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

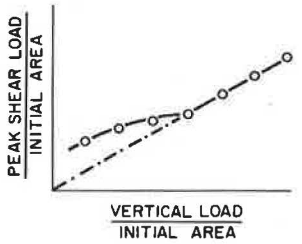
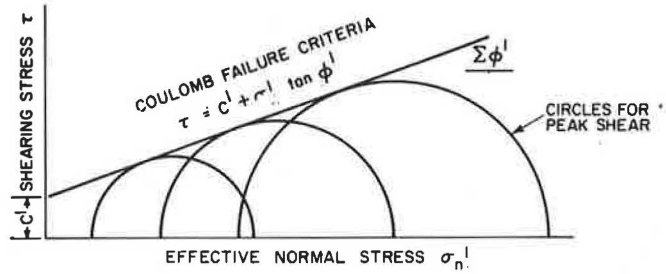



Figure 6. Typical failure criteria from triaxial test results.




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

HARVEY E. WAHLS

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 ($s_u$).

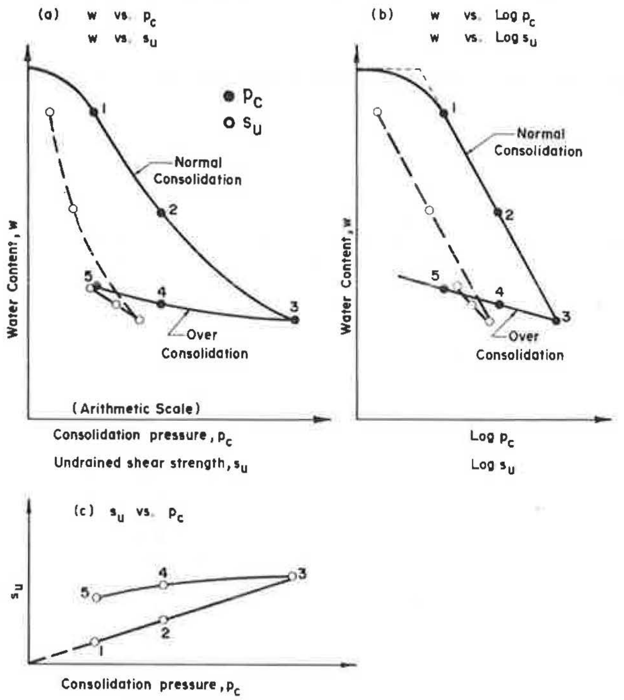
If the undrained behavior of saturated clays is analyzed in terms of total stresses, then the evaluation of pore water pressures is unnecessary. The $\phi = 0$ method of analysis (1) is assumed, and the undrained shear strength ($s_u$) is assumed equal to the cohesion intercept ($c_u$) of the Mohr-Coulomb 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

Figure 1. Effects of consolidation history on undrained shear strength.




that the in situ $s_u$ can be evaluated by any type of undrained shear test conducted on an undisturbed sample at the in situ water content. These assumptions provide the basis for most undrained analyses of saturated clay in current U.S. practice. The limitations of these assumptions will be discussed in the section on factors that affect undrained test results.

## MEASUREMENT OF UNDRAINED STRENGTH

A value of the undrained shear strength of a saturated clay specimen may be obtained by many laboratory and field tests. The common requirement of these tests is that the failure stresses should be developed without drainage or volume change. Also, tests must be conducted on relatively undisturbed soil. The primary tests used in current practice are as follows:

1. Laboratory tests--unconfined compression tests, triaxial compression tests [unconsolidated undrained (UU) and consolidated-undrained (CU)], direct box shear tests (UU and CU), and direct simple shear tests (CU); and
2. In situ tests--vane shear tests, cone penetration tests, and pressuremeter tests.

### Unconfined Compression Test

The unconfined compression test (ASTM D2166) is the most widely used laboratory test of undrained strength. The test is performed on an undisturbed cylindrical sample, which is extruded from a thin-walled sampling tube or trimmed from a block sample. The test specimen should be at least 33 mm (1.3 in.) in diameter and have a length (L) to diameter (D) ratio between 2 and 3. In order to minimize effects of sample disturbance, the test specimen should be as large as possible with the available undisturbed soil and should maintain the proper L/D ratio. The length, diameter, and weight of the test specimen should be measured before testing.

The specimen is placed in a strain-controlled axial compression apparatus and loaded at a strain rate of approximately 1 percent/min. The axial load is measured with a calibrated proving ring or force transducer, and the corresponding axial deformation is measured with a dial gage or displacement transducer. Load-displacement data are recorded continuously with a x-y plotter or recorded manually at regular intervals so that a stress-strain curve may be plotted. The test continues until the axial load remains constant (or decreases) or the axial strain reaches an arbitrarily selected limit (e.g., 15 or 20 percent). After completion of the test, the specimen is weighed and its water content is determined.

The unconfined compression strength ($q_u$) is the peak value of axial load divided by the corrected area, $A_c = A_o/1-\varepsilon$, where $A_o$ is the initial cross-sectional area and $\varepsilon$ is the vertical strain. The undrained shear strength ($s_u$) is assumed equal to one-half of $q_u$.

The undrained strength as evaluated from the unconfined compression test often underestimates the in situ undrained strength of a saturated clay because of the effects of sample disturbance, discontinuities, and sand partings.

### Triaxial Compression Test

The triaxial compression test provides positive control of drainage conditions and the capability for assessing the effect of consolidation pressure on the undrained strength. The apparatus, sample preparation, and test procedures for triaxial compression tests are described by Raymond in a paper in this Record. However, when these tests are used to evaluate only undrained strength, effective stress parameters are not required and pore pressure measurements are not essential.

For UU triaxial tests (ASTM D2850) the drain valves remain closed throughout the test. The chamber pressure is set approximately equal to the in situ effective overburden pressure at the depth from which the undisturbed sample was obtained, and the axial loading may be started immediately.

For CU tests the sample is allowed to consolidate under the chamber pressure before the undrained axial loading is started. When the in situ undrained strength is to be estimated, the consolidation pressure is set equal to the in situ effective overburden pressure.

For UU tests the axial loading is applied at a strain rate of approximately 1 percent/min instead of the slower rates that are suggested by Raymond for tests in which pore pressure measurements are required. The principal stress difference (deviator stress) $(p_1 - p_3)$ is computed as the axial piston load divided by the corrected area of the sample and plotted as a function of the axial strain. The undrained shear strength ($s_u$) is defined as one-half of the peak value of $(p_1 - p_3)$. For CU tests, the slower strain rates of Raymond should be retained to more closely approximate field rates of loading.

The UU test provides a measure of $s_u$ at the in situ water content of the sample. The value of $s_u$ often underestimates the in situ $s_u$ because of disturbance and stress relief effects associated with sampling and testing. However, a CU test conducted at the in situ effective overburden pressure will usually overestimate the in situ undrained strength of normally and lightly overconsolidated clays because the laboratory specimen will reconsolidate to a water content that is lower than the in situ value. Ladd and Lambe (5) have reported that values of $s_u$ from UU tests of normally consolidated clays may be only 40 to 80 percent of the values obtained from CU tests at the in situ effective overburden pressure.

CU triaxial tests also may be used to evaluate the relation of undrained strength to consolidation pressure. A series of CU tests, which is conducted at several consolidation pressures in excess of the in situ effective overburden pressure of the sample, will estimate the increase of $s_u$ with increasing consolidation pressure and decreasing water content. These tests also are used in conjunction with the relatively new normalized analysis of $s_u$, which is discussed in the final section of this paper.

### Direct Box Shear Test

The direct box shear test is not well suited for undrained strength tests because the drainage conditions are difficult to control. The apparatus, sample preparation, and testing procedures for drained direct box or ring shear tests are described by Raymond. For an undrained test the procedures are similar except that the horizontal shearing force should be applied without any volume change.

Both UU and CU direct shear tests may be attempted. For UU tests the horizontal shear loading is started immediately after application of the vertical normal stress. For CU tests the specimen is allowed to consolidate under the applied vertical normal stress before starting shear.

Often assumed is that undrained shear can be accomplished by performing the test rapidly, but experimental evidence (6) indicates that this assumption is seldom valid. Drainage can only be prevented during shear by varying the vertical normal stress so as to maintain a constant sample thickness and thus a constant volume (7).

The box shear test has several other disadvantages. The rigid boundaries of the apparatus cause extremely nonuniform strains and a progressive failure along the horizontal plane. The state of stress within the sample is indeterminate.

The horizontal shear stress is computed as the horizontal force divided by the horizontal area of the test specimen and is plotted as a function of horizontal displacement. The undrained shear strength ($s_u$) is defined as the peak value of the horizontal shear stress.

### Direct-Simple Shear Test

The direct-simple shear test is conducted on a cylindrical sample encased in a wire-reinforced rubber membrane. The flexible boundary allows the stresses and strains to develop relatively uniformly within the sample. Although the general state of stress within the sample is indeterminate, the average normal and shear stresses acting on ahorizontal plane can be evaluated with sufficient accuracy for practical applications (8,9).

The procedures for conducting the direct-simple test are described by Bjerrum and Landva (10). For undrained shear tests, the vertical normal pressure must be varied to maintain a constant volume. The undrained shear strength ($s_u$) is defined as the peak value of the horizontal shear stress.

### Vane Shear Test

The vane shear test is the most commonly used in situ undrained shear test. The conventional vane has four vertical rectangular blades and a height-to-diameter ratio of two (ASTM D2573). The vane is pushed into undisturbed soil and rotated at a rate of 0.1 degree/sec. The torque required to rotate the vane is measured as a function of angular deformation, and the maximum torque ($T_m$) is used to compute the undrained shear strength, assuming that the shearing resistance is uniformly mobilized along the surface and ends of a cylinder of the height (H) and diameter (D) of the vane. For this assumption and H/D = 2,

$$s_u = 6T_m/7\pi D^3 \quad (1)$$

This expression also assumes that $s_u$ is the same on vertical and horizontal planes (i.e., the soil is isotropic). However, because approximately 85 percent of the torque is used to mobilize the shearing resistance around the circumference of the cylinder, the vane test primarily measures $s_u$ along vertical planes.

### Cone Penetration Test

The cone penetration test has been used in Europe for many years but rarely in the United States before the mid-1970s. Although many cone penetration tests have been developed, the Dutch cone test has become the most popular. This quasi-static test employs a cone with a 60° point angle and a base diameter of 36 mm (1.4 in.), which provides a projected area of 10 $cm^2$. The cone is pushed into the ground at the rate of 10 to 20 mm/sec (2 to 4 ft/min), and the penetration resistance of the tip is recorded. Often the cone is fitted with a friction sleeve that can be used to measure local skin friction.

The undrained strength of a saturated clay is computed as

$$s_u = (q_c - p_o)/N_c \quad (2)$$

where

$q_c$ = cone resistance = $R_p/A$,
$R_p$ = point resistance of cone,
$A$ = projected area of cone,
$p_o$ = total overburden pressure for the depth at which $q_c$ is measured, and
$N_c$ = cone factor.

The cone factor ($N_c$) has commonly been assumed equal to 10 for electrical penetrometer tips and equal to 16 for Begemann mechanical tips (11). More recently, Lunne and Eide (12) reported $N_c = 15 \pm 4$ for electrical cones with Scandinavian clays. More research is needed to ascertain the potential variability of $N_c$ and hence the reliability of the cone penetration test for measuring $s_u$.

### Pressuremeter Test

The pressuremeter test measures the pressure required to expand a flexible cylinder against the sides of a bore hole. The original Menard device, which was developed more than 20 years ago, is lowered into a predrilled bore hole. More recent designs in France (13) and England (14) incorporate a small cutting tool, which makes the device self-boring. The undrained shear strength of a saturated clay is evaluated from pressure-radial expansion data for the expandable test cell. Details of the theory and test procedures for pressuremeters are discussed by Baguelin and others (15), Schmertmann (11), and Ladd and others (16).

In recent years use of the Menard pressuremeter has increased in the United States. However, the primary applications appear to have been for cohesionless soils and partly saturated residual soils. For well-behaved saturated clays, the pressuremeter has been used primarily to evaluate the undrained modulus of deformation rather than the undrained strength. Therefore, the pressuremeter test is not discussed further.

FACTORS THAT AFFECT UNDRAINED TEST RESULTS

The evidence in recent literature is ample that different estimates of $s_u$ are obtained by performing different undrained shear tests on identical samples at the in situ water content. These differences generally are attributed to the effects of sample disturbance, anisotropy, strain rate, and creep.

Sample Disturbance

Sample disturbance is present to some degree in every laboratory test specimen. Disturbance results from remolding during the field sampling and laboratory sample preparation and from the stress relief associated with removal of the sample from the ground. The latter effect is not present for in situ shear tests, but some remolding does accompany the insertion of field shear devices into undisturbed soil. However, sample disturbance effects are usually less important for in situ tests than for laboratory tests. Sample disturbance always reduces the undrained shear strength, assuming the water content is unaltered by the sampling and testing procedures.

Anisotropy

Anisotropy is present in most natural clay deposits. Ladd and Foott (17) present an excellent discussion of the effects of anisotropy on various types of undrained shear tests. The $s_u$ measured in each type of shear test depends on the direction of failure plane along which the shearing resistance is mobilized. For example, vane shear, direct shear, and triaxial compression tests measure $s_u$ along vertical, horizontal, and inclined planes, respectively. Each of these tests would be expected to produce a different value of $s_u$ in an anisotropic soil. Ladd and Foott (17) suggest that the $s_u$ measured from a triaxial compression test generally is greater than the value from a direct shear test, which in turn is greater than the value from a triaxial extension test.

Strain Rate and Creep

Strain rate and creep effects are interrelated. The slower the strain rate, the more creep occurs during shear. Undrained creep reduces $s_u$, and thus a reduction of the strain rate reduces the measured $s_u$. Ladd and Foott (17) report that each log cycle decrease in strain rate may result in a 10 $\pm$ 5 percent decrease in $s_u$. They note that the $s_u$ obtained from undrained triaxial tests conducted at an axial strain rate of 1 percent/min may be 20 to 30 percent higher than the value obtained when the strain is reduced to permit meaningful pore pressure measurements in a high plasticity clay. The strain rates used in undrained shear tests are much faster than the strain rates associated with most field design problems. For soils that have significant creep characteristics, the $s_u$ determined from undrained shear tests should be adjusted to provide a more appropriate estimate of $s_u$ for use in undrained analysis and design.

In laboratory tests the above effects tend to produce errors that cancel each other; however, correction factors have been developed to provide improved estimates of $s_u$ from in situ tests. A correction factor for vane shear tests was proposed by Bjerrum (18) on the basis of analyses of embankment failures. Bjerrum's recommended correction curve and the data on which it is based are shown in Figure 2 [Ladd and others (16)]. Additional data subsequently provided by other researchers also are shown in Figure 2. The data vary by $\pm$25 percent from Bjerrum's curve.

ESTIMATING IN SITU STRENGTH

Conventional Method

The current U.S. practice for estimating design values of in situ undrained shear strength from results of undrained shear tests is based on the concepts shown in Figure 1 and described in the section on concepts of undrained strength. The undrained strength is either determined from laboratory or in situ undrained tests performed on undisturbed samples at the in situ water content or from CU laboratory tests consolidated to effective overburden stress. The effects of sample disturbance, anisotropy, and strain rate usually are not considered quantitatively. However, qualitative consideration of these factors undoubtedly plays a role in the selection of design values of $s_u$. In some instances empirical correlations or correction factors (such as the curve shown in Figure 2) are employed. Field vane, unconfined compression, and unconsolidated undrained triaxial compression tests are the most commonly used tests. Sufficient local experience with a given clay deposit leads to appropriate design estimates of $s_u$ by using one or more of these tests. When local experience is lacking, the potential exists for significant error in the evaluation of $s_u$.

A degree of empiricism can be found in the application of all of the tests described here. Recognition of this is responsible for the resistance to introduction of new test methods for which less experience and empirical data are available.

Normalized Analysis

Ladd and Foott (17) proposed a new method for evaluating the in situ $s_u$ for design from a normalized analysis of laboratory CU tests. The procedure involves the anisotropic consolidation of undisturbed samples to consolidation stress greater than the in situ maximum past pressure. For some samples the consolidation stress subsequently is reduced to create overconsolidated samples. Undrained shear tests are conducted on these normally consolidated and overconsolidated samples, and the results are plotted in terms of the normalized parameters ($s_u/p_{vc}$ and $p_{cm}/p_{vc}$), where $p_{vc}$ is the vertical consolidation stress during the shear test and $p_{cm}$ is the maximum past consolidation pressure. A typical diagram is shown in Figure 3 (16). The in situ $s_u$ for design is com-

Figure 2. Correction factor for vane shear data.

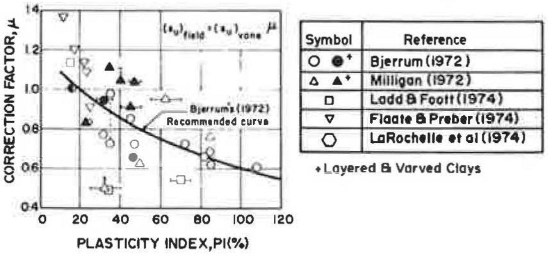
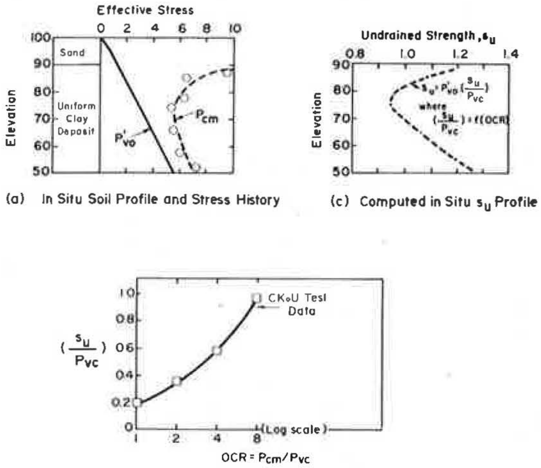


Figure 3. Normalized analysis for in situ $s_u$.




(a) In Situ Soil Profile and Stress History

(c) Computed in Situ $s_u$ Profile

(b) Normalized Undrained Strength vs. OCR from Laboratory Shear Tests

puted by multiplying the in situ vertical effective overburden pressure by the value of $(s_u/p_{vc})$ that corresponds to the in situ overconsolidation ratio.

The procedures appear to provide a rational basis for evaluation of the undrained shear strength versus depth for clays of low-to-moderate sensitivity. Effects of sample disturbance, which are inherent in conventional tests at the in situ water content, are minimized. Anisotropy can be considered by selecting the type of shear test that best models the failure mode of the design problem, and strain rate effects are minimized by shearing the CU tests in accordance with the recommendations of Raymond in a paper in this Record. However, the procedure involves much more testing than is required for conventional analyses. Thus, the normalized approach can only be justified for large projects or for many projects on the same widespread clay deposit (16).

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I am grateful for the assistance provided me by many members of the TRB Committee on Soil and Rock Properties. Special recognition is given to C.C. Ladd, J.A. Tice, and to the past committee chairman, W.F. Brumund.

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