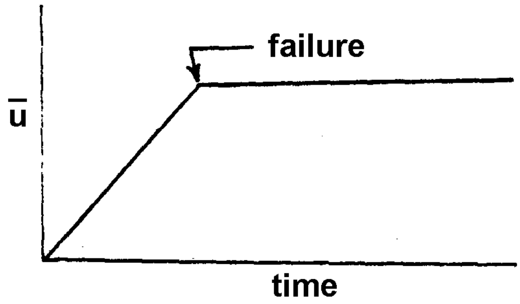

Fig. 8 Assumed Curve of Pore Pressure vs. Shearing Time

$$U_f = 1 - \frac{\bar{u}_f}{\bar{u}_0} \tag{18.5}$$

where u_f is the excess pore water pressure at failure and u_0 is the excess pore water pressure that would have existed in the absence of any drainage. In a fully drained test, u_f would be zero and U_f would then be 1.0. The infinite series solution of U_f can be truncated after the first term for large values of U_f (say over 0.8) to yield a simple solution:

$$U_f = 1 - \frac{H_S^2}{2c_v t_f} \tag{18.6}$$

where H_S is the average drainage distance during shear (sub-s denotes the shearing stage; for the case of top and bottom drainage, H_S is half of the thickness of the specimen), c_V is the coefficient of consolidation, and t t_f is the time to failure. Equation 18.6 can be rearranged to yield:

$$t_f = \frac{H_S^2}{2c_v(1-U_f)} \tag{18.7}$$

Note that the time to failure is infinite if U_f is set equal to one.

In principle, the coefficient of consolidation, c_v, can be evaluated for the last loading increment following the usual methods based on Terzaghi's theory. If the curve fitting is performed at 50% consolidation, then:

$$c_V = \frac{0.197 H_C^2}{t_{50}} \tag{18.8}$$

where H_C is the average drainage distance (half the sample thickness) during the consolidation stage, and t_{50} is the time to achieve 50% consolidation. Insertion of Eq. 18.7 into Eq. 18.8 leads to:

$$t_f = \frac{H_S^2}{2 \cdot 0.197 \cdot H_C^2 (1 - U_f)} \cdot t_{50} \tag{18.9}$$

Gibson and Henkel (1954) noted that for normally consolidated clays, the undrained strength is about half of the drained strength. Thus, as the time to failure is increased from a very short value, the measured partially drained shearing strength should essentially double and should level out at the fully drained strength. Experimental data from their study are replotted in Fig. 9. The measured shearing strength seems to peak at a degree of consolidation of about 95% and thereafter decrease. The decrease in strength is apparently due to viscosity of the soil structure under drained conditions. If the peak strength is used as a measure of the fully drained strength, then a reasonable value of U_f is 0.95 but strengths are not likely to be significantly different for values of U_f over about 90%.

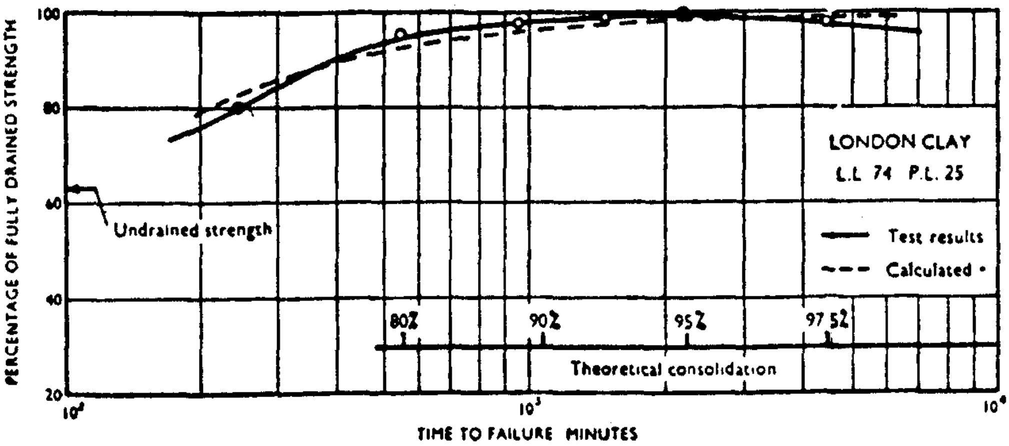

Fig. 9 Shearing Strengths from Direct Shear Tests on London Clay using a Range in Times to Failure (Gibson and Henkel, 1954)

Equation 18.9 can be simplified by assuming that the drainage distances during consolidation and shear are essentially identical, and that satisfactory results can be achieved using U_f=0.95. Thus:

$$t_f = 50\ t_{50} \tag{18.10}$$

Highly overconsolidated clays have undrained strengths exceeding the drained strength so plots of measured strength versus time to failure involve a steady downwards trend and no conclusions can be drawn from tests at different strain rates regarding the proper testing time. Further, for highly overconsolidated soils, time-settlement curve for the last loading (unloading) stage typically cannot be interpreted to yield useful coefficients of consolidation. For such soils, the approach using theory is to, measure the hydraulic conductivity (k), measure the coefficient of compressibility (a_V) for the final load, and calculate c_V. An alternative, and more commonly used, approach is to select a time to failure based on experience. For samples with a thickness of the order of 0.25 to 0.5 inch, we typically use times to failure ranging from 1 to four days.

RESIDUAL STRENGTHS

The direct shear device is well suited for measurement of the residual strengths for stiff clays. Several options exist. In the first, the first stage shearing is carried out to a reasonable limit, say 0.25 inch of horizontal movement. Then the direction of shearing is reversed and carried to a total deflection of -0.25 inch. This process is repeated until the measured strength remains essentially constant for several cycles. Typically, reversal of shear causes a sudden increase in the shearing stress and then a decrease back to a relatively steady value, even after numerous cycles of loading. The residual strength is taken as that steady value.

An alternative approach that saves a great deal of time, is to precut a failure surface at the position corresponding to the contact between the two halves of the shear box, and run one or two cycles on it. The surface may be smoothed using a wide putty knife, or other suitable tool, to get it as smooth as practicable prior to be start of the test.

In some cases, a specimen can be trimmed from a slickensided sample in such a way that the natural shear surface is aligned with the shear surface in the shear box and direct shear tests can be performed on the actual slickensided surface.

UNDRAINED TESTS

An undrained test is one in which the water content in the failure zone does not change during shear. Direct shear tests on clays can be performed fast enough that only relatively small volume changes occur in the sample as a whole. However, shearing stresses, and thus the excess pore water pressures, tend to be concentrated in the shearing zone, and thus not to pervade the entire sample thickness. Water may migrate out of the shear zone and into the rest of the sample, or vice versa, during shear even though the overall change in average water content of the sample is apparently negligible. Such a test is not truly undrained. The same problem exists in the case of triaxial compression tests when failure occurs on a single plane.

An alternative approach is to adjust the axial load to maintain a constant sample thickness during shear. If carried out with sufficient precision, such an approach may maintain the overall water content constant but I does not ensure that water migration will not occur within the sample.

Attempts to seal the sample by using impervious end caps, similarly does not prevent internal migration of moisture.

The direct shear device, by its very nature, is not suitable for undrained tests and should not be used for that type of strength testing.

ACCURACY AND REPRODUCIBILITY

We have used drained direct shear tests to estimate shearing properties of clay shales for slope stability analyses, for several decades, and find that the accuracy of field conditions is apparently controlled by factors unrelated to details of the laboratory test, especially to prediction of pore water pressures in the field, to accounting for effects of non-homogeneous soils, and for analytical methods. The direct shear apparatus itself thus seems to give results with more accuracy than we actually need.

On the other hand, there have been a few studies in which direct shear failure envelopes were compared with envelopes from triaxial or other types of shear tests, or the results from direct shear tests in one laboratory are compared with data from another laboratory.

One such study was performed in 1938-39 and reported by Converse (1952). Samples of dry Ottawa sand, in the size range between the #20 and #30 sieves, were sent to seven laboratories (Cal. Tech., Columbia, Michigan, Princeton, U.S.Army Engr. labs at Fort Peck and Ithaca, and a U.S.Dept. of Interior lab), with direct shear tests to be performed on the sand in loose and dense conditions. Some of the laboratories performed only direct shear tests; others performed direct shear and double ring shear tests.

Direct shear tests yielded the following results:

condition	number of labs	$\bar{\phi}$	range in $\bar{\phi}$
dense	__	33	24 to 50
loose	__	28	23 to 36

The double-ring shear tests yielded 55 deg. for the loose sand (one value) and 38 degrees (range from 27 to 44 degrees) for the dense sand.

Converse (1952) also summarized his interpretation of data collected by the U.S. Army Corps of Engineers on samples of cohesive soils from the West Atchafalaya FIoodway project. His data were as shown below:

Soil Description	Direct Shear		Triaxial Shear	
	$\bar{c}$ (psf)	$\bar{\phi}$ (deg.)	$\bar{c}$ (psf)	$\bar{\phi}$ (deg.)
silty clay	440	14	500	14
clay	480	17	510	17
clay	730	16	730	18
clay	600	16	400	19
silty clay/clay	700	7	600	8
silty clay/clay	920	7	470	8
silty clay/clay	700	12	400	8

The direct shear and triaxial shear data seem reasonably comparable but the low values of $\bar{\phi}$ from some of the tests make it appear that full drainage was not achieved.

A more recent study was carried out in Southern California (Lee and Singh, 1968). Samples of a clean sand and a silty clay were sent to thirty cooperating laboratories and data were received back from seventeen of them. Shear tests included direct shear, double ring shear, and triaxial shear.

The sand was clean Ottawa sand in the size range between the #20 and #50 sieves. The minimum density was 94 pcf (one laboratory) and the maximum density was 105 or 113 pcf (two laboratories). Laboratories were asked to perform tests on the sand in a "loose" and a "dense" condition. Values of the secant $\bar{\phi}$ are plotted against the measured dry density in Fig. 11. The failure envelopes were apparently much better defined using triaxial shear data but it is probable that the triaxial data came from a single laboratory.

For the Lee and Singh study, several laboratories used several technicians to perform supposedly identical tests. The results are summarized below:

Laboratory	Technician	Loose	Dense

X	A	31	28
	B	28	34
	C	36	40
Y	A	33	39
	B	31	40
	C	33	39

The results are inconsistent, one technician actually obtaining a lower friction angle for dense sand than for loose sand.

One laboratory tried to determine if the spacing of the boxes during shear was a factor contributing to the scatter. The basis for this concern was that sand grains could be heard fracturing between the shear boxes during shear. Results of tests on duplicate tests on a "silica sand" at a void ratio of 0.75, and under a normal stress of 1500 psf, are shown below:

Box Spacing (inch)	$\bar{\phi}$ (degrees)
0.025	39
0.035	40
0.045	39
0.066	38
0.126	36

Several laboratories regularly performed double ring shear tests and chose to make comparative runs for direct shear and double ring shear. The results are shown below:

Laboratory	Relative Density	Direct Shear (degrees)	Double Ring Shear (degrees)
A	loose	35	25
B		29	28
C		33	31
A	dense	42	32
B		34	33
C		36	33

Results for the silty clay were inconsistent, with $\bar{\phi}$ ranging from 25 to 55 degrees, and the cohesion intercept from 0 to 1500 psf. The data were difficult to interpret because of a number of extraneous effects and will not be considered further here.

The above data are inconsistent but some of the following inferences seem appropriate:

1. Even for dry sand, the range in friction angles obtained by different laboratories was unacceptably large. A knowledgeable geotechnical engineer could probably have guessed values of $\bar{\phi}$ with an accuracy similar to that achieved by most of these laboratories using tests.

2. Results for sand may have been influenced by grains being trapped between the boxes and being fractured. The triaxial shear device is likely to be a more useful device for testing sands.

3. There is no consistent trend in comparisons of $\bar{\phi}$ values obtained using direct shear and

double ring shear tests. In the Lee and Singh study, the narrow range in $\bar{\phi}$ values for loose and dense sands may indicate progressive failure in the double ring shear tests, leading to both loose and dense sands tending toward a residual $\bar{\phi}$.

4. Within any one laboratory, technicians should periodically perform tests using a standard soil, and compare results among themselves to ensure that they are using consistent testing procedures.

5. Comparative tests on standard soils should be made between different machines in any one laboratory, and between different laboratories in a community, to ensure that the measured soil properties are consistent

REFERENCES

Bishop, A. W. (find reference on large direct shear tests

Converse, F. J. (1952), "The Use of the Direct Shear Testing Machine in Foundation Engineering Practice," Symposium on Direct Shear Testing of Soils, ASTM, Vol. 131, pp. 75-80.

Gibson, R. E., and D. J. Henkel (1954), "Influence of Duration of Tests at ConstahTTlate orStrain on Measured 'Drained' Strength", Geotechnique, Vol. 4, No. 1, pp. 6-15.

Hirschfeld - ref where he smeared soil on stones, should be in a Harvard soil mechanics series

Hvorslev - find reference on effects of groove height on disturbance Lee, K. L, and A. Singh (1968), "Report on the Direct Shear Comparative Study", unpublished report.

REFS - on use of very large shear devices, up to one mete