Laboratory Methods for Determining Engineering Properties of Overconsolidated Clays

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One objective of a site and soil characterization program is to determine pertinent engineering properties, including the state of stress, stress history, and basic mechanical soil properties such as consolidation and strength characteristics. This is best done in the laboratory since boundary conditions and strain rates can be controlled. Laboratory test equipment and procedures for evaluating the consolidation and stress-strain-strength properties of saturated overconsolidated (OC) clays are described. A distinction is made between the state of the art and the state of the practice with respect to equipment and test methods. An overview is given of common laboratory equipment for determining these properties. Background is provided on preconsolidation pressure (σ_p') mechanisms and respective stress history profile characteristics. Guidance is offered on strength parameter selection for OC clays, which includes stability class, soil behavior issues, and methods for reducing the effects of sample disturbance. General procedures are given for performing one-dimensional consolidation tests on OC clays and estimating values of σ_p' and the coefficient of earth pressure at rest. Recommendations are given on evaluating and analyzing strength data, and it is concluded that obtaining reliable engineering properties for OC clays requires a comprehensive knowledge of deposition mechanisms, soil behavior, and appropriate experimental procedures.

Laboratory testing of overconsolidated (OC) clay soils presents a series of unique and challenging problems for geotechnical engineers. Formulating an appropriate laboratory testing program requires a broad knowledge of the soil's deposition mode, stress history, preconsolidation mechanism(s), and the various aspects of OC clay behavior. The relatively higher-strength and dilative nature of OC clays may require adopting very different testing methods, equipment, and instrumentation, necessitating input from a seasoned experimenter.

The two general objectives of a site and soil characterization program are (*a*) to determine the soil profile, identifying soil types and their relative states, and (*b*) to determine pertinent engineering properties, including the initial state variables such as the state of stress and prior stress history, and basic soil properties such as consolidation and drained/undrained shear characteristics. A combination of in situ testing and undisturbed sampling for laboratory testing should be employed. Each approach has certain advantages and limitations, as summarized in Table 1.

Laboratory test equipment and procedures for determining the consolidation and stress-strain-strength properties of saturated OC clays are described. The comments and recommendations represent a compromise between the state of the art and the state of the practice. Several laboratory devices and test procedures currently used in practice are not considered reliable by the authors and are not recommended for determining design parameters. However, an attempt has been made to concentrate the recommendations on the use of laboratory equipment that is realistically available to practicing geotechnical engineers in North America. Particular emphasis is given to oedometer, triaxial, and direct simple-shear (DSS) equipment.

Dynamic testing or testing of stiff-fissured clays, clay shales, and volumetric measurement of expansive soils is not covered. Publications by Rowe (*1*), Skempton (*2*), Brooker and Peck (*3*), and Fredlund and Rahardjo (*4*) address these issues.

LABORATORY EQUIPMENT

A number of laboratory test devices have been developed to evaluate the consolidation and stress-strain-strength behavior of clays. These tests range from very simple, taking only a few minutes to perform, to extremely complicated, with one test taking several weeks to perform. Details regarding the capabilities of most laboratory equipment and the practical significance of test results obtained using them are provided elsewhere (*5–10*). Some sophisticated laboratory equipment, including the torsional shear hollow cylinder (*6,11*), directional shear cell (*12*), and the multidirectional direct simple hear apparatus (*13*), that is primarily used in research to investigate soil anisotropy will not be described in this paper.

Radiography of tube samples can show variations in soil type, macrofeatures, intrusions, voids, or cracks and variations in degree of sample disturbance. Many of these features cannot be readily identified from visual inspection of extruded samples. Therefore, radiography of sample tubes provides a nondestructive means for selecting the most representative and/or less-disturbed portions of each tube for engineering tests. Such information can be considered essential for projects having a limited number of tube samples. For descriptions of various radiography methods, see ASTM Standard D4452.

Proper evaluation of the stress history (especially the preconsolidation pressure σ_p') is one of the most important objectives of any site and soil characterization program. The most effective laboratory method for determining σ_p' uses oedometer equipment to perform the one-dimensional consolidation test. Although the common incremental loading (IL) device is usually sufficient, more advanced consolidometers may yield more accurate results. Constant rate of deformation or strain (CRS) devices allow back pressure saturation of the specimen, provide continuous stress-strain data, and produce more timely results than the IL device. Estimates

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TABLE 1 Comparison of Advantages and Limitations of In Situ Versus Laboratory Testing for Cohesive Soils (*10*)

IN SITU PENETRATION TESTING	UNDISTURBED SAMPLING - LABORATORY TESTING
Advantages	
1. More economical and less time consuming. **2.** Semi-continuous profile. **3.** Response of large soil mass. **4.** Response to natural environment, i.e., in situ temperature and no stress relief. *Therefore* Best for Soil Profiling	**1.** Known soil type, i.e., classification & index properties. **2.** Well defined and controlled boundary conditions: drained or undrained; variable stress paths; specified strain rate. *Therefore* Best for Most Engineering Properties
Limitations	
1. Unknown effects of installation. **2.** Poorly defined stress & strain boundary conditions. **3.** Cannot control drainage conditions. **4.** Nonuniform and high strain rates. *Therefore* Interpretation of Data Depends on Empirical Correlations	**1.** Expensive and time consuming. **2.** Unavoidable stress relief. **3.** Effects of sample disturbance may be difficult to identify and minimize. **4.** Small, discontinuous test specimens. *Therefore* Not Well Suited for Soil Profiling

of σ'_p can also be obtained from triaxial and direct simple shear (DSS) strength tests that involve one-dimensional consolidation of specimens to a vertical effective stress, $\sigma'_v \gg \sigma'_p$.

Strength index tests, including the torvane, pocket penetrometer, lab miniature vane, Swedish fall cone, unconfined compression (UC), and unconsolidated-undrained triaxial compression (UUC), are relatively simple and inexpensive to perform. For cohesive soils, these tests represent an unconsolidated-undrained (UU) procedure and provide some measure of the undrained shear strength (c_u) of the soil. However, because these test types are greatly affected by sample disturbance and involve very fast rates of shearing and different modes of shearing, the c_u data reflect, at best, relative changes in strength rather than values suitable for design. Considerable data scatter is common in strength profiles developed from strength index test data. Ideally, for determining design values of c_u, the combined use of consolidation and consolidated-undrained (CU) shear tests should be emphasized over strength index tests. However, data from strength index tests can provide a general picture of the consistency of different soil layers (spatial variability) and an assessment of variations in the degree of sample disturbance within individual tube samples. In addition, useful site-specific correlations can often be obtained between strength index tests and more sophisticated laboratory tests.

The direct shear box test is one of the earliest and simplest devices developed for measuring the behavior of soils. This device cannot produce valid stress-strain data because the complete state of stress is unknown. Also, generally only the drained residual strength of OC clays can be obtained using the direct shear box by repeatedly reversing the directions of shear until a well-developed failure surface is obtained. However, the ring shear apparatus is better suited for determining this property (*14*). The DSS apparatus was developed to improve the limitations of the direct shear box, used increasingly even though the complete state of stress during shear is unknown. Through the use of proper testing procedures, triaxial equipment can provide reliable design parameters for clay soils. During the past decade, an increasing number of geotechnical engineering laboratories, including state departments of transportation, have supplemented their triaxial equipment with more versatile automated triaxial stress path cells. Plane strain compression/extension devices are appealing because they apply a stress-strain condition found in many geotechnical engineering problems. These devices can provide reliable CK_oU plane strain data (*15*) but they are also complicated and not common in geotechnical engineering laboratories.

PRECONSOLIDATION PRESSURE MECHANISMS

In its simplest definition, the preconsolidation pressure σ'_p is the "the maximum past pressure" that acted on a clay soil. However, it is now more generally recognized that σ'_p represents the one-dimensional yield stress of the soil, separating stress states that cause largely elastic, small-strain behavior from those causing large-strain, plastic behavior. For horizontal soil deposits with geostatic stresses, Jamiolkowski et al. (*7*) identified five preconsolidation causal mechanisms: (*a*) changes in total stress, (*b*) changes in pore pressure, (*c*) drained creep, (*d*) physicochemical effects, and (*e*) desiccation. Preconsolidation caused by either changes in total stress or pore pressure are relatively easy to identify since ($\sigma'_p - \sigma'_{vo}$) is constant with depth, where σ'_{vo} is the in situ vertical effective stress. Drained creep or aging is continued deformation of a soil under constant effective stress; over time, it will cause a normally consolidated soil to increase in yield stress so that $\sigma'_p > \sigma'_{vo}$ In this case, the stress history profile will be characterized by a constant value of overconsolidation ratio (OCR) versus depth. Physicochemical effects are caused by natural cementation and related phenomena and typically result in variable stress history profiles. Although this mechanism is pronounced in eastern Canadian clays, it is generally poorly understood and difficult to prove (*7*). Desiccation caused by evaporation, vegetation, and freeze-thaw cycles is common near the surface of clay deposits and typically results in variable σ'_p usually decreasing with depth. The near-surface zone influenced by desiccation in a clay deposit is often referred to as a "clay crust" and is very common in North America.

SELECTION OF STRENGTH PARAMETERS FOR DESIGN

There are many issues to consider when selecting consolidation and strength parameters for the design of constructed facilities built on OC clays. Evaluation of these issues is important when developing a laboratory testing program so that relevant and realistic design values are determined for a given problem. The different classes of stability problems are discussed and some of the more important issues that need to be considered when conducting a laboratory test program for OC clays are identified. Two design methods that have been developed to explicitly deal with these important issues, during both laboratory testing and final selection of design parameters, are reviewed.

Classes of Stability Problems

Stability problems involving cohesive soils are typically divided into the following three categories depending on the drainage conditions anticipated during construction and during a potential failure (*9*): (*a*) undrained or short-term, where the stability is controlled by the undrained shear strength (c_u) of the soil; for limit-equilibrium computations, a total stress analysis or undrained strength analysis (USA) should be used; (*b*) drained or long-term, where the stability is controlled by the drained shear strength (s_d) of the soil; for this class, an effective stress analysis (ESA) should be used for limit-equilibrium computations; and (*c*) partially drained or intermediate; staged construction is an example of this class and selection of an analysis method is controversial, with both the ESA and the USA currently being used to evaluate stability during construction (*9*).

When selecting a particular stability class and limit-equilibrium analysis method, it is also important to distinguish between problems that involve loading (e.g., embankments, tanks, building foundations) and those that involve unloading (e.g., excavations, Rankine active earth pressure). What happens some time after undrained loading/unloading is critical in determining appropriate strength parameters. Undrained loading of OC clays will generally produce positive excess pore pressures (Δu) unless the soil is heavily overconsolidated. As the stress paths in Figure 1(a) indicate, subsequent dissipation of these excess pore pressures causes the shear strength of the soil to increase at constant total vertical stress (σ_v) as effective stresses change (Path B-C). As a result, the undrained shear strength is the critical design strength. In the case of heavily overconsolidated clays, the excess pore pressure at failure may be negative and thus the drained shear strength may be more critical for design; in these cases stability should be checked using both the drained and undrained strengths. Undrained unloading of OC clays produces negative excess pore pressures and the stress paths indicated in Figure 1(b). With time after unloading, the negative excess pore pressure will be satisfied by taking in water, decreasing the shear strength at constant σ_v. In this case, the most critical design strength is the drained shear strength.

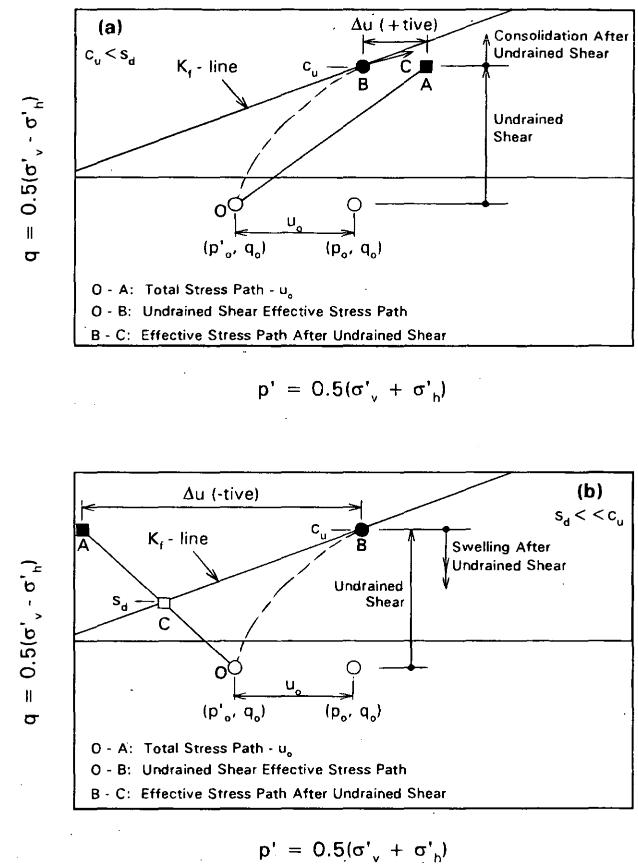

FIGURE 1 Stress paths for undrained shear followed by drainage of an OC clay (*Ko* > 1): *(a)* loading TSP; *(b)* unloading TSP.

Important Soil Behavior Issues

The following important soil behavior issues must be considered when developing a laboratory testing program for OC clays: anisotropy, stress history, rate effects, preshear consolidation method (i.e., isotropic versus K_o), drainage conditions (drained versus undrained), sample disturbance, and peak versus residual strength. Consideration of these issues and the class of stability problems being analyzed should dictate the type of information desired from the laboratory testing program. Specific recommendations on how to account for these issues are provided in the following section on the recompression and stress history and normalized soil engineering properties (SHANSEP) methods and also in the consolidation and strength testing sections of this paper. Additional details can be found in a number of publications (*7–10*).

Recompression and SHANSEP Methods

To obtain reliable design parameters for OC soils, the important soil behavioral issues listed in the previous section must be considered. Sample disturbance is essentially the primary problem in laboratory testing of clays. If it is not properly accounted for and measures not taken to mitigate its effects on measured parameters, all subsequent data obtained from the soil may misrepresent the in situ behavior. The most important effect of sample disturbance on strength testing is significant reductions in the effective stress of the sample (σ_s'). Thus, highly variable c_u data are often obtained from UU-type testing (e.g., UC and UUC) and consolidated-undrained (CU) tests must be used to minimize these effects. Both the recompression (*16*) and the SHANSEP (*9,17*) techniques were developed to minimize the adverse effects of sample disturbance on laboratory strength testing of clays. Table 2 gives the basic procedures, advantages, and limitations of both methods. Both require the use of CK_oU tests with shearing in different modes of failure [e.g., triaxial compression (TC), DSS, and triaxial extension (TE)] at appropriate strain rates to account for anisotropy and strain rate effects.

CONSOLIDATION TESTING

General Test Procedures

Accurate determination of the stress history profile of a clay deposit is the most important goal of any laboratory test program for OC clays. Knowledge of the stress history profile provides valuable information regarding what stress-strain-strength behavior should be expected, allows better planning of the number and type of strength tests that need to be conducted, and is also critical for accurate estimation of consolidation settlements. Consolidation testing provides the following information for clays: (*a*) one-dimensional (1-D) compressibility and estimates of σ_p'; (*b*) flow characteristics needed to predict rates of consolidation, and, with special equipment, (*c*) the relationship between the horizontal and vertical consolidation stresses for no lateral strain (i.e., $K_o = \sigma_{hc}'/\sigma_{vc}'$). Estimates of one-dimensional creep behavior (i.e., rate of secondary compression) may be desirable in some cases but typically are not important for OC clays.

The two most common 1-D consolidation tests are (*a*) the conventional incremental loading (IL) oedometer test and (*b*) the

TABLE 2 Recompression and SHANSEP Techniques: Basic Procedures, Advantages, and Limitations (*9*)

RECOMPRESSION	SHANSEP
Basic Procedures	
1. Perform CK_oU tests on specimens reconsolidated to the in situ state of stress, i.e., $\sigma'_{vc}=\sigma'_{vo}$. **2.** Select appropriate combination of TC, DSS and TE tests to account for anisotropy. **3.** Use strain rates of 0.5 to 1 %/hr for triaxial tests and 5 %/hr for DSS tests. **4.** Plot depth specific strength values versus depth to develop c_u profile.	**1.** Establish the initial stress history. **2.** Perform CK_oU tests on specimens consolidated well beyond in situ σ'_p to measure NC behavior and also on specimens rebounded to varying OCR to measure OC behavior. **3.** Select appropriate combination of TC, DSS and TE tests to account for anisotropy. **4.** Use strain rates of 0.5 to 1 %/hr for triaxial tests and 5 %/hr for DSS tests. **5.** Plot results in terms of log (c_u/σ'_{vc}) vs. log OCR to obtain values of S and m for the equation $c_u/\sigma'_{vc} = S(OCR)^m$, where $S = c_u/\sigma'_{vc}$ for OCR = 1 and m is strength increase exponent. **6.** Use above equation with stress history to compute c_u profile.
Advantages/Limitations/Recommendations	
1. Preferred method for block samples. **2.** More accurate for highly structured clays. **3.** Preferred for strongly cemented clays and for highly weathered and heavily OC crusts. **4.** Should not be used for NC clays. **5.** Reloads soil in laboratory. **6.** Only gives depth specific strength values. **7.** Should be accompanied by thorough evaluation of stress history to check if c_u/σ'_{vo} values appear to be reasonable.	**1.** Strictly applicable only to mechanically OC and truly NC clays exhibiting normalized behavior. **2.** Preferred for conventional tube samples of low OCR clays having low sensitivity. **3.** Should not be used for highly structured, brittle clays and strongly cemented clays. **4.** Difficult to apply to heavily OC clay crusts. **5.** Unloads soil in laboratory to relevant OCR. **6.** Forces user to explicitly evaluate in situ stress history and normalized soil parameters.

constant rate of strain (CRS) consolidation test. In CRS consolidation tests, the specimen is loaded at a constant vertical strain rate ($\dot{\epsilon}_v$) with measurements of excess pore pressure (Δu) at the bottom, impermeable base (*18*). This approach has the distinct advantage of providing continuous end of primary compression curves and flow properties in less time than possible with IL loading tests. However, CRS tests also require application of back pressure to ensure saturation, pressure transducers, and expertise. As a result, relatively few geotechnical laboratories currently have reliable CRS devices, whereas most laboratories have IL oedometer equipment.

General recommendations for conducting IL oedometer and CRS tests are given in ASTM D2435 and D4186, respectively. Both tests should ideally be conducted by first loading the soil beyond the σ'_p and hence onto the virgin compression line (VCL), have an unload-reload cycle, load to the maximum stress, and finally unload back to the seating load. Because of the effects of sample disturbance, the slope of the recompression line should be estimated from the unload-reload cycle. For OC soils, it is important to prepare the specimen using dry stones to prevent swelling. After application of the initial seating load, water can be added provided that the specimen's deformation is carefully monitored and σ_v' is continuously increased to prevent swelling, if necessary. For IL oedometer tests, the load increment ratio (LIR) should initially be equal to 0.5 up to $\sigma'_{vc} \approx 2\sigma'_{vo}$(required for accurate use of the strain energy method to determine σ'_p; see next section) with subsequent LIR = 1 if the VCL is linear; however, if an S-shaped compression curve is expected (e.g., sensitive soils), an LIR = 0.5 from σ'_{vc} = 0.5 to 3.0 σ'_p should be used (*10*). The ASTM D4186 recommended procedures for CRS tests typically produce strain rates that are too high, especially during virgin compression, and can result in overpredicting σ'_p in some soils (*19*). Ladd and DeGroot (*10*) suggest that the strain rate be selected such that the normalized base excess pore pressure ($\Delta u/\Delta\sigma_v$ where $\Delta\sigma_v$ is equal to σ_v minus the back pressure) is not greater than 0.2.

Preconsolidation Pressure

Numerous techniques have been proposed for estimating σ'_p, but their accuracy depends on a reliable determination of the location of the VCL from tests on high-quality samples and should use end-of-primary data. Two of the more common methods of estimating σ_p will be presented here. Casagrande's (*20*) method is the oldest, simplest, and most widely used technique. However, it is difficult to perform for relatively stiff soils and is subjective, often leading to a significant range in estimated values. In these cases, both the best estimate and range should be reported. The method of Becker et al. (*21*) for interpreting compression data uses work per unit volume or "strain energy" as the criterion for estimating σ'_p. For IL oedometer tests, the work per unit volume associated with each load increment is computed as the product of the average value of σ'_{vc} and the change in natural strain for the increment. The strain energy method involves less judgment than Casagrande's method, especially for "rounded" curves, and can be easily performed using a computer. For final determination of stress history profiles, both the Casagrande and strain energy methods should be used. It is also important to try to discount values of σ'_p that appear to be too low because of sample disturbance. This should be done based on a collective evaluation of the laboratory consolidation test data and results from in situ testing.

Coefficient of Earth Pressure at Rest

The coefficient of earth pressure at rest (K_o) is one of the most difficult soil properties to accurately measure in the laboratory because of the need for special equipment. Several specialized oedometer cells for measuring the relationship between K_o and OCR have been developed (*22*), but most geotechnical engineering laboratories do not have such devices. Another approach uses computer-controlled triaxial stress path cells that can vary the cell pressure to maintain equal axial and volumetric strains, and hence K_o conditions, during strain-controlled consolidation (*23*). However, even with special oedometers and stress path cells, there are still problems in relating laboratory measurements of K_o to in situ values. For example, K_o is much lower when reloading to a given OCR than for unloading to the same OCR. Therefore, accurate prediction of the in situ K_o from laboratory data requires information on how the soil reached its present OC state. The problem is further complicated if the OC mechanism is not mechanical, with a simple loading-unloading stress history, but is due to other, more complex mechanisms such as desiccation and physicochemical effects. Developing the relationship between K_o and OCR using laboratory equipment implicitly assumes mechanical OC. In the absence of reliable laboratory or in situ test data, estimates of K_o versus OCR can be obtained using the empirical correlation presented by Mayne and Kulhawy (*24*).

STRENGTH TESTING

Triaxial Testing

Triaxial testing offers a reasonable means for determining the stress-strain-strength properties of OC clays. However, a number of specialized procedures are required to obtain high-quality results. Recent reviews of triaxial testing (*25–28*) have included some important aspects of testing OC clays, but none has dealt exclusively with these soils. Two significant soil behavior aspects of OC clays govern many, if not all, of the specialized procedures. First, OC clays tend to dilate during shear, which produces potentially large negative excess pore pressures during undrained shear and potentially large water content changes during drained shear. Second, OC clays are usually very stiff and have relatively high strength.

Trimming and Specimen Setup

Trimming can usually be done using a wire saw and the common lathe-type trimming jig. If the specimen shows any friability, this common procedure may result in irregular lateral surfaces after extrusion and trimming. Baldi et al. (*26*) describe a method for continuously supporting the soil as it is extruded and trimmed. Regardless of method, specimens should be trimmed to a right cylinder such that specimen ends are smooth, parallel, and horizontal. Ladd and Dutko (*29*) designed a split-tube trimming device to facilitate this process. Poor-quality end trimming in stiff OC clays will lead to specimen-bending and applied stress nonuniformity. The other, so-called bedding errors from untrue ends can include large initial deformations and resulting errors in the initial modulus. Baldi et al. (*26*) have suggested casting over irregular specimen ends with plaster or resin to create a smooth surface; this should only be done if a smooth surface cannot be achieved by careful trimming. Problems with slight end imperfections can be reduced by consolidating specimens prior to shear (*27*). The use of a fixed-top cap should be avoided; a moment break between the loading ram and top cap is desirable to permit slight top cap adjustment to the top specimen surface.

Stiff specimens should be mounted with saturated pore stones that have a high air-entry value to reduce the chance of cavitation in the stones. The drainage lines from the stones to the first closed valve should be initially left dry. Thus, if cavitation does occur, the resulting pressure gradient cannot be satisfied with water in this area. Filter paper should be applied dry in strips not to exceed 0.64 cm in width. For compression tests, eight 0.64-cm-wide vertical strips can be used (for a 3.6-cm-diameter specimen) provided that a correction for axial load is made and that K_o is not much greater than unity (which may lead to extension strains on the filter strips). For extension tests and tests on heavily OC clays (with K_o much greater than unity), four to six spiral strips, each 0.32 cm wide with small lateral cuts at frequent intervals, should be used. These cuts permit full functioning of the strips, but will reduce the strips' constraining effect at higher extension strains. For a typical 3.6-cm-diameter, 8.1-cm-high specimen, a rule of thumb is to keep the strip inclination such that it wraps about 1¼ to 1½ times around the specimen from bottom to top.

Smooth or lubricated end platens are essential for high-quality triaxial tests (particularly undrained ones) on OC clays to reduce specimen stress-strain nonuniformity and to delay or prevent strain localization in the form of rupture surfaces (shear planes or necking). Although a number of researchers have proposed smooth end platen designs (*25,30*), all of these designs share some basic elements. Lacasse and Berre (*28*) describe a typical setup in which end drainage is eliminated and radial filter strips are draped over enlarged, smooth-end platens to drain into annular or ring-shaped porous stones. Pins extend from the platens to prevent the specimen from sliding out of alignment. The specimen can come into direct contact with the untreated platens. Another common scheme is to place a piece of membrane on each end of the specimen and grease the end platens. However, preshear consolidation pushes the grease out, leaving a high-friction membrane-platen interface. In addition, the extruded grease can block radial filter paper.

Germaine and Ladd (*27*) summarize the advantages and disadvantages of common, frictional ends and smooth or lubricated end platens. For heavily OC clays, frictional ends will cause measurable pore-water migration to the specimen's middle third since the end restraint reinforces OC clay dilatant behavior in the middle by pushing water to that zone. For drained or undrained triaxial tests in heavily OC clays, this will lead to a reduction in measured strength because of a higher water content in that zone. In undrained tests, at a given strain, frictional ends will result in higher measured pore pressures. It has been shown (*27*) that this leads to a failure envelope that overestimates the cohesion intercept (c') and underestimates the friction angle (ϕ').

Two latex membranes, without grease between them, should be used to isolate the specimen from the cell fluid, and for tests of long duration (longer than 1 week), silicon oil should be used as the cell fluid (e.g., Dow Corning "200 fluid" is used at Northeastern University).

Saturation

Since an OC clay specimen is mounted with saturated stones and dry drainage lines (see above), the first step in saturation is to flush the dry lines. The cell pressure, σ_c, should be raised to the estimated σ'_s, which reduces the soil's tendency to take in water. Water should then be flushed through the drainage system without applying any back pressure to the system.

Back-pressure saturation is accomplished in steps by increasing the cell and back pressures in equal increments to maintain the applied effective stress $\sigma'_o = \sigma'_s$ constant. The smaller each increment, the less likely changes in σ'_o are to occur. Specimen height changes and water inflow to the specimen should be monitored during back-pressure saturation to ensure that neither becomes excessive. If they do become excessive, the value of σ'_s has probably been misestimated and the specimen is swelling or consolidating; the applied effective stress should be adjusted up or down, respectively. At Northeastern University the incremental saturation process is facilitated by an automated triaxial apparatus in which multiple-pressure increments of any magnitude can be programmed to be applied over any time schedule.

While final back pressures of 200 to 300 kPa are typically sufficient to obtain saturation, higher pressures may be necessary in stiff soils with a low initial saturation level (e.g., soils from a drying crust). Lacasse and Berre (*28*) report using back pressures of 1500 kPa in stiff clays. Such high pressures prior to consolidation (during which pressures will increase further) require special equipment and instrumentation. For undrained tests, an initial back pressure should be used that allows a large negative pore pressure change to occur during shear without losing saturation (as air comes out of solution).

The value of $B = \Delta u/\Delta\sigma_c$ of the specimen can be checked at various points during saturation. Since by definition the B-value is a function of the ratio of water compressibility to that of the soil skeleton, stiff soils may be saturated without having a B-value close to unity (*31,32*). In such cases, the back pressure should be incrementally increased and the B-value checked after each increment until it levels out.

Consolidation

Consolidation prior to shearing to a representative in situ state of effective stress is the preferred preshear method for testing clays. Triaxial specimens can be consolidated either isotropically (CI) or anisotropically (CA), including the special case of K_o consolidation (CK_o). The CK_o method is preferred for all stress histories. However, for OC clays with an in situ OCR $\geq$ 5 to 6, isotropic consolidation to an effective stress, σ'_c, equal to σ'_{vo}, is an acceptable consolidation method provided that recompression strains do not greatly exceed those in a K_o consolidation test (triaxial or oedometer). Lacasse and Berre (*28*) have also proposed a simplified method for K_o recompression that requires only simple stress path application to the specimen. Figure 2(a) indicates that for a lightly OC clay of moderate plasticity, the estimated K_o line can be reached in steps alternating between axial load and cell pressure increments. This method avoids having the consolidation stress path approach the soil's yield surface, an event that could lead to severe alteration of the soil's preshear state. For higher OCRs, as in Figure 2(b), the K_o state can be reached by isotropically consolidating the specimen to the in situ σ'_{ho} and then decreasing σ'_{vc} to achieve the appropriate K_o value. The necessity for these simplified stress paths is being reduced by the introduction of automated testing. For example, referring to Figures 2(a) and 2(b), the paths OD and OA, respectively, can be directly applied in the Northeastern University automated triaxial system using a drained linear stress path program. An

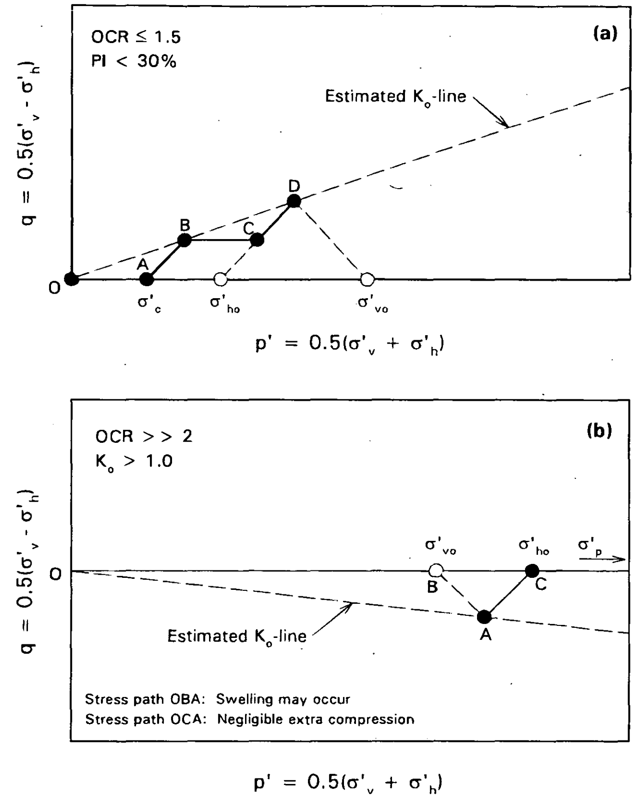

FIGURE 2 Simplified consolidation stress paths: *(a)* lightly overconsolidated clays; *(b)* highly overconsolidated clays *(28)*.

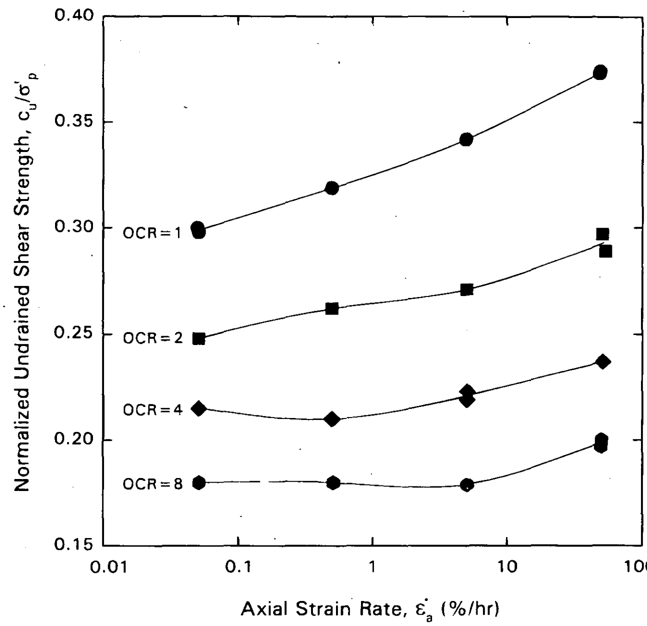

FIGURE 3 Normalized shear strength versus strain rate from SHANSEP CK_oUC tests on resedimented BBC *(35)*.

alternative method used in the system is to compare the product of the original area and axial deformation with the specimen volume change; cell pressure is adjusted to keep them equal, thus achieving K_o conditions.

Shearing

The primary considerations during the shearing phase, other than specimen nonuniformity (discussed above), are the rate of shearing and stress system (compression or extension). For undrained tests, it has been well documented that significant strength changes can occur with strain rate (*33,34*). Most recently, Sheahan et al. (*35*) have shown that this rate dependence varies with both OCR and strain rate level tested in SHANSEP CK_0UC tests on resedimented Boston blue clay (BBC). For OCR = 4 and 8, Figure 3 shows that c_u normalized by σ'_p (for plotting different OCRs on the same axes) is virtually rate independent across the three lowest rates tested (0.05, 0.5, and 5 percent/hr). Across 5 and 50 percent/hr, c_u/σ'_p increases similarly for all four OCRs tested. Sheahan et al. (*35*) used smooth-end platens and mid-height pore pressure measurements, and observed no pore-water migration.

In UUC tests, the typical strain rate is 60 percent/hr, whereas Ladd and Foott (*17*) recommend an axial strain rate, $\dot{\epsilon}$ = 0.5 to 1 percent/hr for CK_oU tests and this is considered "conventional" for such tests. For stiff soils specifically, Berre (*25*) suggested $\dot{\epsilon}$ = 2 to 4 percent/hr. In light of Sheahan et al.'s (*35*) data, it appears that stiff specimens (OCR ≥ 5 to 6) can be sheared undrained at rates close to those suggested by Berre (*25*) without impacting observed behavior. However, smooth-end platens are required and soils of higher plasticity should be checked for rate dependence. For drained tests, Bishop and Gibson (*36*) developed a solution to estimate the appropriate rate of drained shear based on specimen dimensions, coefficient of consolidation and the estimated strain at failure. However, the drained shear strain rate should not exceed 0.1 to 0.2 percent/hr and the back pressure system should contain sufficient water to satisfy specimen intake during shear.

Direct Simple Shear

The DSS apparatus has the unique ability to test soil specimens wherein the major principal stress is free to rotate during simple shear strain conditions. DSS tests are easy to run, have fewer experimental problems, and use little soil compared with triaxial and other shear devices. In the commonly used Geonor DSS (*37*), a circular specimen is trimmed to fit a wire-reinforced membrane allowing specimens to be K_o consolidated. Thus the same compression curve and coefficient of consolidation data are obtained as those in conventional IL oedometer tests (*38*). Undrained shear is typically performed by running constant volume tests for which a number of different methods can successfully be used (*39*). DSS devices cannot impose complementary shear stresses to the sides of a specimen and as a result, a condition of nonuniform stress and strain occurs within the specimen. However, in addition to theoretical analysis, several experimental programs have shown quite convincingly that for plastic soils the uniformity of stress and strain in the device is acceptable up to the peak shear stress (*40*).

In the Geonor DSS, only the vertical effective stress (σ'_v) and the shear stress (τ_h) on a horizontal plane are known. As a result the complete state of stress during shear is unknown. Seven different failure criteria (*38*) have been proposed to estimate Mohr's circle of stress at failure; however, there is still insufficient evidence to indicate which, if any, of the proposed failure criteria are correct. Ladd and Edgers (*41*) and DeGroot et al. (*38*) conclude that measured values of $(\tau_h)_{max}$ give fairly reliable estimates of c_u appropriate for undrained stability analysis and bearing capacity analysis for non-varved sedimentary soils. Since Mohr's circle at failure cannot be determined, the DSS device is not recommended for determining effective stress parameters (i.e., c' and ϕ').

Unlike triaxial testing, procedures for CK_oUDSS tests on OC clays generally do not need to be different than those for NC clays. For recompression tests, the final preshear consolidation stress ($\sigma'_{vc} = \sigma'_{vo}$, Table 2) is typically low, necessitating the use of special stones with pins that penetrate the specimen. The pins reduce the potential for slippage between the specimen and porous stones during shear but also cause an unknown degree of sample disturbance. For SHANSEP tests, in which specimens are mechanically unloaded in the laboratory to varying OCRs (Table 2), normal porous stones can be used without risk of slippage so long as the preshear consolidation stress is not less than 50 kPa. In the standard trimming procedure for the Geonor DSS, the specimen is left unsupported laterally for a short period of time (<30 sec). This typically is not a problem for NC clays but can cause problems with friable OC clay specimens. Based on the method presented by Baldi et al. (*26*) for triaxial specimens, the use of a temporary membrane (not wire-reinforced) that can be rolled over the specimen just prior to the stage where the specimen is unsupported may be effective in preventing specimen degradation.

Ladd and Edgers (*41*) and DeGroot et al. (*38*) provide comprehensive reviews of CK_oUDSS test procedures, data reduction, interpretation, and typical results for a variety of clays. CK_oUDSS test program results should be compared with data reported in these and other relevant references.

Evaluation of Strength Data

Final selection of strength parameters for design should be based on a collective evaluation of results from both the consolidation and strength testing programs. An attempt should be made to assess the reliability of the measured data based on knowledge of the local geology and comparison with prior test programs and information available in the literature.

Undrained shear strength data for a given mode of failure (e.g., TC, DSS, TE) determined from either the recompression or SHANSEP technique should be normalized and plotted as c_u/σ'_{vc} versus OCR on a log-log plot for use in the equation

$$c_u/\sigma'_{vc} = S(\mathrm{OCR})^m \quad (1)$$

where $S = c_u/\sigma'_{vc}$ for OCR = 1 and m is the strength increase exponent. Both of these variables can be determined from linear regression analysis. The data should have a high degree of correlation, especially for data from SHANSEP tests, where the stress history of the soil is created in the laboratory and is therefore well known. The correlation may not be as strong for recompression data, primarily because of uncertainties in estimates of the in situ σ'_p for each test specimen. Except for varved clays, the relative values of c_u should typically follow $c_u(\mathrm{TC}) > c_u(\mathrm{DSS}) > c_u(\mathrm{TE})$. Accounting for anisotropy, the average normalized c_u should approximate (*9*)

$$c_u/\sigma'_{vc} = 0.23 \pm 0.04\ (\mathrm{OCR})^{0.8} \quad (2)$$

These values are typical of those measured from CK_oUDSS tests (*38,41*); therefore, in some cases consideration can be given to conducting only CK_oUDSS tests for determining design values of c_u.

Recommendations given previously for triaxial testing of OC clays should minimize many of the sources of errors in determining the effective stress parameters c' and ϕ'. Values of c' are particularly susceptible to testing errors, and measured values should be compared with those reported by Mesri and Abdel-Ghaffar (*42*). Measured values of ϕ' can be compared with those from a number of researchers' publications, including Jamiolkowski et al. (*8*). Values of ϕ'(TC) are typically less than ϕ'(TE). However, considerable variation in the magnitude of the difference between the two friction angles has been reported in the literature. Jamiolkowski et al. (*8*) provided a summary of results from a number of investigators on this issue. Drained residual strength of clays and clay shales can be compared with typical values documented by Stark and Eid (*14*).

SUMMARY AND CONCLUSIONS

Various issues have been described to consider when formulating a laboratory testing program to determine consolidation and strength characteristics of OC clays. These issues include the deposition and stress history of the soil deposit, the class of problem (drainage and loading mode), soil behavior issues, and methods available for overcoming sample disturbance effects. Reliable shear strength estimates can be obtained only when the specimen is K_o consolidated prior to shear. Special procedures for both consolidation and strength testing of OC clays were outlined, with emphasis placed on two behavioral aspects of these clays: relatively high stiffness and strength and their dilative nature. Analysis of laboratory strength data should include evaluation of normalized strength parameters as well as the traditional c' and ϕ' values. To obtain laboratory data on OC clays that are representative of in situ conditions, significant care is needed in developing the testing program. When knowledgeable practitioners use testing and analysis procedures such as those described here, valid engineering properties on OC clays are attainable from laboratory tests.

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