Estimation of Effective Strength Parameters of Well-Behaved Clay

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Many testing techniques and interpretative methods are currently used to determine the strength parameters of saturated clays. The physical circumstances under which clays are likely to be well-behaved are described in this paper. A well-documented procedure for determining the effective stress-strength parameters of such materials is recommended. The direct shear and triaxial compression tests are examined. Other tests or modifications can be conducted, but these are not in common use (e.g., triaxial extension). The point is made that soils often exhibit different properties, depending on the type of testing (e.g., plane strain or radial symmetry).

Three main types of shear tests are used to determine the effective strength parameters of saturated clay. These tests are as follows:

1. Drained direct shear test,
2. Drained triaxial compression test, and
3. Undrained triaxial compression test with pore water pressure measurements.

Other tests or modifications of these tests (i.e., triaxial extension) may be performed but are not in common use and are not covered here. Note that soils often exhibit different properties, depending on the type of testing (e.g., plane strain or radial symmetry). In general, refinements in testing soils are only performed, if at all, on major jobs where savings may be realized despite the additional costs of the testing techniques.

TYPE OF SOIL

The procedures outlined are for soils that do not exhibit large creep effects during testing. The rate of strain should therefore be minimal and the sensitivity and strain softening characteristics should be minor. The soil is assumed to be saturated, to have an unconfined compressive strength of less than 150 kPa (3,000 lb/ft^2), sensitivity less than 8, and an overconsolidation ratio of less than 4.

APPARATUS

Shear Box

The constant rate of strain direct shear box apparatus is shown in Figure 1. The apparatus consists of a square or circular box, generally of brass, approximately one-third its width deep with an open top and bottom. The length depends on the sample size, but it is normally 50-75 mm (2-3 in.). The box is divided horizontally into two halves, a top and a bottom. The two halves can be fitted together by two alignment pins (or screws) in opposite corners that pass vertically through the walls of the upper half and screw into the lower half of the box. In the other two corners are screws that screw into the upper half of the box to rest on the lower half of the box. These latter screws are used once the box has been set up, when the sample in the box is ready for shearing, to raise the upper half slightly and thus reduce friction between the upper and lower parts of the box.

The box is placed in a rectangular brass container designed so that the bottom half of the shear box is held rigidly in position. For the strain-controlled model the container rides on frictionless bearings and is restricted to move in a longitudinal direction only. The sample is sheared, generally at a constant rate of strain, although a constant rate of loading may be used. In the case of a constant rate of strain test, a geared jack driven by an electric motor is normally used. The jack pushes against the container and hence the bottom of the shear box. This movement is transmitted through the sample under test to the upper half of the box, which is connected to a force transducer or proving ring.

Two porous square stones, which fit easily into the shear box, are used above and below the sample. The stones come in various thicknesses, depending on the depth of the box. The total thickness is such that the depth of soil sample is about 20 mm (0.8 in.).

The load to shear of the sample is measured by a proving ring complete with dial gage or by means of a force transducer. Whichever method is used, the force measured is that transmitted to the upper half of the box. The proving ring or force transducer is aligned horizontally with the horizontal shear plane through the sample.

A metal pressure pad that fits into the box on top of the upper porous stone is used to distribute the load from a hanger mechanism that fits over and above the sample normal to the shear plane. The hanger usually rests on the metal pressure pad through a ball bearing located centrally on the pad. Dead weights generally act through a lever system to apply normal loads to the sample. The lever system may be leveled by screw jacks fixed to the underside of the chassis of the apparatus. For light normal pressures the loading may be done directly through the hanger without a lever system.

A square (or circular for circular sample) cutting ring of inside dimensions the same as the inside dimensions of the box and 20-mm (0.8-in.) thick with a 10-degree (or less) sharpened cutting edge is used to obtain the test specimen. A wooden dolly is often used to retrieve the soil specimen from the cutting ring. The wooden dolly will be the same dimensions minus clearances so that it just fits inside the cutting ring.

Figure 1. Schematic of direct shear box.

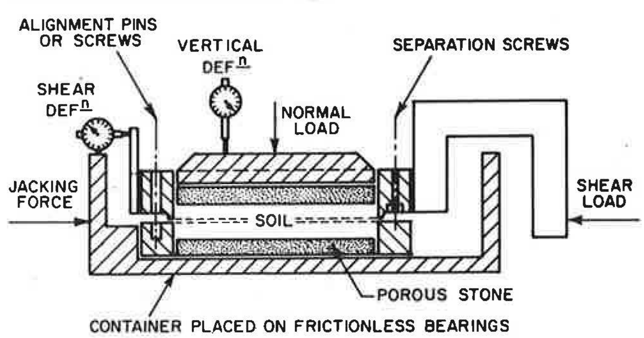


Figure 2. Schematic of triaxial cell.

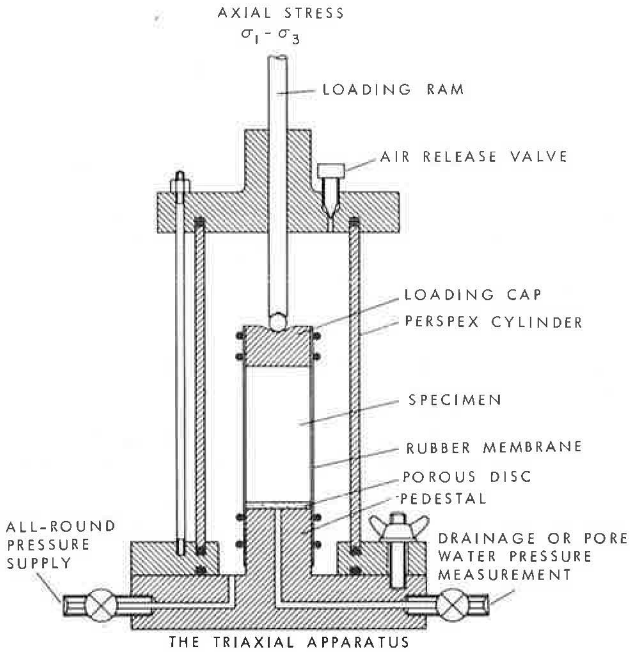
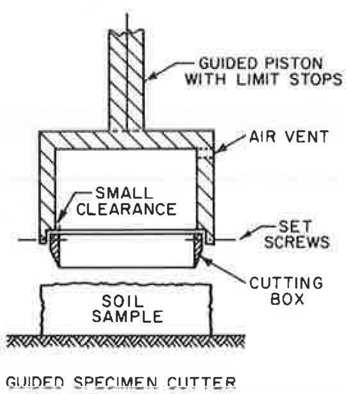


Figure 3. Fine-grained soil specimen cutter for direct shear box.



Triaxial Compression Equipment

A constant rate-of-strain compression machine is normally used to perform the triaxial test. Alternatively, the test may be performed by using incremental loads. The constant rate-of-strain compression machine can be fitted with either a proving ring or force transducer to measure the vertical load. The machine is normally operated at a constant rate through a geared screw jack that is driven by a small electric motor.

A hydraulic or air-operated pressure apparatus is used to apply fluid pressure to the outside of the sample membrane. For short-term tests air is often used acting on water in a water-air reservoir. The cell is nearly always filled with water. In long-term tests, due to the possibility of air permeating through the rubber membrane, some form of mercury constant pressure pot system is often used (1).

Figure 2 shows a typical triaxial cell. In North America a sample 50-mm (2.0-in.) in diameter is commonly used, although other sizes may be used. The length of the sample is generally twice its diameter. It should not be less than one and one-half times the diameter; otherwise end platten friction can become severe and cause major errors in the test results. It should not be more than 3 diameters long to prevent buckling of the specimen. The inner diameter of the cell is generally about 50 mm (2 in.) larger than the specimen size so that the cell can be placed over the specimen without touching and possibly disturbing the set-up. The space between the specimen and the cell is filled with water to allow an all-around hydrostatic test pressure to be applied to the specimen. A piston through the central axis of the top of the cell allows a vertical load to be applied to the specimen through a pressure cap. A rubber membrane is placed around the specimen and O-rings are used at the top and bottom of the rubber membrane to separate the cell fluid from the specimen. This allows the cell pressure to apply a load to the specimen through the rubber membrane and specimen top cap.

A trimming apparatus is usually available for soft soils, along with some form of cradle for trimming the specimen to length. The top cap for the specimen is normally made of plastic or of a light nonporous metal.

General Apparatus

In general, samples will come to the laboratory in field sampling tubes. A universal extruding machine will be required to obtain the undisturbed sample from the sampling tube. Although not common, the ability to apply a suction to the extruding end of the sample will facilitate extrusion of the soil and also minimize disturbance. Other equipment will include stopwatch, moisture content apparatus, balance, and other accessory equipment generally found in most soils laboratories.

PREPARATION OF SAMPLE

Direct Shear Test

The undisturbed soil sample cut from the sample tube should be about 25-mm (1-in.) thick and slightly larger in diameter than the diagonal distance across the shear box cutter. From this sample a specimen is cut with the guided cutting ring apparatus (see Figure 3). In some soils it may be desirable to pretrim the undisturbed sample to a size only slightly greater than the inside of the cutting ring. A wire saw is used to trim the top and bottom of the cutting ring. In some cases final trimming may be performed with a sharp steel straight edge. When the soil specimen has been trimmed to the right thickness, it is pushed out of the cutting ring by using a wooden dolly. In some cases a porous stone is placed next to the soil rather than the wooden dolly. After weighing the sample, saturated porous stones are placed above and below the specimen and the specimen is transferred to a dry shear box. Excess moisture should be dried from the stones before placement next to the soil. A certain amount of skill is needed in cutting the soil, and the use of trial-and-error methods is often required to ensure a good fit of the specimen within the box. Only samples in an undisturbed condition should be tested. Any waste soil may be used to obtain a moisture content of the in situ soil. This may then be used to calculate the bulk dry and wet density of the soil. A check may also be run on the degree of saturation if the specific gravity of the solids is known or assumed.

Triaxial Test

Before a sample is extruded from the sample tube, the triaxial cell should be flushed of all air from the water lines that lead to the specimen pore water. These lines are often known as back pressure lines. If possible the cell should be filled with water and left under pressure when not in use. This will ensure that the cell leads remain de-aired. When the triaxial cell is dismantled for testing, all pore water cocks should be closed.

A sample of undisturbed soil is extruded, generally vertically with the extruding apparatus, from the field sampling tubes. The length of sample extruded from the tubes should be slightly longer than that required for the triaxial specimen. The triaxial specimen is obtained by trimming the extruded soil sample to the correct diameter with the aid of a wire saw. Final trimming of a soft soil can be facilitated with a sharp, steel straight edge. When the required diameter has been achieved, the sample is transferred carefully to a cradle, which permits the sample to be trimmed with the wire saw to the correct length. If the sample is soft and wet, a nonabsorbent piece of paper may be wrapped around the specimen to prevent it from adhering to the cradle or to one's fingers during handling. The trimmed specimen is then weighed. After weighing, porous stones are placed on top and bottom and the specimen is placed on the pedestal of the triaxial machine. The porous stones should have been saturated with de-aired water. Excess water should be removed from the stones. De-airing can be achieved by boiling the stones in water. Either heat or vacuum may be used. Vigorous boiling is not recommended. The stones should be allowed to return to room temperature under water before use.

Filter paper strips along the sides of the specimen may be used to speed testing. The 5-mm (0.2-in.) wide filter paper strips should be saturated and excess moisture removed by squeezing so that they cling to the sample. The strips should cover the soil specimen and stones only. The top cap should now be placed in position.

A suitable rubber sleeve is inspected to detect pin holes. Only sleeves in good condition should be used. The sleeve is placed inside a former, the ends carefully pushed over the former, and the sides straightened so as to remove all wrinkles. By sucking on a rubber tube attached to the side of the former, a slight vacuum is created, which causes the rubber sleeve to cling to the walls of the former. The rubber membrane is now slipped over the specimen and, when in the correct position, the suction on the former is released. The sleeve should fit snugly around the sample without enclosing air bubbles when the vacuum is released. The rubber sleeve should then be slipped over the pedestal and top cap. O-rings, at least two at the bottom and two at the top, are used to seal the rubber sleeve to the metal pedestal and top cap.

Should the material be difficult to trim to a cylinder, a direct shear test should be performed to check the soil properties.

With the pore water cocks closed, the cell should be assembled and filled with water. A cell pressure should be applied and the specimen should be allowed to come to equilibrium, preferably overnight. The drainage cocks should remain closed. If a pore water transducer is connected to the cell base, the cell water loading pressure should be raised to maintain a positive pore water pressure in the specimen during the overnight undrained equilibrium period.

If the specimen pore water pressure rises overnight to equal the cell pressure, leakage through either the membrane or membrane-end cap seal is to be suspected. Alternatively, the sample is set up with excess water on the porous stones or filter paper. This is common when procedures that flush water up the sides of the sample are used. (These procedures are not recommended here.)

The water pressure lines that lead to the pore water of the specimen (often known as back-pressure lines) should be pressure controlled in the same manner as is the cell water.

Depending on the equipment available and its limitations, the cell pressure applied for the overnight equilibrium period should be about one-half that to be used during the shear testing, unless a higher pressure is required to ensure a positive pore water pressure.

CONSOLIDATION OF SPECIMEN

Direct Shear Test

Before water is added to the box, an initial seating load of 2-5 kPa (0.3-0.7 lb/in.^{2}) should be applied to the sample while the dial reading corresponding to the initial height of the sample is quickly obtained. While the dial reading is checked, water should be added. If the sample starts to swell, the load should be increased to prevent swelling and to obtain an initial equilibrium consolidation stress. The standard consolidation testing procedure of doubling the load (or halving the load in the case of swelling) should be conducted on the specimen for each increment until the desired test normal stress is obtained. Readings should be taken to establish 90 percent primary consolidation before the addition of the next increment.

Triaxial Test

Once the specimen has come to equilibrium and before the drainage cock is opened, the back pressure is adjusted to give an effective consolidation pressure equal to that desired for the first load increment. In triaxial testing, unless a void ratio pressure relation is desired, the initial effective consolidation pressure will be half the final effective consolidation test pressure. This allows a consolidation test to be performed, which is free of errors due to trapped water during set-up, by doubling the effective consolidation pressure, which is normal practice in a consolidation test. Time volume change readings are taken until 90 percent or more of the primary consolidation is complete for the drained test and for at least twice the time interval for 90 percent primary consolidation in the undrained test.

In the drained test the drainage cock is momentarily closed, the cell pressure is raised quickly to its test value, and a final consolidation test is started by opening the drainage cock. The value of the coefficient of consolidation obtained from this test is the value to be used for all calculations of testing rate.

In the undrained test the drainage cock and cell pressure cock (if cell leakage is negligible) are closed and the cell pressure is raised quickly to its test value. Pore pressure readings are taken by a method similar to a consolidation test when the cell pressure cock is opened (or the cell pressure is increased if cell leakage is apparent). If the change in pore pressure responds quickly and equals the increase in cell pressure, the drainage cock is opened and a final consolidation test is started. The value of the coefficient of consolidation obtained from this test is to be used for all calculations of testing rate. If the pore pressure response is sluggish or does not equal the increase in cell pressure, either large quantities of air were trapped in the pore water system or the transducer compliance is too large for the soil under test. In both cases undrained testing is not recommended and drained testing is preferred. In special circumstances, where only undrained testing is specified, an attempt to saturate the specimen fully may be undertaken by raising both the cell and back pressure equally. The back pressure should be as high as possible so as to dissolve the trapped air quickly. Note that if the back pressure is later reduced, air may come out of solution, which would defeat the original intention. If the pore pressure responds quickly to a value less than the increase in cell pressure and remains constant, the soil structure compressibility is not large in comparison with the compressibility of water or the soil solids. Discussion of this type of response is beyond the scope of the material being presented here. Such response can be expected from soft or hard rocks.

Where it is desired to measure the axial deformation during consolidation, or to ensure proper seating of the plunger on the top cap, a small axial stress difference equal to no more than 0.1 of the effective cell pressure may be used.

RATE OF TESTING

The usual criterion for the rate of drained testing of soils is based on the time to failure being the same as the time to reach 95 percent average consolidation of an oedometer constant rate-of-loading test. For this criterion to apply, the minimum time to failure is approximately 50 times the time to reach 50 percent average consolidation in a single increment loading test. This ratio is approximately the same for radial drainage; thus the speed of drained testing, whether in the shear box or triaxial cell, is obtained from the last consolidation increment. The result will naturally include any drainage imperfection because it was obtained from the specimen under test.

Similarly the rate of testing for undrained specimens is based on the setting up of a parabolic distribution of pore pressure in a constant rate-of-loading test and the achievement of 95 percent equalization. Under these conditions the minimum time to failure is approximately three times the time to 50 percent consolidation in the last increment test.

If stress-strain-pore pressure and volume change readings before failure are required, the minimum times calculated above are minimum times to the first reliable reading. Larger times to failure are sometimes desirable to facilitate the recording of readings, particularly in more permeable soils.

CRITERIA USED FOR FAILURE

The peak axial stress difference is the most common failure criterion used in soil mechanics. Also used sometimes is the effective principal stress ratio (i.e., ratio of major to minor effective principal stress). In drained tests these two criteria are the same. In undrained tests the former usually occurs first. Although the growth of modern soil mechanics is based on the principle of effective stress, engineering problems are normally load controlled and thus the former criterion is favored. Where a peak deviator stress is pronounced, for both drained and undrained tests, the test should be continued until an approximately constant post-peak stress is obtained.

PLOTTING OF TEST DATA

The consolidation test data, where applicable, should be plotted in accordance with the TRB report, Estimation of Consolidation Settlement (2), and volume reading should be replaced in the triaxial test for dial gage reading. In the triaxial test, when the axial deformation is measured during consolidation, the ratio of axial to lateral strain may be plotted to give an indication of the anisotropy of the soil and its variation with consolidation pressure. The lateral strain is calculated as half the difference of the volumetric and axial strains.

The shear test data should be reduced and the stress-strain data plotted during testing. In the triaxial test the diameter and length of the sample before shear testing should be calculated from the consolidation volume changes (and vertical dial gage if used) and the initial trimmed diameter and height of the specimen. If vertical dial readings are not taken, isotropic consolidation should be assumed.

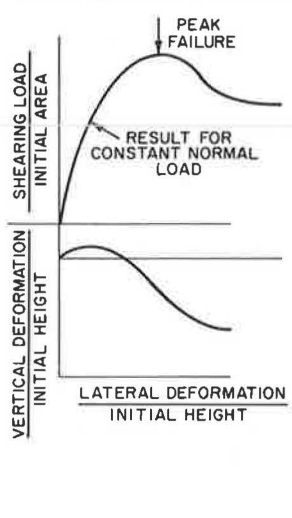
In the direct shear test the normal and shear stresses are generally calculated by assuming that the shear area remains constant and equal to the area of the shear box. If this is assumed, the shear stress and vertical strain (vertical deformation divided by original height) as ordinates may be simply calculated and plotted against shear strain (lateral deformation divided by original height) as abscissi (see Figure 4). The peak shear stress as ordinates from different tests are then plotted against normal stress as abscissi (see Figure 5). Contours of equal shear strain may be plotted in this graph in the event that shear deformations will control the selection of shear strength.

In the triaxial test, unlike the direct shear test, the instantaneous area of the sample is normally calculated by assuming that it remains a right cylinder. This instantaneous area is used to calculate the axial stress difference, which along with the pore pressure in the undrained test or volume strain in the drained test is plotted as ordinates against axial strain as abscissi. For the drained tests, plots akin to those in Figure 4 are obtained.

The effective stress Mohr circle should be drawn for several different tests from the peak shear stress (see Figure 6).

The plot of shear stress plotted against effective normal stress is used to obtain the effective stress or long-term strength parameters. The slope of three or more tests over the in situ working range is the effective friction angle (ϕ) and the intercept the effective cohesion (c'). As seen in Figure 5, c' and ϕ are not unique and must be selected with care.

Figure 4. Typical plot of direct shear box test data.



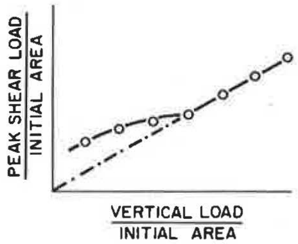


Figure 5. Typical failure criteria plot from direct shear box test results.

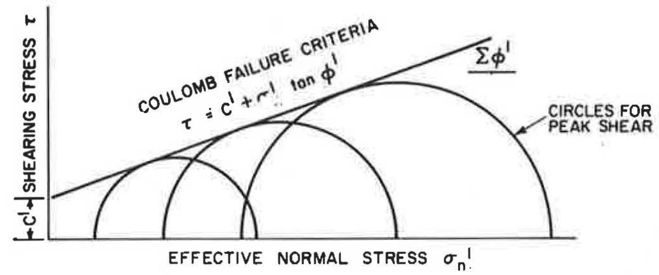


Figure 6. Typical failure criteria from triaxial test results.

CONCLUDING REMARKS

Although the discussion has been restricted to the direct shear and triaxial compression test, the reader should understand that other methods of test may be used with equal satisfaction.

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Undrained Shear Strength of Saturated Clay

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A commonly used method for determining the undrained shear strength of saturated clays is examined. Some of the advantages and disadvantages of this procedure, which is proposed for use with normally and lightly overconsolidated saturated clays of low to moderate sensitivity, are summarized. The properties of normally consolidated deposits change with time, primarily due to secondary compression effects. Tests of aged, normally consolidated deposits will behave as lightly overconsolidated materials and the measured s_u will be related to the quasi-preconsolidation pressure. This hypothesis serves as the basis for the model described for predicting the in situ undrained shear strength of a saturated clay.

The procedures described in this paper are proposed for use with normally and lightly overconsolidated saturated clays of low to moderate sensitivity. They should be suitable for a saturated clay that has an undrained shear strength less than 1 ton/ft^2 (100 kPa) and an overconsolidation ratio less than 4. Monotonic loading is assumed, and the effects of cyclic or repeated loads are not considered.

CONCEPT OF UNDRAINED STRENGTH

Natural deposits of saturated clay frequently are loaded (or unloaded) rapidly relative to the rate at which consolidation or drainage can occur. For such circumstances an ideal undrained condition may be assumed. The water content and the volume of the clay remain constant during the undrained loading, and excess pore water pressures are generated. The shear strength for such conditions is defined as the undrained shear strength (s_u).

If the undrained behavior of saturated clays is analyzed in terms of total stresses, then the evaluation of pore water pressures is unnecessary. The $\phi = 0$ method of analysis (1) is assumed, and the undrained shear strength (s_u) is assumed equal to the cohesion intercept (c_u) of the Mohr-Coulomb envelope for total stresses. For these assumptions the undrained strength of a saturated clay is not affected by changes in confining stress so long as the water content does not change.

The undrained shear strength of a saturated clay is related to the consolidation history of the deposit. For young, normally consolidated deposits, the water content may be assumed to be uniquely related to the consolidation pressure (p_c), which is equal to the in situ effective overburden pressure (p_o') and thus s_u also is presumed to be a linear function of p_o'. The use of the ratio s_u/p_o' was suggested by Skempton (2). For lightly overconsolidated clays, s_u becomes a function of the current consolidation pressure [or water content (w)] and the maximum past consolidation pressure (p_{cm}). These relations are shown in Figure 1. For normally consolidated conditions, the curve of log s_u versus w is assumed to be approximately parallel to the virgin compression curve.

The properties of normally consolidated deposits change with time, primarily due to secondary compression effects [Bjerrum (3) and Leonards and Ramiah (4)]. Thus, the water content does not remain a unique function of the effective overburden pressure. The undrained shear strength increases and a quasi-preconsolidation pressure develops. As a result, tests of aged, normally consolidated deposits will behave as lightly overconsolidated materials and the measured s_u will be related to the quasi-preconsolidation pressure.

The preceding hypotheses provide a simple model for prediction of the in situ undrained shear strength of a saturated clay. The implication is