TRANSPORTATION RESEARCH

RECORD

No. 1479

Soils, Geology, and Foundations

Engineering Properties and Practice in Overconsolidated Clays

A peer-reviewed publication of the Transportation Research Board

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
WASHINGTON, D.C. 1995

Transportation Research Record 1479
ISSN 0361-1981
ISBN 0-309-06117-2
Price: $27.00

Subscriber Category
IIIA soils, geology, and foundations

Printed in the United States of America

Sponsorship of Transportation Research Record 1479

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Foreword

Overconsolidated clays of different geologic origins underlie many parts of the United States and form the foundation materials supporting highway and railroad bridges, pavements, airport runways, buildings, walls, slopes, tunnels, and other transportation facilities. With regard to the characterization of earth-related materials such as soils, clays are more notorious than sands in their difficulty of measurement of relevant strength, compressibility, and stiffness under both short-term and long-term loading conditions. Moreover, clays can be saturated, partially saturated, or dry, and their stress-strain-strength properties depend significantly on whether undrained, partially drained, or fully drained conditions prevail.

The properties of overconsolidated clays are affected by initial stress state, anisotropy, stress history, strain history, mineralogy, aging, fabric, degree of fissuring, geochemistry of the pore fluid, and numerous other factors. The complexities associated with the measurement of any one particular property are shown by a plethora of different instruments and techniques that have been developed for laboratory and field (in situ and nondestructive) assessment of shear strength, for example. In this regard, the strength of clay can be measured in the laboratory using the triaxial apparatus, simple shear, miniature vane, or fall cone or in the field using the cone penetrometer, flat dilatometer, borehole shear, or pressuremeter. Differences among these tests occur because of variations in boundary constraints, different times to failure, and directional changes of loading. Unfortunately, each device does not provide a measure of both drained and undrained strength, nor does it allow the distinction and separate evaluation of the peak, remolded, or residual strength parameters.

Because of the uncertainties associated with assessing soil properties, the TRB Committee on Soil and Rock Properties has been interested in better means and mechanisms for quantifying the characteristics of overconsolidated clays. This volume contains 13 technical papers that address some of the major facets associated with the engineering evaluation of these natural materials. The first six papers address the general conduct of sampling and testing of overconsolidated clays and the remaining seven present case study examples involving site characterization and engineering problems associated with specific local geologic formations in the United States. The authors of these papers come from all corners of the continental United States and therefore provide a representative compilation of practices and methods in use.

An introductory paper by Mayne et al. presents the results of a TRB questionnaire completed by practicing geotechnical professionals and state department of transportation engineers and provides an overview of the state of the practice in the sampling and strength testing of overconsolidated clays. In that survey summary, the most prevalent geotechnical problems deal with slope stability issues in overconsolidated clays, followed by high-swell and expansive clay situations, difficulties in installing and evaluating pile foundation behavior, and excavation construction projects in clay deposits. McManis and Lourie discuss push, drive, rotary, and block methods of sampling. Laboratory test methods appropriate for determining the peak and remolded undrained strengths and effective stress parameters of intact clays are reviewed by DeGroot and Sheahan. The importance of obtaining quality undisturbed samples, stress history effects, and reconsolidation procedures on the measured effective stress paths are discussed. Stark discusses the residual strength of overconsolidated clays, particularly in reference to preexisting slides and slope instability problems. The necessity and importance of using the ring shear device for defining the true residual strength parameters over repeated direct shear testing are emphasized, although guidelines for requiring detailed ring shear tests are given in relation to the mineralogical constituency of the material (i.e., plasticity indices). Huang reviews the common types of in situ tests useful and appropriate for characterizing the undrained strength and field properties of clays. These include the electronic cone penetration and piezocone tests, flat dilatometer test, pressuremeter test, and vane shear test. Mayne gives a unified approach for evaluating the preconsolidated pressure or effective yield stress of clays from the results of different in situ tests. The approach is exemplified using case studies involving natural clay deposits that were tested by the flat dilatometer and piezocone penetrometer.

Macari and Arduino provide an overview of the developments and capabilities of constitutive models for describing the behavior of overconsolidated materials. Predictive examples using the Modified Cam Clay model are given to illustrate a simple means of representing stress-strain-strength behavior in both compression and extension loading over a range of overconsolidation ratios and requiring only four soil properties.

With regards to the interpretation of test results in clays and examples of the site characterization of firm to stiff to hard clays, Lutenegger presents a detailed look at the cause and development of overconsolidated clay crusts, including laboratory and field measurements by a variety of methods.

McGuffey discusses the importance of evaluating the profile of preconsolidation pressures in mechanically overconsolidated clays and stiff clays modified by desiccation effects, particularly with respect to defining crustal layers and slope stability analyses.

O'Neill and Yoon present an overview of the Pleistocene age deltaic clay of the Beaumont and Montgomery formations that underlie the city of Houston, Texas. Here, the overconsolidation has been predominantly caused by desiccation effects, and the properties of laboratory triaxial, consolidation, index, and K_0 measurements have been compiled and compared with the results of standard penetration, cone, pressuremeter, K_0 blade, and geophysical crosshole tests. The University of Houston site, selected and funded by the National Science Foundation and FHWA, is one of the initial five National Geotechnical Experimentation Sites in the United States.

Martin et al. compile a summary of laboratory and in situ testing results used in characterizing the stiff and sensitive Calvert formation that underlies the city of Richmond, Virginia. Erosional processes have been the primary cause of overconsolidation for this Miocene clay.

Edil and Mickelson review the effects of stress history caused by glaciation as relevant to laboratory strength, compression properties, indices, and mineralogies of overconsolidated tills in Wisconsin.

Finally, a synopsis of the geotechnical practice and properties of expansive stiff clay soils in San Diego, California, is given by Spang, particularly emphasizing their influence on slope stability and foundation design.

U.S. State of the Practice in Sampling and Strength Testing of Overconsolidated Clays

PAUL W. MAYNE, ROBERT D. HOLTZ, AND MEHMET T. TUMAY

An indication of current sampling and strength-testing practices in stiff, overconsolidated clays across the United States has been obtained from a nationwide survey of private geotechnical consultants, state and federal highway engineers, and academic institutions. A diversity of sampling techniques, laboratory tests, and in situ field measurements is used in practice depending on the particular geologic setting, local conditions, economics, and experience. Problems involving overconsolidated clays appear primarily related to the proper site characterization and the determination of soil properties for analyses of slope stability, pile foundations, high shrink-swell subgrade soils, and deep excavations. Recent advances in laboratory procedures and in situ testing offer alternative means of assessing the stress-strain-strength behavior of stiff to hard and fissured clays, leading to better economy, reliability, and productivity.

Overconsolidated clays constitute a significant portion of the upper surficial soil formations of the North American continent. These clays, diverse and varied in origin, have primarily been formed by sedimentary deposition in shallow seas or lake beds, although a few clays occur as residuum formed by the in-place weathering of bedrock (*1*). A variety of geologic depositional processes, including marine, glacial, aeolian, lacustrine, alluvial, fluvial, diluvial, and deltaic, together with various time periods and differing environmental conditions, have resulted in a wide assortment of clay deposits found across the United States. As a consequence, each clay deposit is unique, with a different thickness, mineralogical composition, fabric, particle gradation, pore arrangement, geochemistry, and other microstructural features. Common periods of clay deposition resulting in sediment include the Quaternary (Holocene and Pleistocene), Tertiary (Pliocene, Miocene, and Eocene), and Cretaceous. Typical well-known clay deposits include the Pleistocene Beaumont clay of Texas (*2*), Pleistocene Seattle clay (*3*), Cretaceous Potomac Group formation of Washington, D.C. (*4*), Miocene Calvert clay of Richmond, Va. (*5*), Miocene Tampa Bay clay (*6*), and Cretaceous Benton Sea clays (*7*).

Since their deposition, these clays have been geoenvironmentally altered. They are much stiffer and harder than when initially formed as soft, normally consolidated sediments. Natural clays obtain overconsolidated characteristics, having been preconsolidated by one or more of the following processes: mechanical unloading (erosion and glaciation), desiccation, aging, secondary compression, cementation, groundwater fluctuations, freeze-thaw cycles, alternate wetting and drying, seismic events, and other environmental factors.

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Also, human construction activities such as excavation, preloading, surcharging, and ground improvement methods can preconsolidate clay soils.

Conventionally, the results of one-dimensional consolidation tests are used to define the magnitude of the preconsolidation pressure or yield stress ($\sigma_p' = \sigma_{vmax}' = P_c'$), which separates the elastic from plastic behavioral domains (*8,9*). It is common to express the degree of preconsolidation in a normalized form termed the overconsolidation ratio (OCR), σ_p'/σ_{vo}', where σ_{vo}' is the current effective overburden stress (*10*). An alternative method of obtaining the magnitude of σ_p' or OCR from the results of laboratory strength tests is presented by Mayne (*11*). More recently, interest has focused on the possible use of in situ tests for profiling the OCR in clays (*10,12,13*).

AVAILABLE SAMPLING AND TESTING METHODS

For routine site investigations, methods of sampling and testing have been developed in geotechnical practice to characterize the engineering properties of stiff clays. Sampling and testing procedures and equipment have become standardized so that some degree of reliability and consistency of test results among different commercial laboratories, testing agencies, and research institutions can be ensured. In some instances, it has been necessary for consultants and testing agencies to adopt modified procedures or to develop specialized sampling and testing methods because of local anomalies and difficulties not considered by standard practice. For example, standard hydraulically pushed thin-walled (Shelby) tube sampling methods are inadequate for very hard Cretaceous clays. Specimens of these clays are more easily obtained by either drive or rotary coring methods. Additional examples include difficulties with the retrieval of high-quality samples of highly fissured clays, particularly those deposits responsible for shrink-swell damage to foundations and slope stability problems. The degree of fissuring affects the specimen quality during laboratory extrusion, trimming, and saturation. Consequently, swelling and softening often occur in fissured clays, which alter the engineering properties of the soil when subsequently measured in laboratory testing.

A variety of sampling and testing methods has been developed for determining the engineering properties of soil, primarily the shear strength and compressibility characteristics. Unfortunately, strength is difficult to quantify properly because of the inherently variable nature of these soil materials and the effects of disturbance caused by sampling, in situ testing, or both. Because a detailed discussion of sampling procedures and effects is beyond the scope of this paper, the reader is directed to the classic reference by Hvorslev (*14*) and the brief review of common sampling techniques in ASTM

Standard D-4700 (*15*). Sampling disturbance effects on shear strength have been discussed in detail by Ladd and Lambe (*16*) among others.

The complex facets of soil behavior include the effects of anisotropy, nonlinearity, stress rotation, drainage, creep, strain rate, temperature, and rheological factors within a three-dimensional formulation of its stress-strain-strength time response to loading (*10,17*). Consequently, each particular test used to measure strength, for instance, gives a different interpreted value depending on the specific boundary conditions, initial stress state, rate and direction of loading, and induced failure pattern. Figure 1 illustrates some of the laboratory and in situ tests used for measuring and assessing the engineering properties of soil. The applicability of the various laboratory methods for stiff clays is discussed in more detail by Simpson et al. (*18*), whereas Robertson (*19*) and Lunne et al. (*20*) provide details on the in situ testing methods. For strength testing of clays under undrained loading, a wide assortment of laboratory devices is available, ranging from simple index tests (e.g., the fall cone) and routine unconfined compression to sophisticated cubical-type triaxial tests. In the field, undrained strengths can be determined from the simple vane shear and penetrometer devices or complex self-boring pressuremeters. Each of these tests may be used to evaluate the strength of a particular clay, but each device results in a different value because the loading direction, boundary constraints, rates of loading, and disturbance effects are all different (*21*).

Because drainage conditions markedly affect the behavior of clay soils, it is paramount to distinguish between the undrained shear strength (designated τ_f, s_u, or c_u) and the drained shear strength (represented by the effective strength parameters c' and ϕ). For overconsolidated clays, it is not always clear to the engineer whether undrained parameters or drained properties are appropriate for a given problem. In addition, if very large strains are likely to be mobilized, the shear strength of clays is reduced to a frictional response related to a mineralogical phenomenon (*22*) and is most appropriately reported in terms of residual effective strength parameters (c_r' and ϕ_r').

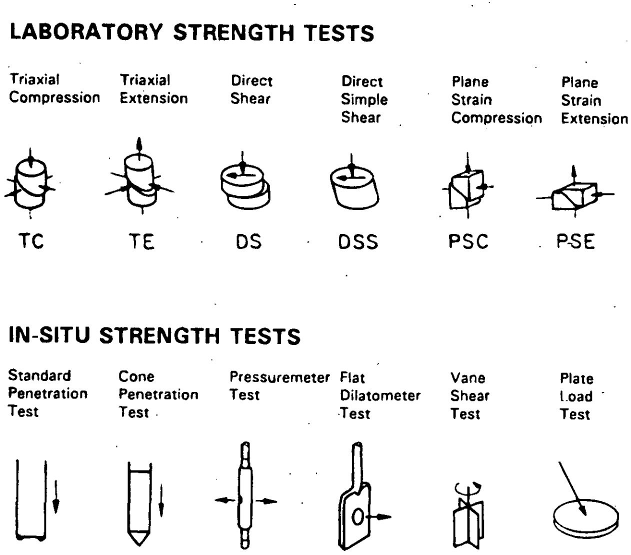

FIGURE 1 Types of laboratory and in situ strength tests.

SURVEY OF THE PRACTICE

To obtain an understanding of the current state of the practice in the sampling and testing of overconsolidated clays, two series of questionnaires were sent out by the Transportation Research Board Committee on Soil and Rock Properties in 1989 and 1992 to representative members of the geotechnical community. A total of 48 replies was received. Figure 2 shows the geographic locations of respondents, indicating that the results are generally representative of the U.S. practice. The West Coast, East Coast, southeastern, and midwestern states appear to be adequately represented, whereas the Mountain States and the Southwest may be slightly underrepresented. This also may reflect the paucity of overconsolidated clay deposits in those regions of the country.

The source categories of the survey respondents are given in Figure 3. Approximately 83 percent of the responses were from practitioners, including both geotechnical consulting firms (48 percent) and state highway departments (35 percent). A few additional replies were returned by academic institutions (12 percent) and representatives of FHWA (4 percent).

Figure 4 summarizes the typical problems encountered in characterization, analysis, and construction in overconsolidated clays. Almost 50 percent of the respondents reported that slope stability was paramount. The construction of spread footings, slabs, and pavements on expansive clays was a common problem 35 percent of the time, and apparent difficulties with the analysis and construction of deep foundations in overconsolidated, fissured clays, or both were reported by about 20 percent of the community.

The geotechnical profession uses a variety of different techniques for sampling, laboratory testing, and field measurements to assist in the evaluation of stiff to hard clay deposits. Figure 5 shows that across 90 percent of the country, both hydraulically pushed thin-walled tube and driven split-barrel (split-spoon) sampling methods are commonly used in these materials. Rotary techniques including both Denison and Osterberg samplers are used at about 47 percent of the locations. Once samples have been retrieved and transported to the laboratory, the consolidated-undrained (CU) triaxial compression test is the most often chosen method as a regular test (70 percent) to determine the shear strength of overconsolidated clays (Figure 6). The CU triaxial test is probably chosen because it provides the stress-strain response and assessments of both undrained strength ($\tau_f = s_u$) and the effective stress strength parameters (c' and ϕ') if pore pressure measurements are taken.

Approximately 45 percent of all the respondents use unconfined compression (UC) and unconsolidated-undrained (UU) triaxial compression tests for assessing τ_f. However, neither test provides effective confinement to clay specimens before shearing to failure. Therefore, the UC and UU tests do not simulate the geostatic stress state of the deposit. Ladd (*23*) discourages the common practice of using UC and UU testing because of the uncontrolled effects caused by sampling disturbance, high strain rates, and lack of appropriate effective confining stresses.

A significant number (40 percent) of laboratories use drained direct shear box (DS) tests on clay specimens to determine effective stress strength parameters (c' and ϕ'). A few laboratories also use repeated DS tests for evaluating residual parameters (c_r' and ϕ_r'), although the fully mobilized residual strength is probably not realized unless a ring shear apparatus is used (which apparently none of the respondents regularly use).

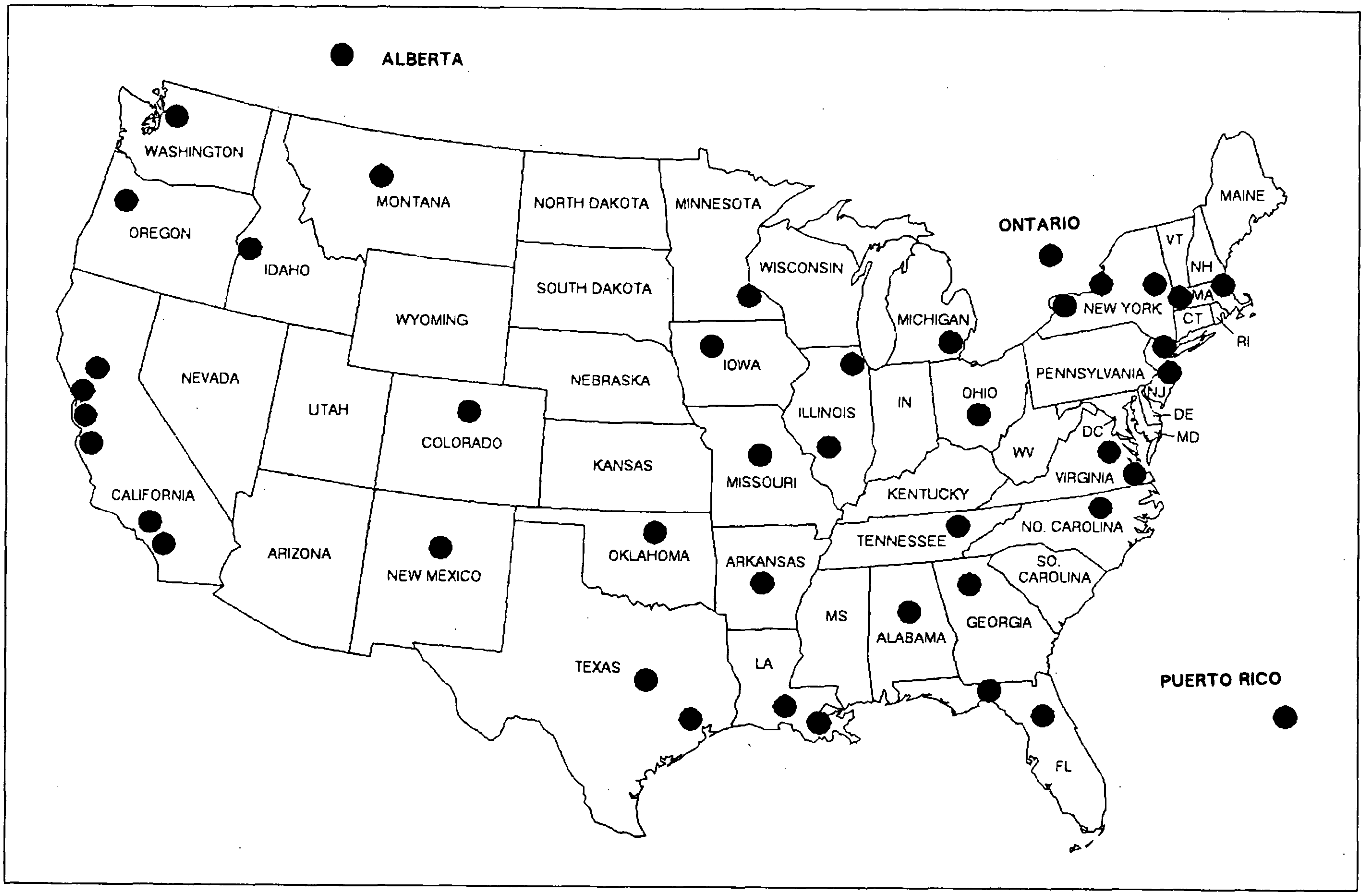

FIGURE 2 Location of survey questionnaire respondents.

The recent increase in the use of in situ testing methods is evident from the survey results. Figure 7 indicates that although the standard penetration test (SPT) still dominates U.S. practice, the cone penetration test (CPT) and pressuremeter test (PMT) appear to be gaining acceptance by practicing engineers. The dilatometer test (DMT) is also being increasingly used in practice as well. The use of in situ devices for evaluating clay properties on actual engineering projects has been frequently documented (*3,5,6*).

RECOMMENDATIONS FOR STRENGTH MODE SELECTION

For use in geotechnical practice, the following guidelines are suggested for selecting intact (undrained versus drained) versus residual strengths of clays in stability analyses. The short-term undrained shear strength ($\tau_f = s_u$) is the critical mode for designs involving soft clays in which the overconsolidation ratio ($OCR = \sigma_p'/\sigma_{vo}'$) is gen-

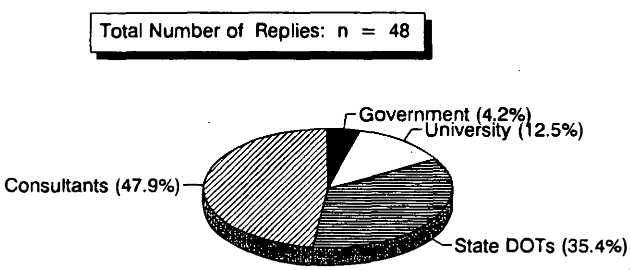

FIGURE 3 Occupational category grouping of survey respondents.

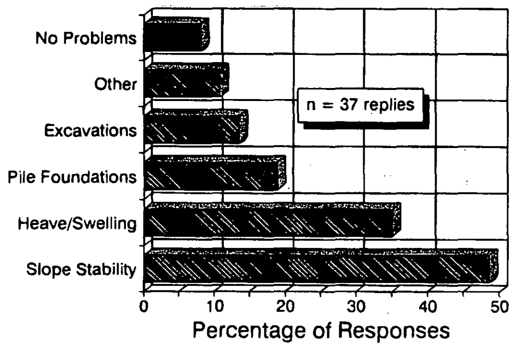

FIGURE 4 Problems facing practicing geotechnical engineers in overconsolidated clays.

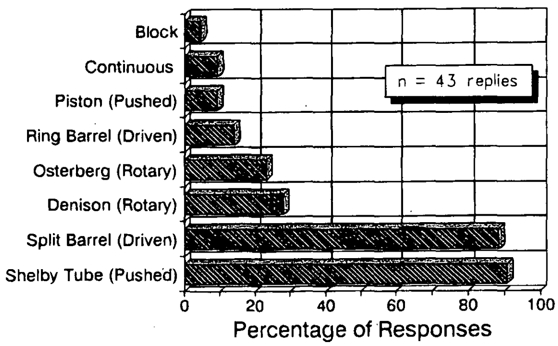

FIGURE 5 Sampling methods used in overconsolidated clays.

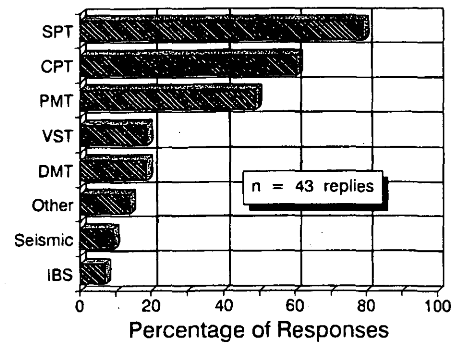

FIGURE 7 In situ testing practices in overconsolidated clays (SPT = standard penetration test, CPT = cone penetration test, PMT = pressuremeter test, VST = vane shear test, DMT = dilatometer test).

erally less than 4. For axisymmetric loading conditions, it is best determined as the average of undrained triaxial compression, direct simple shear, and triaxial extension tests measured on high-quality specimens (*24*). For long embankments, retaining walls, or continuous footings, plane strain tests are more appropriate. Unfortunately, these types of laboratory testing are normally beyond the ability or budget of the commercial or state laboratory. The use of only routine triaxial compression tests is unconservative because vertical loading is the strongest (*23*). In fact, the direct simple shear appears to give a reasonable average strength (*25*). Therefore, for nonorganic clays, undrained strengths may be best evaluated using the interpreted OCRs from conventional oedometer tests via (*10,17,23,25*)

$$(s_u/\sigma'_{vo})_{\mathrm{DSS}} = (0.23 \pm 0.04)\ OCR^{0.8}$$

where s_u/σ_{vo}' is the normalized undrained strength ratio. For OCRs generally greater than 4, the long-term drained strength is usually the most critical condition for slope stability analyses. If uncertainty exists as to which mode (undrained or drained) is critical, both analyses should be investigated.

Many commercially available computer stability packages adopt a Mohr-Coulomb failure criterion and thus

$$\tau = c' + \sigma'_v \tan\phi'$$

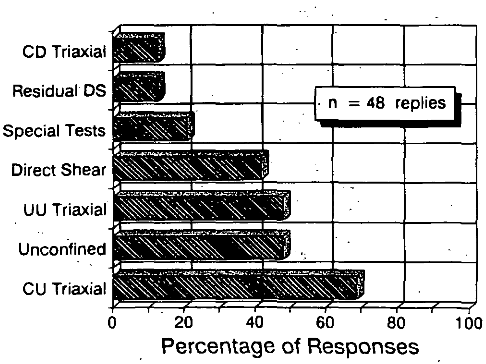

FIGURE 6 Laboratory strength test methods for stiff to hard clays.

where c' and ϕ' are determined by consolidated-undrained triaxial (with pore pressures measured), consolidated-drained triaxial, or drained direct shear box tests over a range of effective confining stresses. Conservative results should apply to the selected value of c' because poor saturation procedures, excessive strain rates, and inadequate back pressures may cause unduly high c' parameters. The best recommended practice is to adopt $c' = 0$ for effective stress analysis (*26*). If necessary, however, a small but assumed value of $c' \approx 0.02\sigma'_p$ may be appropriate (neglecting stress level dependency) on the basis of the extensive review of back-analyzed slopes by Mesri and Abdel-Ghaffar (*27*). The effective friction angle (ϕ') of clay should not be estimated from plasticity charts alone. Measured ϕ' values of natural clays worldwide (*28*) fall within the range 17 degrees $\leq \phi' \leq$ 43 degrees, and values outside this range should be further tested and verified or considered suspect.

Clays that are very heavily overconsolidated ($OCR > 30+$) are also most often fissured, and therefore no longer behave as a continuum. Passive failure, resulting from K_o-values reaching K_p during extensive unloading or desiccation cracking, slickenside features, faulting, and additional factors have occurred in most deposits of highly overconsolidated clays. Residual strength parameters may be appropriate for analysis and design, although the use of such values will result in very flat slopes. Similarities with the famous fissured London clay (*29*) are found in overconsolidated clays and clay shales throughout the United States and Canada (*7,30*). The residual strength is basically a frictional characteristic of the clay mineralogy and, c'_r is usually very close to zero (*22,31*). Skempton (*32*) has discussed in detail the residual strength of cohesive soils and its relevance to landsliding and stability problems.

RECENT TRENDS

Developments in sampling, laboratory testing, and field measurement devices offer improved characterization of stiff natural clays. With regard to sampling issues, Holm and Holtz (*33*) and Lacasse et al. (*34*) discuss the use of larger tube, piston, and block sampling techniques for providing quality laboratory specimens. A sample disturbance will often result in e-log σ'_v curves that plot below the true field curve and consequently underestimate the yield stress (σ'_p) of clays in routine consolidation testing (*8*). To minimize swelling of stiff clays in the oedometer, the Norwegian Geotechnical Institute recommends the use of dry filter stones (*9*). The procedures in this regard recommended by ASTM D-2435 (*35*) for incremental consolidation tests are satisfactory.

Internal measurements of strain within the triaxial chamber are preferred so that the true nonlinearity of the stress-strain-strength curve can be fully appreciated (*36*). Measured soil stiffness from laboratory specimens now agrees more closely with observed field measurements and back-calculated equivalent moduli from full-scale performance data (*37*). However, internal strain measurements are possible only at research laboratories at the present, and most commercial and government laboratories will continue to measure deformations outside the triaxial cells.

A variety of different laboratory devices (a resonant column, bender elements, and internal strain measurements), as well as improved field measurements, have been used for the measurement of low-strain shear modulus (G_{max}). This is an important and fundamental engineering property that can be useful in the characterization of all types of civil engineering materials (soil, rock, steel, concrete, etc.). Several commercial laboratories use resonant column testing for this purpose, and the growth of nondestructive testing using geophysical methods (cross hole, down hole, and spectral analysis of surface waves) permits an evaluation of G_{max} in the field.

Additional improvements in the laboratory assessment of clay behavior have been made through extensive stress path testing to define the full three-dimensional locus of yield surfaces (*13,17,21,28*). This test provides a complementary effective stress interrelationship between ϕ', c', s_u, K_o, σ_p', and stress state variables as a function of time. Complementary to these findings is a recent thorough study of more than 60 back-calculated slope stability failures, which quantifies the stress-dependency effects on the effective cohesion intercept (c') and illustrates that the magnitude of c' also depends significantly on stress history (*27*). This latter study showed that the normalized ratio of c'/σ_p' falls within the range of 0.02 to 0.10 and depends on the confining stress level.

A variety of field and in situ tests also have been introduced (*10,20,38*). Figure 8 shows the conceptual chronologic progress of evolution of the science of geotechnical engineering (*39*). The role of in situ testing has increased because of more detailed stratigraphy, better economy, and more immediate results provided by these new tools, especially the electric cone, piezocone, dilatometer, and geophysical techniques. Recent methods such as spectral analysis of surface waves for obtaining profiles of G_{max} are of interest because they are noninvasive and conducted at the ground surface. Improved means of interpreting the results of in situ tests also have been developed, and these tests now can be used to provide initial estimates of the degree of overconsolidation of a particular clay deposit (*12,13,20,38*). A theoretical assessment of the effective stress parameters (c' and ϕ') of clay deposits from piezocone test results has been proposed as well (*40*).

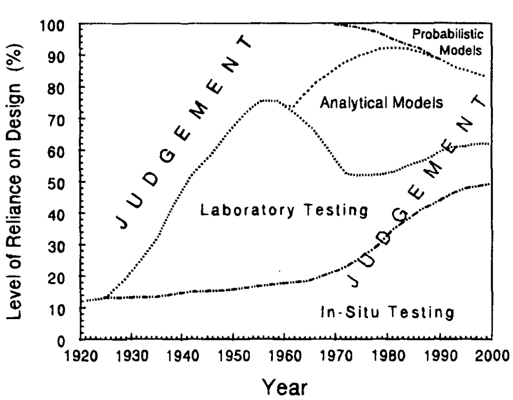

FIGURE 8 Evolution of selection of engineering design parameters for clays [modified after Lacasse (*39*)].

A number of hybrid devices also have emerged that optimize data collection and benefit from several techniques at the same time. Such devices include the come pressuremeter (*41*) and seismic piezocone (*42*), which provide independent measurements of failure stresses as well as soil stiffness at either low-strain G_{max} or intermediate-strain levels in the ground. Further research, validation, and applied technology programs may prove these to be the routine tools of the geotechnical engineer in the 21st century.

SUMMARY

Current means of sampling and testing stiff to hard natural clays in geotechnical practice are reviewed via the results of a survey questionnaire, with responses coming from all parts of the United States. Principal difficulties with civil engineering projects situated in overconsolidated clay occur in the proper characterization and evaluation of the material properties for analyses involving slope stability, expansive subgrades, and deep foundation systems. Observed trends in practice show increased use of more reliable laboratory tests (consolidated-undrained triaxial type and internal strain measurement systems), as well as the implementation of in situ tests (cone, piezocone, pressuremeter, and dilatometer) for assessing the strength and deformational characteristics of natural overconsolidated clays.

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Issues and Techniques for Sampling Overconsolidated Clays

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Obtaining high-quality samples in stiff, overconsolidated clay requires attention to many key issues. The sampling techniques used along the Louisiana and Texas Gulf Coast are compared with those found in the state-of-the art literature. Many of these conventional methods do not strictly follow recommended standards such as those presented by ASTM. However, evidence in the literature is sufficient to support several differences. The effects of sample size, extrusion, packaging, and storage on the engineering properties of stiff, overconsolidated clays found in Louisiana are summarized.

Stiff, overconsolidated clays are common in the United States and elsewhere. Their characteristic features often include a network of fissures caused by desiccation and other postdepositional occurrences. Many of the practices and methodologies used to sample soft clays do not work particularly well in stiff, overconsolidated clays. The anomalies of the soil structure and the effects of sampling techniques can make it difficult to determine their engineering properties.

The two most common concerns in geotechnical investigations involving overconsolidated clays are the occurrence of fissures and swell potential (*1*). The stiff, overconsolidated clays found in the Louisiana-Texas Gulf Coast are known for their heterogeneous characteristics and complex soil structure. Joints, fissures, silt and sand seams, root holes, and other irregular features are common characteristics of the soil macrostructure. These features can greatly affect the strength and drainage characteristics of the soil mass.

This paper's objectives are to

- Review the effects of conventional, undisturbed sampling techniques on stiff, overconsolidated clays;
- Present the authors' Louisiana and Texas Gulf Coast sampling experiences and the results of other sampling investigations in similar soils; and
- Identify the methods commonly used and some of the issues and concerns in the sampling of overconsolidated clays.

SAMPLING METHODS: STATE OF THE ART VERSUS PRACTICE

Geotechnical literature contains many references to soil sampling practices. However, sampling's influence on measured soil properties is still an issue that must be addressed. The following section reviews the state-of-the-art information and compares it with the commonly used sampling practices in the Louisiana-Texas Gulf Coast.

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Literature: State of the Art

Hvorslev's (*2*) work is one of the earliest studies to evaluate sampling. Since then, numerous studies have been completed that continue to support and update Hvorslev's information.

Borehole Drilling

Sampling activities normally consist of a combination of borehole advancement and sampling. The borehole is usually advanced with an auger or by rotary drilling methods. Auger borings are commonly used in soils of medium to stiff consistency and sands above the water table. Rotary drilling methods are used in all soils and rock.

In the rotary drilling process, the drilling fluid, circulated down the drill pipe, serves two purposes: (a) it seals and stabilizes the borehole, and (b) it carries the soil cuttings in suspension to the surface. The borehole's advancement and penetration rate depend on the material being penetrated. To remove cuttings in a clay soil, they must be small enough to be carried to the surface by the drilling fluid. Improper cleaning of the borehole can result in inadequate removal of soil cuttings, leading to accumulation of cuttings at the bottom of the borehole and in the top portions of the sample. In clay soils, studies conducted by the Bureau of Reclamation (*3*) found that it was effective to use a bit rotation of 200 to 300 rpm with a penetration rate of 25 to 50 mm (1 to 2 in.) per minute.

Soil Samplers

The thin-walled tube sampler (ASTM D 1587) is used most often in obtaining undisturbed samples in clays with consistencies that range from soft to very stiff (*1,4,5*). This method is generally considered to produce a quality sample that is suitable for quantitative testing. Hvorslev (*2*) arbitrarily identified a thin-walled sampler as one whose area ratio does not exceed 10 percent. Area ratio is a comparison of the projected cross-sectional area of the sampler with that of the soil specimen. The ASTM tube's cutting edge is sharp and crimped to a smaller diameter to allow an inside clearance. However, a large inside clearance has also been found to be detrimental with respect to swelling and expansion in fissured soils (*6,7*).

The sample's length and degree of disturbance is influenced by the speed and continuity of motion with which the sampler is forced into the soil. Hvorslev (*2*) recommended fast pushing with a uniform and uninterrupted advance of 0.15 to 0.31 m/sec (0.5 to 1.0 ft/sec). However, in measuring the forces developed during sampling with an open-drive sampler in stiff clay, Lang (*8*) used a penetration rate of 0.02 to 0.05 m/sec (0.06 to 0.18 ft/sec).

As noted above, thin-walled push samplers can be used over a fairly wide range of soil consistencies. However, the penetration resistance of a very stiff to hard clay may be too great for such samplers, which can be damaged in the process. Solutions are to increase the tube's wall thickness or use a double-tube core barrel sampler.

Double-tube core barrel samplers are designed to recover samples from formations that are too hard or brittle for thin-walled tube samplers. Core drilling differs from push sampling in that sampling and borehole advancement occur simultaneously. A stationary sampling tube located inside the rotating cutter barrel contains the sample. The cuttings are removed by circulating drilling fluid or air. Table 1 provides guidelines for selecting the sampler type, drive length, and inside clearance for different clays (*3*).

Sample Size

Lo et al. (*9,10*) found that for a stiff, fissured clay, the effect of sample size was the single most important factor in influencing the shear behavior. Rowe (*11*) emphasized the importance of the natural soil fabric as a guide for site investigations in the selection of the quality and size of specimens and boring technique (Table 2). To achieve relevant laboratory test results for clay soils that exhibit layered fabrics (varves, silt, and organic inclusions or fissures), large-diameter specimens are needed. Hand-cut or large block samples are generally considered to provide the least disturbed sample.

Practice: Typical Gulf Coast Procedures

Along the Gulf Coast, most of the sampling of cohesive soils is done using a 75-mm (3-in.) diameter thin-walled tube sampler that is pushed into the soil. However, the standard thin-walled tube and borehole advancement methods may differ somewhat from those specified in ASTM 1587.

In general, most geotechnical drilling operations consist of advancing the borehole using a combination of dry-auger and wet-rotary drilling methods. The dry-auger method is used to advance the borehole to the depth where water is encountered. After obtaining water level readings, the driller continues the drilling using wet-rotary methods.

In wet-rotary drilling, the borehole is usually advanced to the desired depth by using a bottom-discharge bit. The driller typically obtains soil samples at about 0.6-m (2-ft) intervals to about 3-m (10-ft) depth and at 1.5-m (5-ft) intervals below 3 m (10 ft). When the desired sampling depth is reached, the driller obtains undisturbed samples by hydraulically pushing a 76-mm (3-in.) diameter thin-walled tube sampler about 0.6 m (24 in.) into the soil. Many commercial firms extract the tube from the borehole without rotation.

TABLE 1 Sample and Sampling Procedures for Overconsolidated Clays (*3*)

	Soil Type	Clay & Shale	OC Clays		Expansive Clay
	Moisture Condition	Dry to Saturated	Moist	Saturated	Wet to Saturated
	Soil Consistency	Hard	Firm	Firm	
SAMPLERS	Open-Drive Samplers	Not Suitable			
	Bit Clearance Percent of Tube Diameter		½ to 1	0 to 1	½ to 1
	Drive Length, cm		46	46 to 61	46 to 61
	Recovery		Good	Good	Good
	Fixed-Piston Samplers	Not Suitable			
	Bit Clearance Percent of Tube Diameter		½	½	½ to 1-1/2
	Drive Length, cm		61	61	46 to 61
	Recovery		Good	Good	Good
	Double-Tube Samplers without Core Catchers				
	Recovery	Fair to Good	Good	Good	Good

TABLE 2 Specimen Sizes (*11*)

EXCEPTIONS: DEPOSITS TOO { WEAK VARIABLE / STRONG STONY }

CLAY TYPE	MACRO FABRIC	MASS k m/s	PARAMETER SPECIMEN	IZE, mm*
NON-FISSURED SENSITIVITY < 5	NONE	10^{-10}	c_u, $c'\phi'$ m_v, c_v	37 76
	PEDAL, SILT, SAND LAYERS, INCLUSIONS. ORGANIC VEINS	10^{-9} -10^{-6}	c_u $c'\phi'$ m_v c_v	100-250 37 75 250
	SAND LAYERS 2mm 0.2 m SPACE	10^{-6} -10^{-5}	$c'\phi'$ m_v	37 75
SENSITIVITY > 5	CEMENTED WITH ANY ABOVE		c_u, $c'\phi'$, m_v, c_v	50-250 ‡
FISSURED †	PLAIN FISSURES	10^{-10}	c_u $c'\phi'$ m_v c_v	250 100 75
	SILT OR SAND FILLED FISSURES	10^{-9} -10^{-6}	c_u c_v $c'\phi'$ m_v	250 100 75
JOINTED	OPEN JOINTS		ϕ'	100
PRE-EXISTING SLIP			$c_r\phi_r$	150 OR REMOULDED

Minimum Sizes of Specimens from Quality I Thin Walled Piston Samples of Natural Clay Deposits. Foundations for Buildings, Bridges, Dams, Fills. Stability of Natural Slopes, Cuts Open or Retained.

* 75 mm samples for continuous Quality 2-4 samples for fabric examination, strength as index test, c_u and $c'\phi$ for intact low sensitivity.
† Size and orientation dependent on fissure geometry.
‡ Tube area ratio 4%, sample dia. 260 mm.

After recovery of the sampler, the logger removes the soil specimen in the field. The logger then examines the specimen, visually classifies it, and preserves representative portions of each specimen for laboratory testing. Sample preservation methods vary, but they typically consist of wrapping a portion of the undisturbed specimen in plastic wrap, aluminum foil or both. The wrapped specimen is then placed in a protective container for shipment to the laboratory. Often a disturbed but representative portion of the sample is placed in glass or plastic containers.

Variations from ASTM Procedures

The sampling methods used along the Gulf Coast have evolved over the years in response to subsurface conditions and acquired experience. Therefore, they differ somewhat from the ASTM methods. Some of these variations are:

- Advancing the borehole using a bottom-discharge bit rather than a side-discharge bit as required by ASTM;
- Use (by many commercial firms) of a thicker thin-walled tube with no internal clearance (a 14-gauge thin-walled tube is used by many firms in the Gulf Coast; in contrast, the ASTM thin-walled tube is a 16-gauge tube with an inside clearance ratio of 1 percent;
- Removing the tube from the ground without rotation (a common practice by many commercial firms); ASTM requires rotating the tube before it is extracted from the bottom of the borehole; and
- Extruding the samples in the field; ASTM procedures specify sealing the tube in the field with wax and shipping the tube to the laboratory for later sample extrusion.

Many of the variations from the ASTM guidelines are due to efforts to obtain better-quality samples than can be obtained by strictly following ASTM's guidelines. Other changes, however, are done for productivity and cost reasons.

The thicker tubes are used by some because of the strength of the overconsolidated clays, the presence of calcareous and ferrous nodules, and the presence of cemented zones. Experience with the ASTM tubes indicates that they can become crimped near the sampling end and they can become out-of-round. In some cases, the extraction of the tube or rotation process has caused the top of the ASTM tubes to shear off. Many of the Gulf Coast overconsolidated clays are expansive and contain slickensides and fissures. Studies suggest that inside clearance ratios in these soils may be detrimental (*6,7,12*).

Drilling with a side-discharge bit in stiff clay is slower than drilling with a bottom-discharge bit. One commercial firm with which the authors are familiar has estimated that drilling with side-discharge bits is about 20 to 30 percent slower than drilling with bottom-discharge bits. The primary reason for this is that side-discharge bits tend to plug in stiff clays because the cleaning action of the drilling fluid on the bit is reduced by the deflectors welded onto the bit.

Field extrusion and packaging are standard Gulf Coast practices; ASTM recommends performing these operations in the laboratory. There appear to be several good reasons for using field extrusion methods. One is cost considerations and the other reasons are technical. Field extrusion allows for the immediate reuse of the sampling tube. If the tube had to be extruded in the laboratory, many more tubes would be required at a significant cost. Also, a full sample tube is quite heavy and takes up more space than the extruded specimen. This would likely cause increases in the costs associated with sample shipping and handling. By extruding the sample in the field, the logger is able to visually examine it and if necessary make adjustments to the sampling program.

Laboratory Testing Methods

For most routine to moderately complex geotechnical projects involving overconsolidated clays, undrained shear strength is usually the most important engineering parameter. Soil compressibility, drained-strength parameters, and other properties usually are of less concern. Measurements of undrained shear strength usually consist of unconfined compression tests or unconsolidated-undrained triaxial compression tests on undisturbed samples. The confining pressure in the triaxial test is usually equal to or slightly more than the effective overburden pressure. The triaxial test is believed to give a better overall indication of the undrained strength, probably because of compensating errors. Some firms remold overconsolidated clays before they perform undrained strength tests. In slickensided or fissured clays, this procedure would eliminate the influence of the structure on measured strength.

SAMPLING ISSUES

Since 1965 the Louisiana Department of Transportation (LDOT) has conducted studies involving methods for site investigations (*4,13–15*). Louisiana soils were transported and deposited in waters during the early Tertiary and Quaternary periods. Depositional conditions involved marine, deltaic, and continental environments. The layering and thickness of the strata and type of sediments found are the result of sea-level fluctuations and local and regional structural movement. The types of cohesive soils range from soft organic silty clays to stiff fissured clays. The softer sediments often are Holocene (Recent) sediments, whereas the stiffer materials often are Pleistocene deposits. The stiff clays are generally weathered at the surface and weakened by a network of fissures and slickensides created through severe periods of desiccation (Figure 1).

Sampling Disturbance

In undrained strength tests it has been observed that sampling disturbance can produce two opposing outcomes with test specimens of stiff fissured clays. Tube sampling can partially remold the fissures and strengthen the test specimen. However, stress release is most critical for a stiff, fissured clay. Removal of the confining support and the sampling activities can cause the fissures to separate and weaken the soil.

Excessive strains that occur with sampling can also have a profound effect on the engineering properties of nonfissured, overconsolidated soils. A comparison of the strength determined with specimens of varying quality demonstrated just the opposite of the strength found for the stiff fissured clays (*16*). Hand-cut blocks and larger-diameter tube samples of overconsolidated clays with a medium consistency provide a better representation of in situ conditions (Figure 2).

Stress Release

Removal of the sample from the ground reduces the total stresses to zero. As the sample attempts to rebound or dilate, a pore pressure less than atmospheric is developed in saturated soils. In sampling an overconsolidated clay sample under ideal conditions, the resulting effective stress, σ'_{ps}, for the "perfect sample" is (*18*)

$$\sigma'_{ps} = \sigma'_{vo}\,[1 + A(K_o - 1)] \tag{1}$$

where

σ'_{vo} = effective overburden stress,
A = pore pressure parameter, and
K_o = coefficient of lateral earth pressure at rest.

Under normal field sampling and laboratory conditions, the actual residual effective stress, σ'_r, has been found to be significantly smaller than σ'_{ps}. The above equation assumes a continuous soil specimen and probably does not accurately portray conditions for a fissured specimen. However, without fissures, the change in stress has little effect on the undrained strength of a saturated clay if there are no changes in moisture content or mechanical damage to the test specimen (*17*).

Disturbance of Soil Structure

Most sampling disturbance is attributed to changes in the soil structure, that is, strain. The soil structure is disturbed in the top and bottom portions of the sample during the boring activities and separation

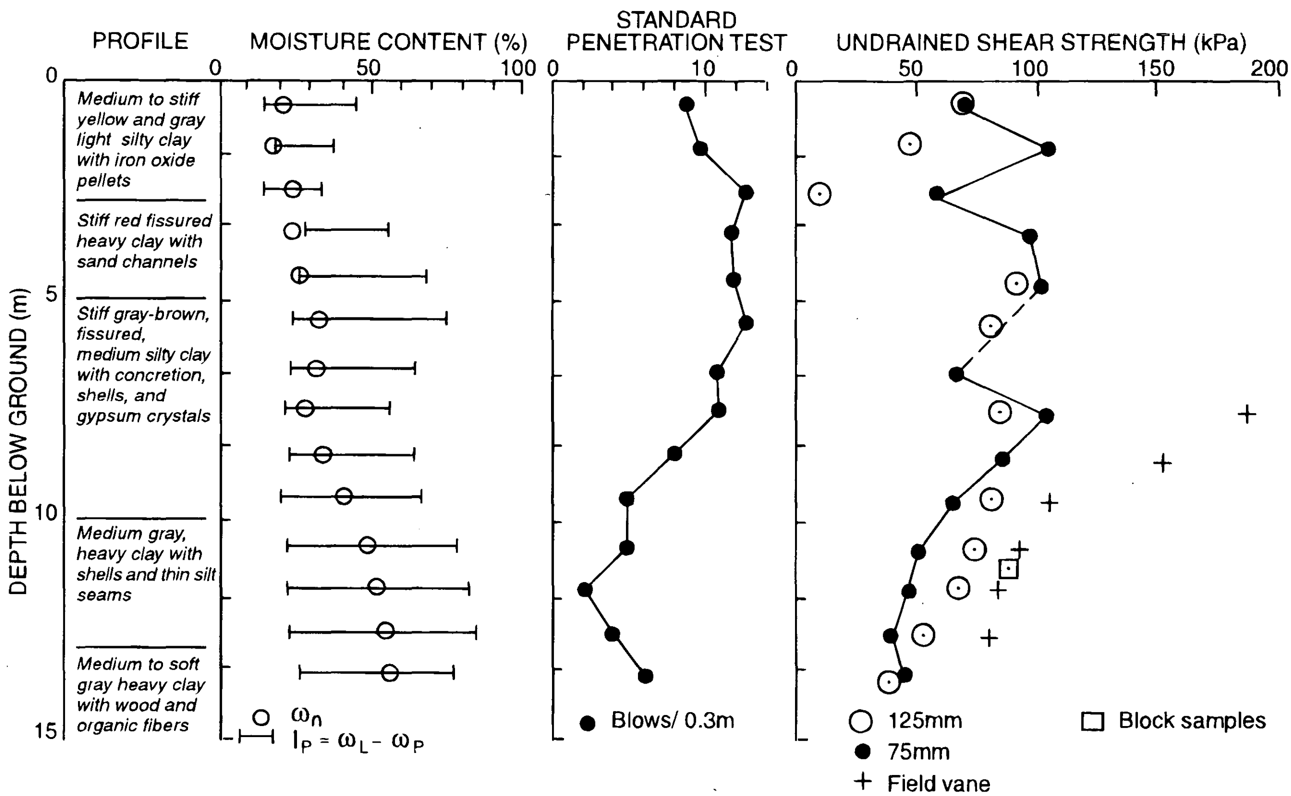

FIGURE 1 Typical soil profile, southwest Louisiana.

from the parent material with sampling. Structural disturbances may also result from friction turn down at the sample edge, planes of failure, and distortion or changes in thickness of soil layers (*2,18*). Internal and external friction between the soil and the sampler is a major source of structural disturbance, producing variations in strength along the sample's length and cross section. Lang (*19*) concluded that the specimen strength and deformation measured from the lower portion of the sample of a stiff, residual clay taken with an auger core soil sampler was more representative of the natural soil.

Baligh (*20*) used the strain path method to evaluate the strain history of Boston blue clay during sampling of a normally consolidated and overconsolidated (OCR = 4) sample. The resulting implications were that, even under ideal conditions, thin-walled sampling caused serious disturbances in overconsolidated specimens and unacceptable disturbance in normally consolidated specimens.

Test Scatter

A comparison of unconsolidated-undrained strength test results demonstrates the scatter typical of a fissured soil (Figure 3). As noted in Figure 3, the tests reported include undisturbed, remolded, and driven samples. The undisturbed specimens were tested within 14 days of sampling or subjected to extended storage periods of 30 days and longer. Much of the scatter in the undisturbed samples can be attributed to the extended storage time. However, the major cause of the test scatter is the frequency and presence of fissures in a particular test specimen.

The deformed shape of the failed specimen in many of the tests followed the irregular orientation of the fissure geometry. A number of the larger samples (125-mm diameter cores and hand-cut blocks) failed when attempts were made to trim them. The analysis of only those test results plotted in Figure 3 is probably biased, since they do not include those 125-mm diameter samples that failed before testing. A few of the 75-mm diameter samples became fragmented while handling, but most of these had long storage periods. Smearing on the periphery of the 75-mm diameter samples hid discontinuities from visual inspection and provided artificial bonding across fissures. Trimming the periphery and removing the remolded portion of the larger samples (125-mm diameter) caused many samples to fail before they could be tested.

Complete remolding seems to generally produce higher values of undrained strength for individual samples. Sensitivity of the test specimens ranged from 0.6 to 1.4. In most of these tests, the remolded strength equaled or exceeded the undisturbed strength.

Sampling with a split-barrel (split-spoon) sampler in the standard penetration test (SPT) (ASTM D 1586) also remolds, compacts, and bonds fissures to produce a stronger, more uniform specimen. The measured undrained strength of the SPT specimens has the highest range of all sample types.

X-Ray Radiography

ASTM methods for x-ray radiography of soil samples (D 4452) are particularly valuable in determining the quality of undisturbed soil samples to be selected for critical testing (*18,21*). X-ray radiography provides many benefits, including identifying the features outlined below:

- Heterogeneity and distribution of anomalous features;
- Internal failure modes of soil specimens, not routinely discernible to the naked eye;

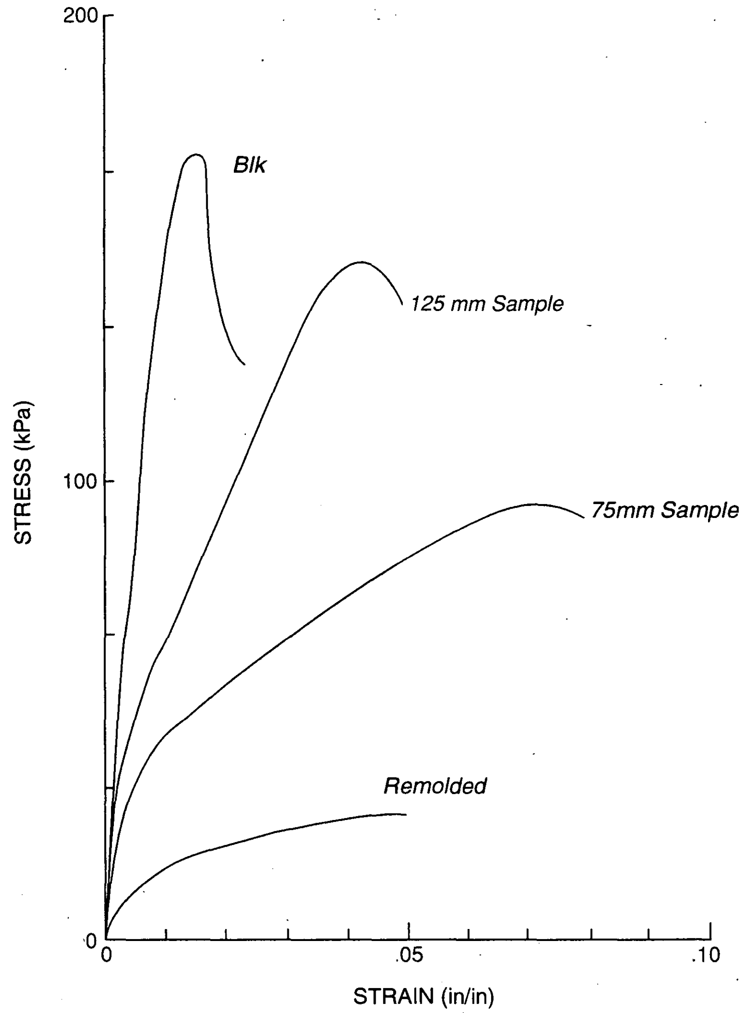

FIGURE 2 Typical stress/strain curves for triaxial undrained compression test.

- Naturally occurring cracks and those produced by sampling (i.e., failure planes, the additional separation of the existing laminations, and fissures during the sampling procedures).

Radiographs have also been useful for predicting the potential for fluid migration in the stiff clays typical to Louisiana and the Gulf Coastal Plain (*22*). A detailed X-ray radiographic analysis identified the presence of a permeable structure in the sediments, thus helping to support the interpretation of fluid movement through the low-permeability clays that contain these permeable conduits. The X-ray radiographs revealed a network of iron-lined fissures, concretions, and roots throughout the affected zone. X-ray radiography is a technique that can support the analysis and evaluation of test data for critical geotechnical projects.

Sample Type and Size

The need for larger samples and field testing has been cited (*9,23*). However, the selection of sampler type and size is still largely controlled by local practice. Lang (*8*) recommended that the trimmed surfaces be as small as possible in relation to the original size of the specimen. Figures 1 and 2 demonstrate the improved test performance achieved with better-quality samples of a medium-to-stiff, overconsolidated clay and the absence of fissures.

According to Rowe (*11*), the sample diameter appropriate for stiff, fissured clays ranges from 75 to 250 mm (3 to 10 in.) and depends on the engineering property of interest (Table 2). Figures 1 and 3 demonstrate a range of test results typical for desiccated soils and sample types. These variations are attributed to (a) the remolding of fissures in the smaller, more proportionally disturbed tube samples and (b) the frequency of fissure occurrence [i.e., the probability that less fissures occur in smaller samples (*21*)].

Extrusion Effects

Studies have shown that extruding the samples in the laboratory caused more disturbance than extruding in the field (*24,25*). In stiff clays, this disturbance is caused by overcoming the adhesion between the sample and the sample tube. The pressure required to extrude the sample often exceeds the unconfined compressive strength of the sample (*8,13,18*).

Pressurized water or hydraulic rams are the methods used to extrude soil cores from sampling tubes in the field. It is generally conceded that there are many negatives in extruding the soil using

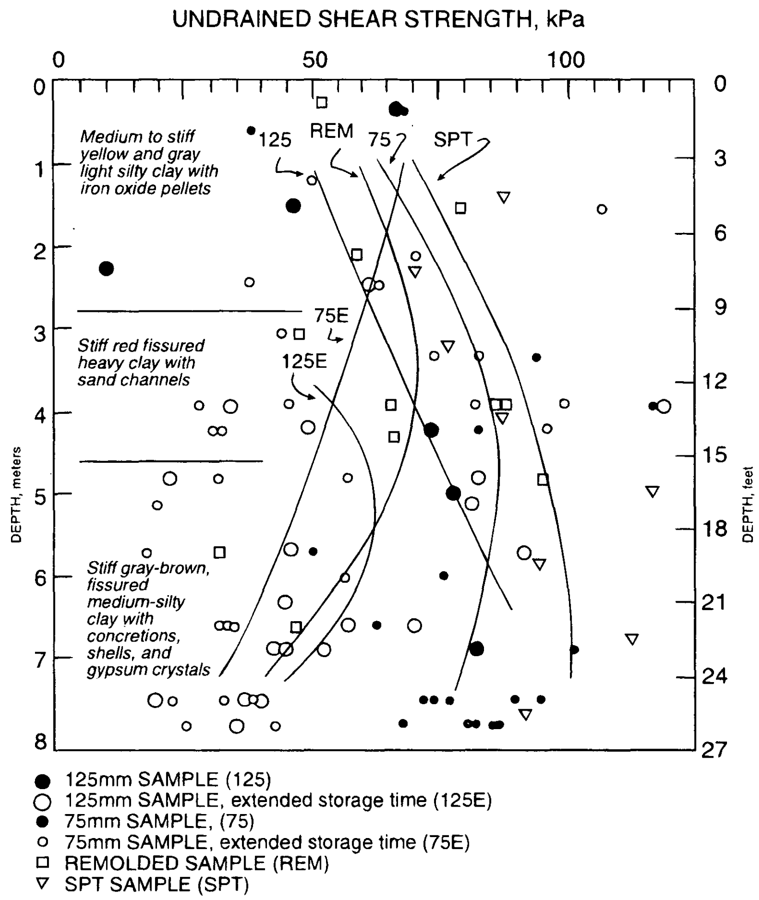

FIGURE 3 Typical fissured soil tests.

the pressurized system of the circulating fluid, and this method is not recommended. LDOTD conducted a study on the effects of extrusion using the hydraulic ram (*4*).

Replicate samples were obtained with a 75-mm (3-in.) diameter thin-walled tube from overconsolidated clay formations. Some cores were extruded in the field and placed in protective packaging. Others were sealed in the tubes. All samples were promptly transported to the laboratory.

In all cases, the applied stress during laboratory extrusion of the sampler exceeded the unconfined compressive strength of the soil by as much as 900 percent. The maximum strain measured before the sample began to move in the tube was about 0.5 percent during extrusion. Sone (*12*) evaluated the extrusion of a silty clay. The pressure to extrude the soil was several times larger than the unconfined strength and produced a compressive strain of 1 percent. The driving forces measured by Lang (*8*) while sampling a stiff clay exceeded the shear strength by 13.4 times.

Sample Protection

Sample packaging in the field is required when field extrusion of the samples occurs. Traditionally, many geotechnical groups used paraffin wax as the sealant. Laboratory observations indicated that even experienced and careful technicians had difficulties when removing the hardened paraffin without damage to the samples. Also noted was sweating under the paraffin. A sample from a depth of 12 m (40 ft) has a body temperature of about 18°C (65°F). When it is wrapped in foil and dipped in hot paraffin (49°C), considerable sweating results, with an increase of moisture content in a zone adjacent to the outer surface of the sample. Another study known to the authors showed that waxed clay samples had temperatures 12° to 16°C higher than ambient air temperature 4 to 5 hr after the samples were sealed with wax. Therefore, it seems reasonable to expect heat to cause redistribution of moisture within a sample. The potential sudden effects of water migration on the pore pressure and moisture distribution, as well as the homogeneity of the soil, are undesirable. Water entering the cracks of a fissured clay causes it to soften and swell unequally and results in further destruction of the sample.

More recently, other forms of sample packaging have become popular. In general, the newer packaging consists of wrapping the sample in plastic, aluminum foil, or both, and placing the wrapped sample in a sample container.

In the LDOTD study (*4*), two methods of protective packaging for field-extruded cores were reviewed with respect to moisture content variations. The coating of undisturbed samples with paraffin had been accepted as one method of preserving sample integrity. A second technique uses aluminum foil and plastic film as a protective coating. The wrapped specimens are then placed in cylindrical-formed styrofoam boxes to secure the specimen for shipment and storage.

The natural moisture contents of sampled specimens were determined immediately upon their arrival in the laboratory. Both the paraffin-coated and the foil/plastic-wrapped specimens were stored at 100 percent humidity and at 22°C (72°F). After storage periods that ranged from 14 to about 30 days, the two specimens of a set were tested for moisture content from each whole section. The results indicated that the foil/plastic wrapping maintained the moisture content of a specimen just as well as the paraffin coating.

After these tests, the Louisiana Department of Highways adopted the use of foil/plastic protection instead of paraffin. Some commercial firms now use only single plastic bags for sample storage. However, reported moisture loss using this method was found by others to be significant (*26*).

Sample Storage

The quality of stiff, fissured clay samples deteriorates rapidly with extended storage time. Removal of the confining pressure through sampling permits the clay to expand, and fissures to open. Many stiff, fissured cores that were stored over longer time periods became fragmented when handled, including hand-cut block samples. The combined negative effects of stress release and storage time were much more severe for the stiff, fissured clays than for other soils.

The effects of long storage on the strength and consolidation characteristics of a nonfissured to slightly fissured, overconsolidated clay of medium to stiff consistency were compared with three different types of samples: 300-mm (12-in.) wide hand-cut block samples, 125-mm (5-in.) diameter thin-walled tube samples, and 75-mm (3-in.) diameter thin-walled tube samples (Figure 4).

The results of the early (4 to 7 days) undrained triaxial shear strength tests on specimens from the 125-mm diameter cores were slightly lower than those from the large hand-cut blocks. Specimens from the 75-mm diameter cores had much lower strengths (Figure 4). Such effects were attributed to disturbances of the outer zones during tube sampling and core extrusion.

The remaining samples were stored at 22°C (72°F) and 100 percent humidity in field-applied packaging and were tested at the end of different time periods. Only samples with similar moisture contents, densities, and classifications were used. A small decrease in shear strengths was observed for tube-sampled specimens through the first 10 days of storage. However, the specimen strength of both the 75- and 125-mm diameter cores seemed to deteriorate at an increasing rate after the first 10 days (Figure 4). Extended storage times also caused a reduction of the measured preconsolidation pressure by as much as 30 percent for specimens from the tube cores.

Sampling and In Situ Testing

Using significantly higher-quality soil boring and sampling techniques often is cost-prohibitive for routine to moderately complex projects. However, in situ testing offers some opportunities to cost-effectively improve the quality of a subsurface investigation program in overconsolidated soils. For the stiff, overconsolidated clay soils typically encountered in the Louisiana-Texas Gulf Coast, in situ tests are useful (Figure 1). They can help to better define strength profiles and soil stratigraphy while reducing scatter in the data.

Since in situ tests do not collect soil samples, they need to be used in conjunction with traditional drilling and sampling methods. The data interpretation should be based on correlations with site-specific conditions. These tools also do not directly measure all routine geotechnical parameters such as moisture content, unit weight, soil compressibility, and plasticity. However, they can provide detailed information about subsurface conditions.

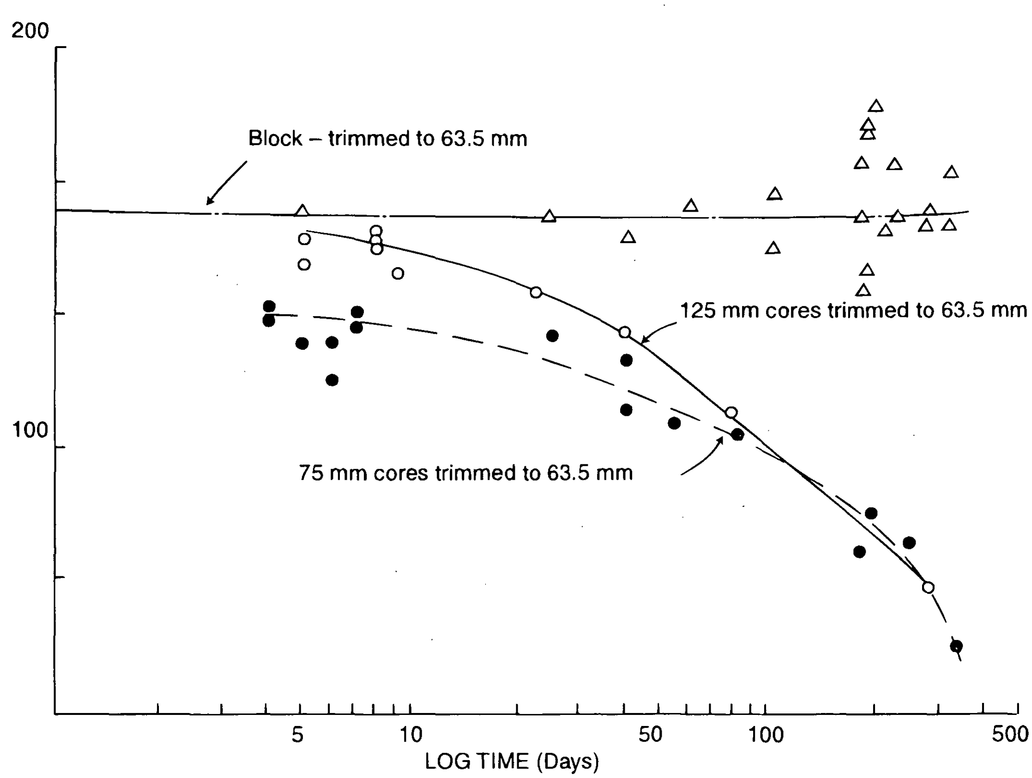

FIGURE 4 Storage time versus undrained triaxial strength.

CONCLUSIONS AND RECOMMENDATIONS

Stiff, overconsolidated clays present challenges for obtaining quality undisturbed samples. These soils require careful attention to sampling techniques as well as sample handling and storage practices. Each project often has its own special requirements that must be considered when developing a data-gathering program. This paper identifies many of the issues associated with sampling these soils, and it presents information about practices in the Louisiana-Texas Gulf Coast. It also summarizes the results of several assessment programs on Louisiana soils.

Most of the sampling disturbance in stiff, fissured clays is caused by the discontinuous structure (fissures) and stress release. Remolding of fissures (mechanical disturbance) produces misleading test information with higher undrained strengths and less permeable characteristics of the soil mass. However, a reduction in the sample quality of a nonfissured, medium to stiff clay specimen produces a reduction of strength. X-ray radiography has been demonstrated to be a useful tool for selecting representative specimens, evaluating sample quality, supporting the analysis of tests results, and reviewing the soil's fabric (macrostructure).

The wide variation in undrained strength tests conducted on stiff, fissured clays is largely influenced by the frequency and geometry of the fissures. Larger samples aid in evaluating the presence and frequency of fissures and their effects on the mass permeability and the shear mechanics of the soil mass.

In medium to stiff, overconsolidated clays, variation between sample types in the laboratory and field tests is attributed to sample quality and the extent of mechanical (sampling) disturbance. Larger, quality samples also provide better estimates of in situ performance of nonfissured, medium to stiff clays.

Extended storage causes reduction in strength (fissured and nonfissured). However, it appears to be most critical for fissured clays (i.e., prolonged removal of total stress).

Obtaining high-quality samples in stiff, overconsolidated clay requires attention to many key issues, including drilling procedures, selection of the proper sampling equipment, and careful extrusion, handling, packaging, and storage practices.

Some of the data-gathering practices used in the Gulf Coast do not strictly follow recommended standards such as those presented by ASTM. However, evidence in the literature is sufficient to support several differences. The inside clearance on sample tubes recommended by the standard procedures may be detrimental to sample quality of expansive and fissured clays. The thicker walls on the thin-walled tubes used by some organizations may result in less sample disturbance than what would occur with thinner-walled tubes recommended in certain standards. Field extrusion usually is not recommended by most standards. The standards usually indicate that sample extrusion should occur in the laboratory. However, studies show that laboratory extrusion causes more sample disturbance than extruding in the field. Some of the currently used sample packaging methods compare favorably with the traditional sample waxing methods for both short- and long-term sample storage. In addition, the newer packaging methods may actually reduce sample disturbance caused by heat and moisture migration.

Most standard geotechnical sampling uses 75-mm (3-in.) diameter samples. Evidence suggests that larger-diameter samples can improve sample quality. However, the field and laboratory costs associated with large-diameter samples can be prohibitive. As an alternative, in situ testing can be used to supplement a traditional soil boring program and cost-effectively improve the quality of data obtained. Although higher-quality studies are desirable, many foundation capacity procedures and other geotechnical analysis methods are empirical. Therefore, changes in data acquisition methods may require adjustments in the empirical procedures. However, the confidence level in the empirical procedures should increase if consistent and better-quality data are obtained.

ACKNOWLEDGMENTS

Much of the information utilized was conducted in research financed by the Louisiana Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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Laboratory Methods for Determining Engineering Properties of Overconsolidated Clays

D. J. DEGROOT AND T. C. SHEAHAN

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.

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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

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 (σ_p' − σ_{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.

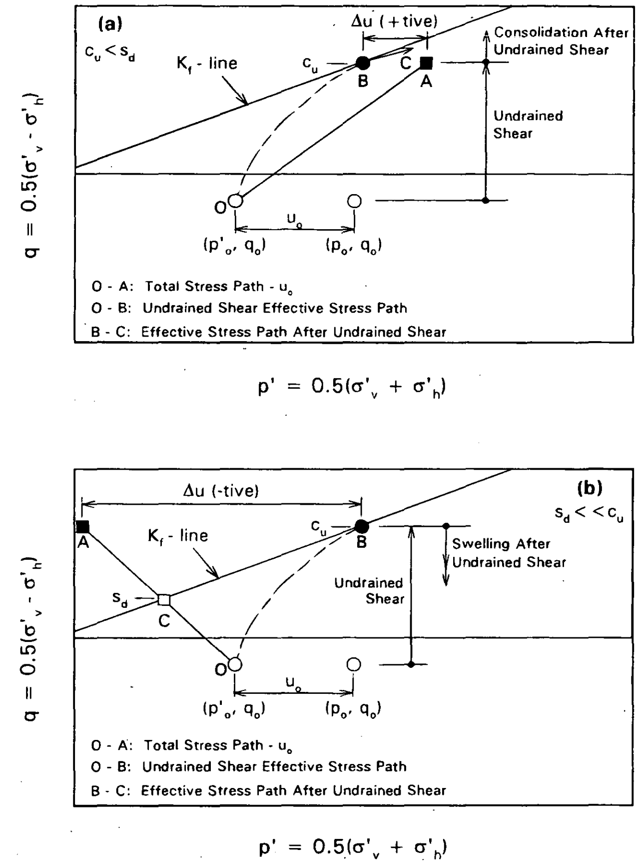

FIGURE 1 Stress paths for undrained shear followed by drainage of an OC clay (*Ko* > 1): *(a)* loading TSP; *(b)* unloading TSP.

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 $\sigma_{vc}' = 0.5$ 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

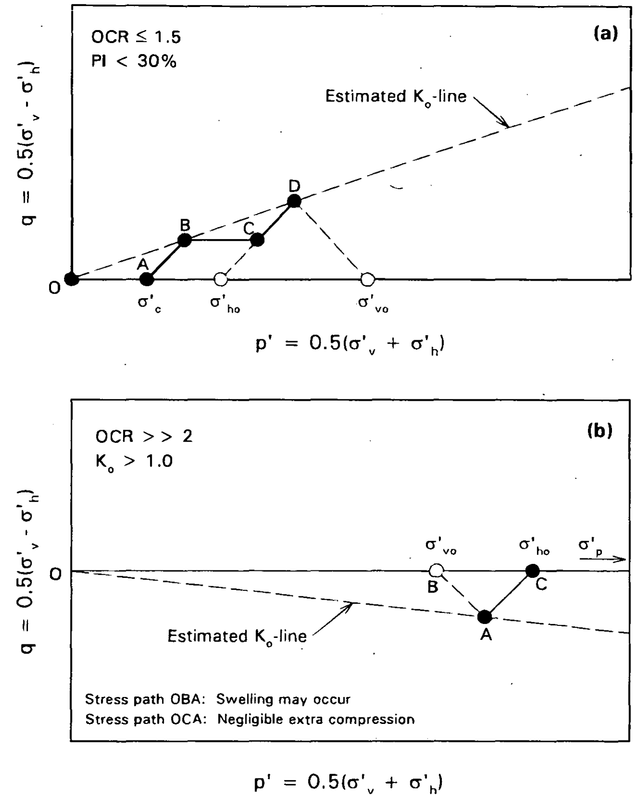

FIGURE 2 Simplified consolidation stress paths: *(a)* lightly overconsolidated clays; *(b)* highly overconsolidated clays *(28).*

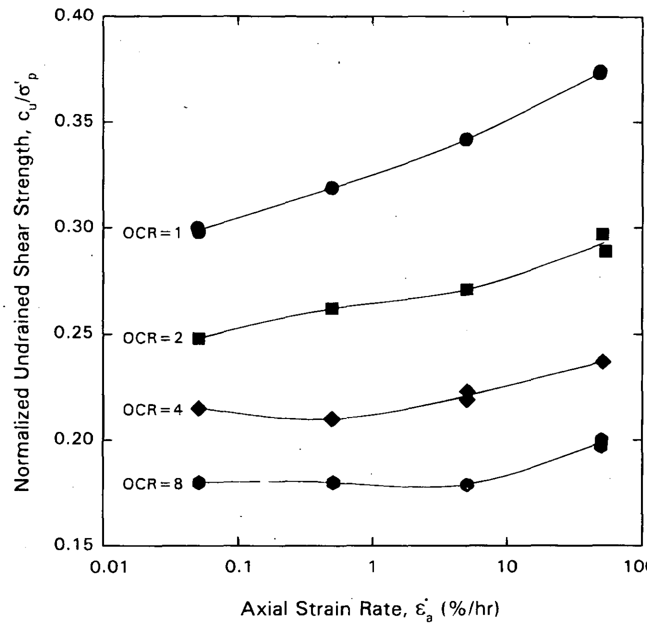

FIGURE 3 Normalized shear strength versus strain rate from SHANSEP CK_oUC tests on resedimented BBC *(35).*

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

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

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 $c_u(\text{TC}) > c_u(\text{DSS}) > c_u(\text{TE})$. Accounting for anisotropy, the average normalized c_u should approximate (*9*)

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

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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Measurement of Drained Residual Strength of Overconsolidated Clays

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The drained residual shear strength of overconsolidated clays is an important parameter in assessing the stability of slopes that contain a preexisting shear surface. The main issue influencing a laboratory testing program to measure the drained residual strength is whether a natural or laboratory-formed shear surface will be used. A multistage test procedure using a modified Bromhead ring shear apparatus and an overconsolidated, precut, remolded specimen is described that provides a reliable and practical method for measuring the drained residual shear strength. Results of ring shear tests on 32 clays and clay shales reveal that the drained residual strength is controlled by clay mineralogy and the quantity of clay-size particles. The liquid limit is used as an indicator of clay mineralogy, and the clay-size fraction indicates the quantity of clay-size particles, which are particles smaller than 0.002 mm. Therefore, increasing the liquid limit and clay-size fraction decreases the drained residual strength. The ring shear tests also reveal that the drained residual failure envelope is significantly nonlinear for overconsolidated clays with a clay-size fraction greater than 50 percent and a liquid limit between 60 and 220. Analysis of several case histories shows that this nonlinearity should be incorporated into a slope stability analysis. Previous correlations do not provide an accurate estimate of the drained residual strength because they (*a*) are based on only one soil index property, for example, clay-size fraction or plasticity; and (*b*) do not provide an estimate of the stress-dependent nature of the residual failure envelope. A new correlation is presented that is a function of the liquid limit, clay-size fraction, and effective normal stress and can be used to estimate the entire nonlinear residual failure envelope.

The concept of residual strength has contributed greatly toward understanding the long-term shearing resistance that can be mobilized in overconsolidated clay slopes. The drained residual shearing resistance can be significantly lower than the peak strength (Figure 1) and is a crucial parameter in evaluating the long-term stability of new and existing slopes and the design of remedial measures. Skempton (*1*) concluded that the drained postpeak strength loss observed in overconsolidated clays is caused by (*a*) an increase in water content because of dilation and (*b*) the orientation of clay particles parallel to the direction of shearing (Figure 1). In normally consolidated clays the drained postpeak strength loss is caused entirely by the orientation of clay particles parallel to the direction of shear. Large continuous shear displacements in one direction are required to orient the clay particles parallel to the shearing direction and to achieve a drained residual strength condition.

Skempton (*2*) concluded that slopes that have undergone 1 or 2 m of displacement should be designed using a drained residual strength. Therefore, the drained residual strength pertains to slopes that contain preexisting shear surfaces, such as old landslides or soliflucted slopes, in shears in folded strata, and sheared joints or faults (*3*). The residual strength also is applicable to failed embankments and the occurrence of progressive failure in slopes. Slopes that have not undergone previous sliding can be designed using the fully softened strength (*2*), which corresponds to the peak strength of a remolded normally consolidated clay (Figure 1).

Stark and Duncan (*4*) and Chandler (*5*) have described several landslide case histories involving overconsolidated clays in which previous sliding had not occurred, and the back-calculated friction angle is less than the fully softened value. These case histories suggest that there may be circumstances in which slopes that have not undergone previous sliding may require design strengths that are less than the fully softened value. One such circumstance is the repeated loading caused by the annual draw-down cycle of a reservoir. Stark and Duncan (*4*) concluded that the slide in the upstream slope of San Luis Dam was caused by the accumulation of shear displacement and associated strength loss induced in the overconsolidated foundation clay during the 14-year reservoir draw-down history. The possibility of mobilizing a strength less than the fully softened value in natural or man-made slopes that have not undergone previous sliding is currently being studied by the author.

MEASUREMENT OF DRAINED RESIDUAL STRENGTH

Laboratory measurement of the drained residual strength requires the use of a specimen that (*a*) contains a natural shear surface or (*b*) can be precut or presheared to form a shear surface. As a result, the main issue to be decided in planning a residual strength test program is whether the shear surface will be formed naturally or in the laboratory. The resulting laboratory testing procedure will be significantly different depending on the technique used to form the shear surface. Other important issues in laboratory testing are whether a direct shear or torsional ring shear apparatus will be used, and the use of a single-stage or multistage test procedure. In a multistage test, after a residual strength condition has been established under the first effective normal stress, shearing is stopped, and the normal stress is doubled. The specimen is allowed to reconsolidate under a higher normal stress before shearing is recommenced. This procedure is repeated for a number of effective normal stresses to estimate the drained residual failure envelope. In summary, the laboratory test procedure recommended to measure the drained residual strength will depend primarily on the type of shear surface (field versus laboratory), test apparatus, and test procedure.

Natural Shear Surfaces

Samples containing natural shear surfaces can be obtained from pits, shafts, tunnels, open faces, and bore holes. However, bore hole samples are the least desirable because it is difficult to locate the slip

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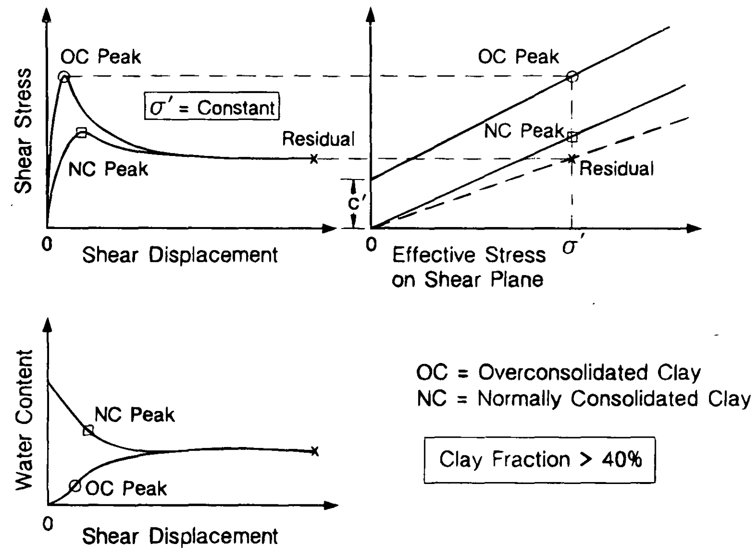

FIGURE 1 Drained shear characteristics of overconsolidated clays (*1*).

surface and to determine the natural direction of the shear in the recovered sample. The undisturbed specimen must be arranged in the direct shear apparatus such that the fully developed field slip surface is located at the gap between the upper and lower halves of the shear box. This is complicated by the fact that natural shear surfaces are usually nonhorizontal. The specimen then is sheared in the natural direction of movement. This test is referred to as a slip surface test and can provide an accurate estimate of the field residual strength (*3*). A similar process can be used to test natural shear surfaces in a torsional ring shear apparatus. However, it is difficult to obtain a natural slip surface sample, determine the direction of field shearing, and trim and properly align the usually nonhorizontal shear surface in the direct shear or ring shear apparatuses.

Laboratory-Formed Shear Surfaces

Because of the difficulties in obtaining and testing natural slip surfaces, the use of remolded specimens to measure the drained residual strength was investigated. Two types of remolded specimens, intact and precut, are used, and the relative merits of each will be discussed. In this study, a remolded specimen is obtained by air drying a representative sample of the overconsolidated clay or shale. The air-dried clay or shale is ball milled until all of the representative material passes U.S. Standard Sieve No. 200. Remolded silt and silty clay material (soil no. 1, 2, 3, 13, and 14 in Table 1) are obtained by using a mortar and pestle until all of the representative material passes U.S. Standard Sieve No. 40. Distilled water is added to the processed soil until a liquidity index of approximately 1.5 is obtained. The sample is then allowed to rehydrate for at least 1 week in a moist room. A spatula is used to place the remolded soil paste into the direct shear or ring shear specimen container to ensure that no air voids are present. The specimen is planed flush with the top of the specimen container using a razor blade, a fine wire saw, or both.

Normally Consolidated, Intact, Remolded Specimens

After consolidation, a normally consolidated, intact, remolded specimen is sheared until a drained residual strength is obtained. The main disadvantage of using an intact, remolded specimen is that a large continuous horizontal displacement, usually 250 to 400 mm, parallel to the direction of shear is required to form a shear surface and then to achieve a residual strength condition. Each reversal of a direct shear box is limited to a horizontal displacement that is usually less than 13 mm. Therefore, the use of a normally consolidated, intact, remolded specimen precludes the use of a reversal direct shear apparatus.

A torsional ring shear apparatus allows any magnitude of continuous shear displacement to be applied in one direction. This allows clay particles of an intact, remolded specimen to be oriented parallel to the direction of shear and the development of a residual strength condition. Other advantages of the ring shear apparatus include a constant cross-sectional area of the shear surface during the shear, a thinner specimen that allows the use of a faster drained displacement rate, minimal laboratory supervision during shear because there is no reversal of a shear box, and the use of data-acquisition techniques.

A number of different forms of the ring shear apparatus have been developed, for example, by Hvorslev (*6,7*), La Gatta (*8*), Bishop et al. (*9*), and Bromhead (*10*). However, the Bromhead ring shear apparatus is becoming widely used because of its cost, availability, and ease of operation. Bromhead and Curtis (*11*) showed that this ring shear apparatus yields results that are in good agreement with those obtained using the more sophisticated ring shear apparatus developed by the Norwegian Geotechnical Institute and Imperial College (*9*). Bromhead and Dixon (*12*) and Stark and Eid (*13,14*) also show that the drained residual strengths measured with the Bromhead apparatus are in excellent agreement with values back-calculated from landslide case histories.

Figure 2 illustrates the importance of continuous shear displacement in one direction on the measured residual strength of normally consolidated, intact, remolded specimens using the Santiago clay stone from San Diego, California (Table 1). It can be seen that direct shear tests using normally consolidated, intact, remolded specimens significantly overestimate the torsional ring shear test results because each reversal of the shear box is limited to a horizontal displacement of 5 mm and a limited amount of clay particles is oriented

TABLE 1 Clay and Shale Samples Used in Ring Shear Tests

Soil No.	Clay and Shale Samples	Clay and Shale Locations	Initial Water Content (%)	Specific Unit Weight (kN/m^3)	Liquid Limit	Plastic Limit	Clay Size Fraction (%)	Activity (PI/CF)
1	Glacial Till	Urbana, IL	8.4	16.1	24	16	18	0.44
2	Loess	Vicksburg, MS	14.0	16.5	28	18	10	1.00
3	Bootlegger Cove Clay	Anchorage, AL	34.8	18.6	35	18	44	0.39
4	*Duck Creek Shale	Fulton, IL	5.3	24.0	37	25	19	0.63
5	*Chinle (red) Shale	Holbrook, AZ	10.9	22.7	39	20	43	0.44
6	*Colorado Shale	Montana, MT	5.6	21.2	46	25	73	0.29
7	Panoche Mudstone	San Francisco, CA	14.2	19.6	47	27	41	0.49
8	*Four Fathom Shale	Durham, England	3.3	25.1	50	24	33	0.79
9	Mancos Shale	Price, UT	4.9	24.5	52	20	63	0.51
10	Panoche Shale	San Francisco, CA	12.0	20.2	53	29	50	0.48
11	*Comanche Shale	Proctor Dam, TX	11.5	23.1	62	32	68	0.44
12	*Bearpaw Shale	Billings, MT	15.7	21.8	68	24	51	0.86
13	Slide Debris	San Francisco, CA	18.1	19.6	69	22	56	0.84
14	Bay Mud	San Francisco, CA	73.0	15.0	76	41	16	2.19
15	*Patapsco Shale	Washington, D.C.	21.6	20.7	77	25	59	0.88
16	*Pierre Shale	Limon, CO	24.3	20.1	82	30	42	1.24
17	Santiago Claystone	San Diego, CA	20.7	19.6	89	44	57	0.79
18	Lower Pepper Shale	Waco Dam, TX	21.0	20.3	94	26	77	0.88
19	Altamira Bentonitic Tuff	Portuguese Bend, CA	62.0	17.5	98	37	68	0.90
20	Brown London Clay	Bradwell, England	33.0	18.9	101	35	66	1.02
21	*Cucaracha Shale	Panama Canal	18.4	20.7	111	42	63	1.10
22	Otay Bentonitic Shale	San Diego, CA	27.0	17.6	112	53	73	0.81
23	*Denver Shale	Denver, CO	30.5	18.7	121	37	67	1.25
24	*Bearpaw Shale	Saskatchewan, Canada	27.3	19.0	128	27	43	2.35
25	Oahe Firm Shale	Oahe Dam, SD	27.6	20.1	138	41	78	1.24
26	*Claggett Shale	Benton, MT	11.7	22.7	157	31	71	1.78
27	*Taylor Shale	San Antonio, TX	35.2	18.0	170	39	72	1.82
28	*Pierre Shale	Reliance, SD	42.8	17.7	184	55	84	1.54
29	Oahe Bentonitic Shale	Oahe Dam, SD	35.4	18.9	192	47	65	1.96
30	Panoche Clay Gouge	San Francisco, CA	34.8	21.8	219	56	72	2.26
31	Lea Park Bentonitic Shale	Saskatchewan, Canada	36.0	17.3	253	48	65	3.15
32	*Bearpaw Shale	Ft. Peck Dam, MT	15.8	21.8	288	44	88	2.77

* Index Properties from Mesri and Cepeda-Díaz (1986)

parallel to the shear. It should be noted that the direct shear tests reported here were conducted with a square specimen 60 × 60 mm in plan dimensions and 38 mm thick. Stark and Eid (*13*) used a landslide case history in the Santiago clay stone to show that the residual failure envelope measured using a ring shear apparatus and normally consolidated, intact, remolded specimens (Figure 2) is in good agreement with field observations.

In the original Bromhead ring shear apparatus (*10*), settlement of the top platen into the specimen container caused by consolidation and shearing can induce significant wall friction along the inner and outer edges of the specimen. This wall friction can lead to an overestimation of the residual shear strength. Stark and Vettel (*15*) concluded that the wall friction will be significant if the vertical displacement caused by consolidation and shearing exceeds 0.75 mm. Soil can be added during the consolidation process such that the intact, remolded specimen is at or near the top of the specimen container before shearing. This is a time-consuming process, but it results in satisfactory test results (Figure 2).

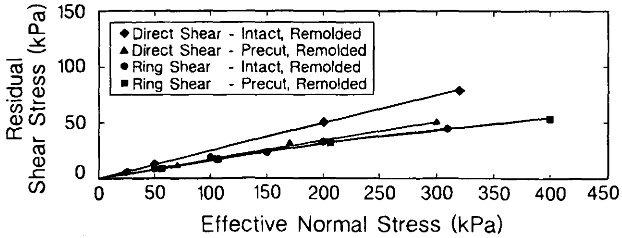

FIGURE 2 Drained residual failure envelopes for Santiago clay stone.

The original specimen container of the Bromhead ring shear apparatus was modified by Stark and Eid (*14*); their device is described subsequently to (*a*) overcome the problem of wall friction; (*b*) allow the use of overconsolidated, precut, remolded specimens; and (*c*) permit the use of a multistage test procedure. It can be seen from Figure 2 that the ring shear tests on intact and precut, remolded specimens are in agreement. It should be noted that the normally consolidated, intact, remolded specimens in Figure 2 were obtained by adding a substantial amount of remolded soil paste during consolidation of the intact specimen, such that the specimen was flush with the top of the container before shearing.

Figure 3 illustrates the effect of wall friction on the measured residual strength of Pierre shale from Reliance, South Dakota (Table 1). It can be seen that using normally consolidated, intact, remolded specimens and a single-stage test procedure in the original Bromhead ring apparatus provides a good estimate of the residual strength at effective normal stress less than approximately 50 kPa. At effective normal stresses greater than 50 kPa, consolidation of the specimen and soil extrusion during shear cause the

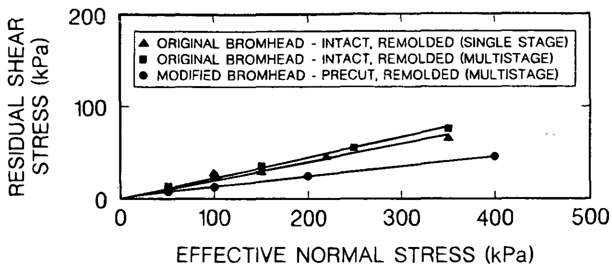

FIGURE 3 Effect of wall friction on measured residual strength of Pierre shale (Reliance).

vertical settlement to exceed 0.75 mm during a single-stage test, resulting in substantial wall friction and an overestimate of the residual strength. It should be noted that soil was not added during the consolidation phase in these tests.

It also can be seen that a multistage test procedure cannot be conducted in the original Bromhead apparatus with a normally consolidated, intact, remolded specimen. During the later stages of the multistage test, settlement of the top platen is significantly greater than 0.75 mm. This settlement and the resulting wall friction are greater than that observed in the single-stage test, which causes the multistage test to yield the highest drained residual failure envelope (Figure 3). It should be noted that additional remolded soil was not added during the consolidation phase in the multistage test.

The modified Bromhead ring shear apparatus allows a remolded specimen to be overconsolidated and precut, which minimizes settlement of the top platen during shear and reduces the horizontal displacement, and thus soil extrusion, required to achieve a residual strength condition. As a result, a multistage test procedure can be used with the modified apparatus and will yield a drained residual failure envelope (Figure 3) that is in excellent agreement with field case histories (*14,16*).

Overconsolidated, Precut, Remolded Specimens

A shear surface may be formed in a remolded specimen by overconsolidating and precutting or preshearing the specimen. The resulting specimen is termed an overconsolidated, precut, remolded specimen and can be used to reduce the horizontal displacement required to achieve a residual strength condition. In addition, it is anticipated that the use of an overconsolidated, precut specimen simulates the field conditions that lead to the development of a residual strength condition in overconsolidated clays. In this study, an overconsolidated, precut, remolded specimen is obtained by consolidating a specimen to a consolidation stress of 700 kPa. This consolidation stress was chosen to represent the maximum effective stress that typically is encountered in slope and embankment field case histories. After consolidation at 700 kPa, the specimen is unloaded and removed from the shear apparatus. The specimen is precut using a razor blade or presheared by subjecting the specimen to at least one revolution in the ring shear apparatus. The direct shear or ring shear specimen is precut in the direction of shear until a smooth and highly polished surface is obtained. After precutting, the specimen is loaded to the desired effective normal stress, which should be less than 700 kPa. This procedure results in a specimen that is overconsolidated before drained shear.

Mesri and Cepeda-Diaz (*17*) used the reversal direct shear apparatus to test overconsolidated, precut, remolded specimens. Each half of the shear box is filled with the remolded soil paste described previously. The two halves of the shear box are consolidated to approximately 700 kPa in separate modified oedometers. Each face of the shear surface is consolidated against a Tetko polyester screen that is supported by a smooth, flat Teflon plate. After consolidation, the two smooth, flat surfaces are precut in the direction of shear with a razor blade. The two precut surfaces are assembled together and sheared under the desired normal stress. Shearing is continued until a constant minimum resistance is measured. Figure 2 shows that this procedure yields drained residual shear stresses that are in good agreement with torsional ring shear tests. However, the use of overconsolidated, precut, remolded specimens in a reversal direct shear apparatus requires substantially more equipment, time, and effort than the use of a ring shear apparatus.

A modified Bromhead ring shear apparatus (*14*) was used for testing the 32 clays and clay shales described in Table 1. The modified and original ring shear specimen containers use an annular specimen with an inside diameter of 70 mm and an outside diameter of 100 mm. Drainage is provided by annular bronze porous stones secured to the bottom of the specimen cavity and to the top loading platen. The specimen is confined radially by the specimen container, which is 5 mm deep.

After consolidation at a normal stress of 700 kPa, the modified specimen container is removed from the apparatus, and the specimen is raised so that it is slightly above the top of the specimen container. This allows the specimen to be precut and minimizes the magnitude of wall friction. The specimen is raised by lowering the inner and outer rings that surround the annular specimen (*14*). The inner and outer rings of the specimen container are lowered so that approximately 0.5 mm of the specimen is visible above the top of the container. A razor blade or the ring shear apparatus is used to precut or preshear, respectively, the exposed specimen. The razor blade is placed on the upper surface of the specimen container, and the specimen is precut in the direction of shear until a smooth and highly polished surface is obtained. This results in a precut surface flush with the top of the specimen container before shearing. The specimen also can be precut or presheared in the ring shear apparatus by shearing the specimen for at least one revolution in the apparatus. Before preshearing, the specimen is raised so that it is approximately flush with the top of the specimen container. It is anticipated that using the apparatus to preshear the specimen is more practical than removing the top platen and using a razor blade. It can be seen from Figure 2 that the precut and intact, remolded specimens yield similar residual shear stresses. However, the precut specimen does not require soil to be added during the consolidation phase and requires significantly less displacement to reach a residual strength condition.

Single-Stage Versus Multistage Test Procedure

The single-stage test procedure involves consolidating a specimen at the desired normal stress and then shearing the specimen. After the residual strength condition is reached, the specimen is removed from the apparatus, and a new specimen is used for the next test. In a multistage test, after a residual strength condition has been established under the first effective normal stresses, shearing is stopped, and the effective normal stress is doubled. This procedure is repeated for a number of effective normal stresses to estimate the drained residual failure envelope. A multistage test can significantly reduce the time required to establish a drained residual failure envelope because a new specimen does not have to be prepared, consolidated, and precut for each effective normal stress. In addition, the horizontal displacement required to reach a residual strength condition is significantly reduced because a residual strength condition was attained during the first stage of the test. A multistage test also ensures that the same material is tested at each normal stress, which results in a more reproducible residual failure envelope.

Figure 4 presents the shear stress–horizontal displacement relationships for a normally consolidated, intact, remolded specimen and the second stage of a multistage test on an overconsolidated, precut, remolded specimen. These tests were conducted on Santiago clay stone (Table 1) at an effective normal stress (σ'_n) of 100 kPa. The precut specimen exhibited a significantly lower peak strength because the specimen had already attained a residual strength condition during the first stage of shearing at an effective normal stress of 50 kPa. As a result, only approximately 10 mm of horizontal displacement is required to achieve a residual strength condition during the second stage of the test.

The intact, remolded specimen exhibited a significantly larger peak strength because a shear plane had not been previously formed and no reorientation of the clay particles occurred before drained shearing. As a result, a horizontal displacement of approximately 70 mm is required to obtain a residual strength condition. Because the shear displacement rate is 0.018 mm/min, it takes an additional 2.5 days to achieve a residual strength condition using an intact specimen compared with the precut specimen. It should be noted that the displacement rate of 0.018 mm/min used in the ring shear tests described here was estimated using the procedure described by Gibson and Henkel (*18*) and a degree of consolidation of 99.5 percent.

It also can be seen in Figure 4 that the precut and intact specimens yielded similar residual strengths. This was accomplished by adding soil and reconsolidating the intact, remolded specimen so that settlement of the top platen was negligible before shear. It should be noted that the vertical displacement of the precut, remolded specimen is only about 0.03 mm (Figure 4). This is less than the vertical displacement observed during the first stage of shearing (0.06 mm) and substantially less than the 0.35 mm measured with the intact specimen. The reduction in vertical displacement is attributable to the overconsolidated nature of the precut specimen and the smaller horizontal displacement required to reach a residual strength condition. This minimal vertical displacement ensures that a negligible amount of wall friction is applied to the shear plane during a multistage test.

Some shear apparatuses may not be suited to a multistage test procedure. For example, in a direct shear apparatus a new shear surface may be created during subsequent shearing stages because of consolidation of the specimen under a successive normal stress. This consolidation may lead to a lowering of the previous shear surface below the gap between the upper and lower halves of the shear box, thus creating a new shear surface.

The original Bromhead ring shear apparatus can be used to measure the drained residual strength accurately if settlement of the top platen, caused by consolidation, soil extrusion during shear, or both, is limited to 0.75 mm (*15*). The modified Bromhead ring shear apparatus significantly reduces the time required to estimate a drained residual failure envelope by allowing the use of an overconsolidated, precut, remolded specimen and a multistage test procedure. An overconsolidated, precut, remolded specimen and a single-stage test procedure can be used in a reversal direct shear apparatus

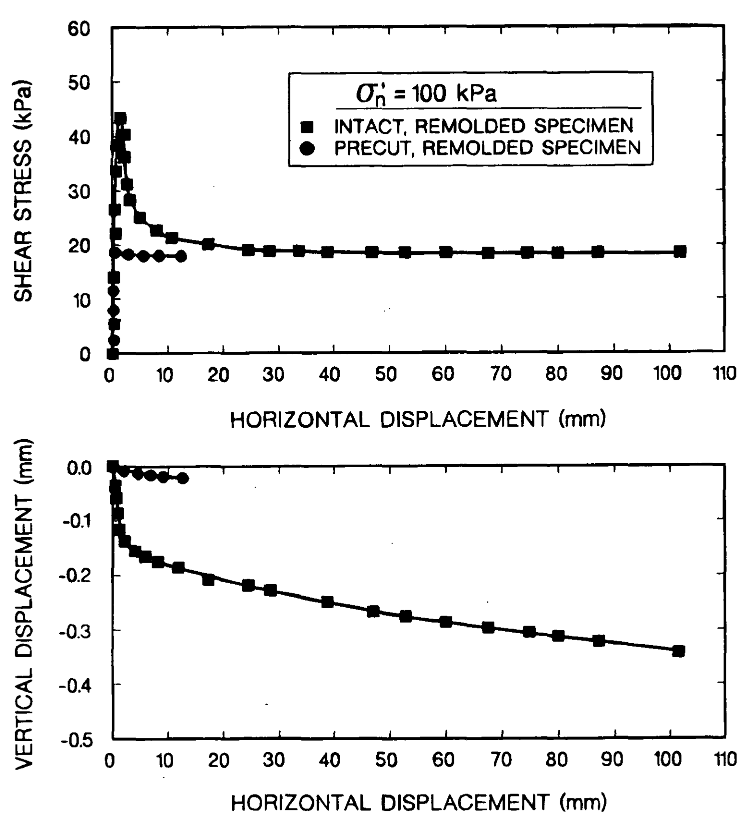

FIGURE 4 Drained ring shear test results for Santiago clay stone at effective normal stress of 100 kPa (*14*).

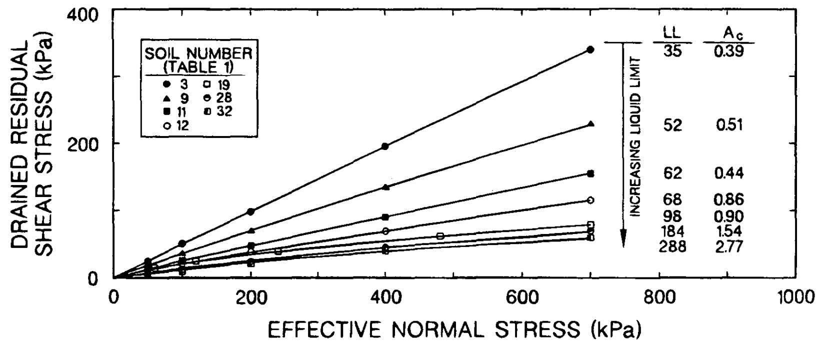

FIGURE 5 Effect of clay mineralogy on drained residual failure envelopes (*16*).

to obtain similar results as the modified Bromhead ring shear apparatus (Figure 2). However, this direct-shear procedure requires substantially more equipment (oedometers and a direct-shear apparatus) and time (*13*).

NONLINEARITY OF DRAINED RESIDUAL STRENGTH ENVELOPE

Stark and Eid (*16*) illustrate the effect of clay mineralogy and clay particle size on the drained residual strength using results of ring shear tests on 32 clays and clay shales (Table 1). The tests were performed using (*a*) the modified Bromhead ring shear apparatus; (*b*) overconsolidated, precut, remolded specimens; and (*c*) the multistage test procedure described previously. Figure 5 presents the drained residual failure envelopes for seven of the clays and clay shales listed in Table 1. It can be seen that the magnitude of the drained residual strength decreases with increasing liquid limit and activity. The activity (A_c) is defined as the plasticity index divided by the clay-size fraction. Both the liquid limit and activity provide an indication of clay mineralogy, and thus particle size and shape. In general, the plasticity increases as the platyness of the clay particles increases. Increasing the platyness of the particles results in a greater tendency for face-to-face interaction and thus a lower drained residual strength.

Figure 5 also shows that the drained residual failure envelope can be nonlinear. The nonlinearity appears to be significant for cohesive soils with moderate to high liquid limit and activity. Figure 6 presents the ratio of the secant residual friction angle at 50 kPa, $(\phi'_r)_{50}$, and 700 kPa, $(\phi'_r)_{700}$, for the 32 cohesive soils shown in Table 1. The secant residual friction angle corresponds to a linear failure envelope passing through the origin and the residual shear stress at a particular effective normal stress. It can be seen that the ratio of $(\phi'_r)_{50}$ to $(\phi'_r)_{700}$ is less than 1.3 for clay-size fractions less than 50 percent. For clay-size fractions greater than 50 percent, the ratio of $(\phi'_r)_{50}$ to $(\phi'_r)_{700}$ reaches a maximum of 1.85 to 1.9 at a liquid limit of approximately 100 and decreases to about 1.1 at a liquid limit of 288. Therefore, it may be concluded that the nonlinearity of the drained residual failure envelope is significant; that is $(\phi'_r)_{50}/(\phi'_r)_{700}$ is greater than 1.3 for overconsolidated clays with a liquid limit between 60 and 220 and a clay-size fraction greater than 50 percent. In this range of liquid limit and clay-size fraction, the residual friction angle undergoes a reduction of 25 to 45 percent for effective normal stresses ranging from 50 to 700 kPa.

The effect of a nonlinear residual failure envelope on a stability analysis was investigated using several case histories (*16*). It was found that a stability analysis is sensitive to the stress-dependent nature of the residual strength. As a result, it is concluded that it is more reliable to use the entire nonlinear residual envelope directly in a stability analysis. However, the case histories (*16*) also revealed that the use of a secant residual friction angle that corresponds to the average effective normal stress on the critical slip surface also will provide good agreement with field observations for long slablike failure surfaces that exhibit minor variations in effective normal stress. Thus, for practical purposes the nonlinear residual failure envelope can be estimated using a residual friction angle that

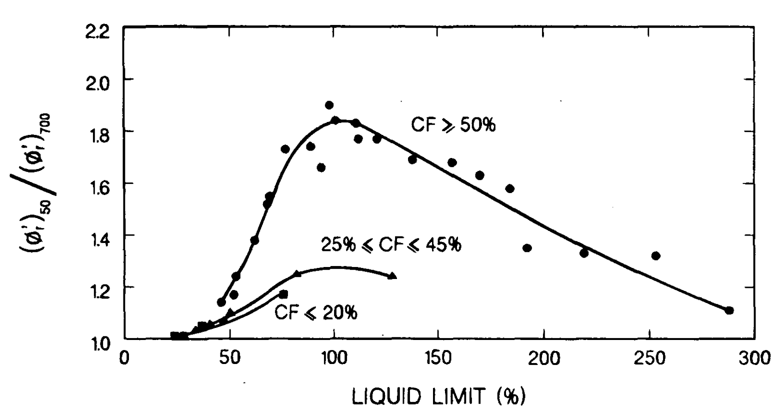

FIGURE 6 Reduction in residual friction angle from effective normal stresses of 50 to 700 kPa (*16*).

TABLE 2 Existing Drained Residual Friction Angle Correlations

Soil Index Property Used in Correlation	Drained Residual Strength Correlation Reference
Clay Size Fraction	Binnie, et al. (19) Blondeau & Josseaume (20) Borowicka (21) Collotta, et al. (22) Lupini, et al. (23) Skempton (3) Skempton (24)
Plasticity Index	Bucher (25) Clemente (26) Fleischer (27) Kanji (28) Lambe (29) Mitchell (30) Seycek (31) Vaughan, et al. (32) Voight (33)
Liquid Limit	Haefeli (34) Mesri and Cepeda (17) Mitchell (30)

corresponds to the average effective normal stress on the critical slip surface.

DRAINED RESIDUAL STRENGTH CORRELATIONS

Because of difficulties associated with obtaining natural slip surface specimens and the laboratory measurement of the drained residual strength, a number of correlations of drained residual friction angle have been proposed (Table 2). These correlations relate the drained residual friction angle to only one soil index property. Stark and Eid (*16*) showed that both the liquid limit and clay-size fraction are required to estimate the secant residual friction angle accurately. In addition, existing correlations ignore the stress-dependent behavior of the drained residual shear strength, which is significant for overconsolidated clays with a liquid limit between 60 and 220 and a clay-size fraction greater than 50 percent.

Figure 7 presents a new correlation of drained residual friction angle. It can be seen that there is a relationship between the secant residual friction angle at effective normal stresses of 100, 400, and 700 kPa and both the liquid limit and clay-size fraction. The higher the liquid limit and clay-size fraction, the lower the secant residual friction angle. The liquid limit appears to be a suitable indicator of clay mineralogy, and thus drained residual strength. However, the clay-size fraction remains an important predictive parameter because it indicates the quantity of clay-size particles, which are particles smaller than 0.002 mm. The proposed correlation differs from existing correlations because the drained residual friction angle is a function of the liquid limit, clay-size fraction, and effective normal stress. Stark and Eid (*16*) compare the proposed and existing correlations using field case histories. It was found that the proposed correlation provided the best agreement with the back-calculated values of the residual friction angle for the field case histories considered, because it incorporates clay mineralogy, the clay-size fraction, and effective normal stress.

The nonlinearity of the drained residual failure envelope is evident by the decrease in the secant residual friction angle with increasing effective normal stress. Figure 7 also confirms that the nonlinearity is significant for cohesive soils with a clay-size fraction greater than 50 percent and a liquid limit between 60 and 220. For example, at a liquid limit of 100 and a clay-size fraction greater than 50 percent, the secant residual friction angle decreases from 9.5 degrees at an effective normal stress of 100 kPa to 6.0 degrees (or 36 percent) at an effective normal stress of 700 kPa. For clay-size fractions less than 50 percent and liquid limits less than 120, the reduction in residual friction angle from 100 to 700 kPa is less than approximately 1.5 degrees.

The secant residual friction angle for a cohesive soil can be estimated for a particular effective normal stress using the liquid limit and clay-size fraction and linearly interpolating between the curves presented in Figure 7. In addition, Figure 7 can be used to estimate the nonlinear residual failure envelope by plotting the shear stress corresponding to the drained residual friction angle at effective nor-

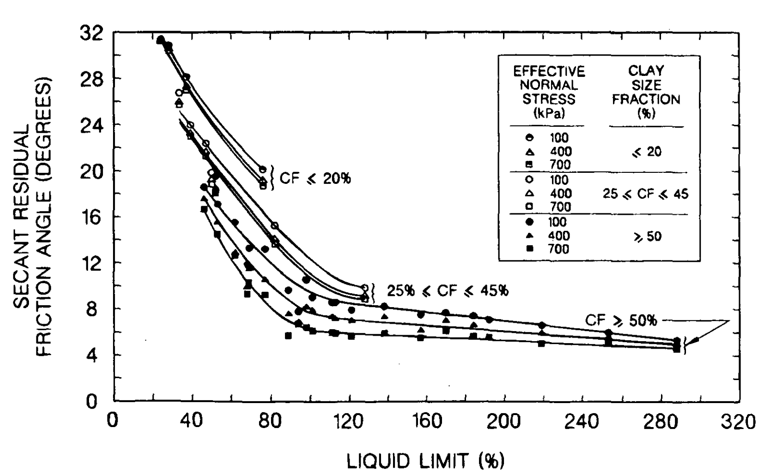

FIGURE 7 Relationship between drained residual friction angle and liquid limit (*16*).

mal stresses of 100, 400, and 700 kPa. A smooth curve can be drawn through these three points and the origin to estimate the nonlinear residual failure envelope.

CONCLUSIONS

The drained residual shear strength of overconsolidated clays is an important parameter in assessing the stability of slopes that contain a preexisting shear surface, such as old landslides or soliflucted slopes, bedding shears in folded strata, sheared joints or faults, or a failed embankment (*3*). Therefore, laboratory measurement of the drained residual strength requires a specimen that contains a shear surface. Because of difficulties in obtaining and testing natural slip surface specimens, a laboratory technique that uses overconsolidated, precut, remolded specimens was developed. A Bromhead ring shear apparatus was modified to permit the use of overconsolidated, precut, remolded specimens and a multistage test procedure. The drained residual strengths measured using this apparatus and test procedure are in excellent agreement with field case histories (*16*).

The magnitude of the drained residual strength is controlled by the type of clay mineral and the quantity of clay-size particles. The liquid limit provides an indication of clay mineralogy, and the clay-size fraction indicates the quantity of clay-size particles, which are particles smaller than 0.002 mm. Therefore, both the liquid limit and clay-size fraction should be used in correlations to estimate the drained residual strength. The results of ring shear tests on 32 clay and clay shales reveal that the drained residual failure envelope can be nonlinear. The nonlinearity is significant for cohesive soils with a clay-size fraction greater than 50 percent and a liquid limit between 60 and 220. This nonlinearity should be incorporated into stability analyses by modeling the entire residual failure envelope or using a secant residual friction angle that corresponds to the average effective normal stress on the slip surface. A new drained residual friction angle correlation is presented that is a function of the liquid limit, clay-size fraction, and effective normal stress. This correlation can be used to estimate the entire nonlinear residual failure envelope or a secant residual friction angle for the average effective normal stress on the slip surface.

ACKNOWLEDGMENTS

This study was performed as part of a National Science Foundation grant. The support of this agency is gratefully acknowledged. The author also acknowledges the support provided by the W.J. and E.F. Hall Scholar Award. G. Mesri provided many valuable suggestions during this study.

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In Situ Testing in Overconsolidated Clays

AN-BIN HUANG

The usefulness of several in situ test methods in the determination of engineering properties of overconsolidated clays is described. The majority of overconsolidated clays have a stiff to very stiff consistency, which is favorable for most in situ test methods. Four popular test methods are considered: cone penetration, pressuremeter, vane shear, and Marchetti dilatometer. The test equipment for these methods is readily available, and they are likely to yield useful data if used properly. For each test method, details of the apparatus, test procedure, and interpretation of the test data are given.

Except in highly overconsolidated, very hard clays, there is little physical limitation in conducting in situ testing in overconsolidated (OC) clays. It is often possible to bore relatively deep holes in OC clays using a solid stem auger without the need for either steel casing or drilling fluid. The soil conditions are often favorable for methods such as the pre-bore pressuremeter test, where a borehole is needed. There are many in situ test methods, and most of them can be used in OC clays. A few commonly used in situ test methods that are known to be effective in characterizing OC clays are described.

Essentially all in situ tests involve complicated and often unknown boundary conditions caused by the installation of the test device. The situation is further complicated by the anisotropic and strain rate-dependent nature of clays. As a result, the test data are valid only if standard equipment and test procedures are followed. It is rarely feasible to interpret the in situ test data rigorously. Laboratory tests performed on relatively undisturbed specimens have been used to establish empirical correlations between in situ tests and soil properties. The validity of an empirical or semiempirical interpretation method should be considered as being site-specific and test-method-specific.

The in situ test methods described in this paper are cone penetration, pressuremeter, vane shear, and Marchetti dilatometer tests. One of the most common tests, the standard penetration test (SPT) is not included because the SPT blow count is known to be sensitive to hammer efficiency and to not have a consistent relationship with cohesive soil properties.

The aim of this paper is to provide brief but sufficient information to highway engineers and to assist in their selection of an in situ test method. For each method, the apparatus, test procedure, and a means of interpretation of test data are presented.

CONE PENETRATION TEST

Apparatus and Test Procedure

According to ASTM 3441 (*1*), a cone penetrometer should have a point angle of 60 degrees and a base diameter of 35.7 mm, resulting in a projected area of 10 cm^2. The friction sleeve has a surface area of 150 cm^2. The cone tip (q_c) and sleeve (f_s) resistance are measured by means of force transducers located within the cone tip. In an older design, generally referred to as the mechanical cone, the q_c and f_s readings are taken at the ground surface with a pressure gauge or a load cell. A thrust machine such as a drill rig or cone truck is used to push the cone. A penetration rate of 20 mm/sec should be maintained when obtaining resistance data.

Electric cone penetrometers may include other transducer measurements as well as, or instead of, the friction sleeve measurement. A common one is a piezometer to provide pore pressure measurements (u) during penetration. The cone penetration test (CPT) using a cone equipped with a piezometer is referred to as a piezocone test (CPTU). There is no standard for the location of the piezometer with respect to the cone tip. Unfortunately, the piezometer location can have a significant effect on the magnitude of pore pressure measurement (*2*). Hence, it is necessary to indicate the position of the piezometer in reporting the CPTU data.

Interpretation

An important advantage of a piezocone is to account for the unbalanced water forces acting on the cone tip and sleeve because of unequal end areas in cone design (see Figure 1). Correction of q_c should be carried out using the following relationship (*3,4*):

$$q_T = q_c + u(1 - a) \tag{1}$$

where

q_T = corrected total tip resistance,
u = measured pore pressure using a filter located at the shoulder point behind the cone tip, and
a = net area ratio.

The difference between q_T and q_c can be very significant, especially in a soft clay. A similar correction is required for sleeve friction data.

The major application of cone penetration has been for soil profiling. Earlier classification charts have been based on q_T and friction ratio $FR = (f_s/q_T) \times 100$ percent. In general, sandy soils have higher q_T and lower FR values, whereas clayey soils have lower q_T and higher FR values. The soil classification can be further refined using the pore pressure readings from CPTU. Earlier studies have suggested the possibility of revealing OCR using the pore pressure readings from CPTU (*5*). Figure 2 shows a classification chart according to Robertson (*6*) based on q_T, FR, and pore pressure parameter ratio B_q, defined as

$$B_q = \frac{\Delta u}{q_T - \sigma_{vo}} \tag{2}$$

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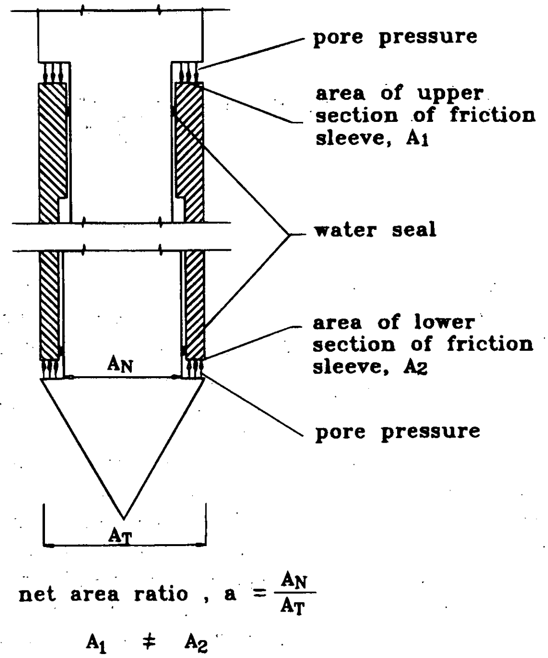


FIGURE 1 Unequal end area correction (*2*).

where Δu is the excess pore pressure ($u - u_0$), and σ_{vo} is the total overburden stress.

The classification chart by Robertson (*6*) should be used with caution as researchers (*7–9*) have indicated that the relationship between B_q and OCR may be very site-specific.

Mayne (*10*) combined the cavity expansion and modified Cam–Clay theories and proposed equations that relate CPTU data with OCR. For CPTU with pore pressure measurements at the cone tip (u_t),

$$OCR = 2\left[\frac{1}{1.95M}\left(\frac{q_T - u_t}{\sigma'_{vo}} - 1\right)\right]^{1.33} \tag{3a}$$

where

σ'_{vo} = effective overburden stress,
$M = 6 \sin \phi'/(3 - \sin \phi')$, and
ϕ' = effective friction angle.

For CPTU with pore pressure measurements behind the cone tip (u_{bt}),

$$OCR = 2\left[\frac{1}{1.95M + 1}\left(\frac{q_T - u_{bt}}{\sigma'_{vo}} - 1\right)\right]^{1.33} \tag{3b}$$

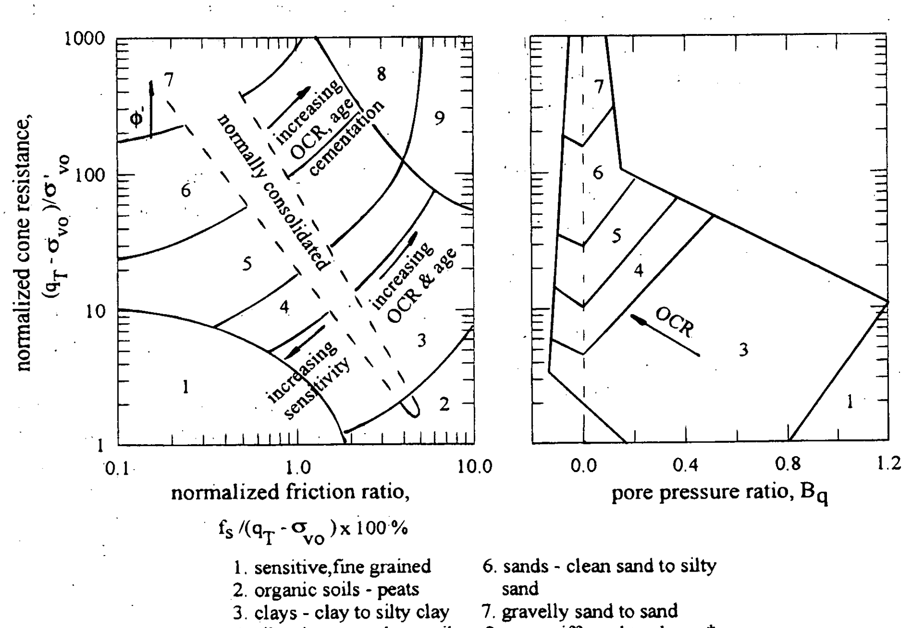
Mayne (*10*) reviewed data with OCR ranging from 1 to over 60 and ϕ' from 20 degrees to 38 degrees, and concluded that Equations 3a and 3b provide reasonable first-order estimates of in situ OCR for a variety of clay deposits.

Undrained cone penetration in clay is a very complex problem, and there is no generally accepted theory for the determination of the undrained shear strength s_u from CPT or CPTU. A common procedure in estimating s_u is to use the bearing capacity equation as follows (*11*):

$$s_u = \frac{q_T - \sigma_{vo}}{N_{KT}} \tag{4}$$

where N_{KT} is the empirical cone factor.

The value of N_{KT} unfortunately could vary between 4 and 30. Factors that may influence N_{KT} include sensitivity, stress history, stiff-



1. sensitive,fine grained
2. organic soils - peats
3. clays - clay to silty clay
4. silt mixtures - clayey silt to silty clay
5. sand mixtures - silty sand to sandy silt
6. sands - clean sand to silty sand
7. gravelly sand to sand
8. very stiff sand to clayey* sand
9. very stiff, fine grained*

(*) heavily overconsolidated or cemented

FIGURE 2 Soil behavior type chart from CPTU data (*6*).

ness, clay macrofabric, and definition of s_u. Often, a representative value of $N_{KT} = 15$ is adopted for obtaining the average s_u in intact clays (*12*). There is no general agreement on what s_u should refer to. Campanella and Robertson (2) suggest that the s_u from field vane shear test should be used as the reference s_u value and should be stated when reporting the interpretation of cone penetration data.

Methods of evaluating the clay coefficient of consolidation (c_h) from the rate of pore pressure dissipation around the cone tip have been proposed. However, these methods are suitable only for normally or lightly overconsolidated (OCR < 4) clays because of difficulties in estimating the initial pore pressure distribution around the cone in a stiff overconsolidated (OCR > 4) clay. Readers interested in the dissipation test are referred to the paper by Levadoux and Baligh (*13*).

PRESSUREMETER TEST

Apparatus and Test Procedure

The basic concept of the pressuremeter test (PMT) is to lower an inflatable cylindrical probe into a borehole and expand it to measure the pressure-deformation properties of soil. The pressuremeter as it was originally developed by Menard (*14*) consists of three independent, water-inflated chambers (tricell) stacked one above the other. The purpose of the top and bottom chambers (guard cells) is to protect the middle chamber (measuring cell) from the end effects caused by the finite length of the apparatus. All the test results are based on the measurements in the middle chamber.

A number of variants of the pressuremeter have been introduced since the late 1960s. Figure 3 shows five of the new alternatives along with the original Menard pressuremeter. The Menard, TEXAM, and OYO pressuremeters are designed to be used in prebored holes. They are referred to collectively as the prebore pressuremeter. The full displacement and stressprobe are introduced into the ground by pushing and therefore displacing soil during insertion. The self-boring pressuremeter is hollow and cylindrical in shape and has its own cutter to make the borehole and remove cuttings through the internal opening. All of the newer designs shown in Figure 3 use a single-cell probe (i.e., it has no guard cells). The single-cell pressuremeters are generally easier to operate than the tricell Menard pressuremeter. The use of non-prebore pressuremeters is not common for general geotechnical engineering exploration work and thus will not be discussed further.

A key to the success of performing a prebore PMT is the preparation of the borehole. According to ASTM D4719-87 (*1*), the borehole diameter should be within 1.03 to 1.2 times the pressuremeter probe diameter. For OC clays, the pressuremeter cavity may be prepared using an auger or a thin-wall Shelby tube sampler. For borehole preparation in highly overconsolidated, very hard clays, Lukas and Seiler (*16*) indicated that a rock bit or shaver along with drill and mud may be used. The shaver is a device that has a rock bit attached at the lower end of a cylindrical tube.

Upon borehole preparation, the PMT probe should be inserted as soon as possible. The probe expansion may be stress or strain-controlled. For the stress-controlled test, readings are taken at 30 sec and 60 sec after the pressure increments have been applied. The volume difference between the 30-sec and 60-sec readings is associated with soil creep and is referred to as the creep volume. There should be sufficient increments to yield data points that can properly define a volume-pressure curve (Figure 4).

Interpretation

The pressuremeter curve has characteristics as shown in Figure 5. Because of soil disturbance and the oversize condition of the bore-

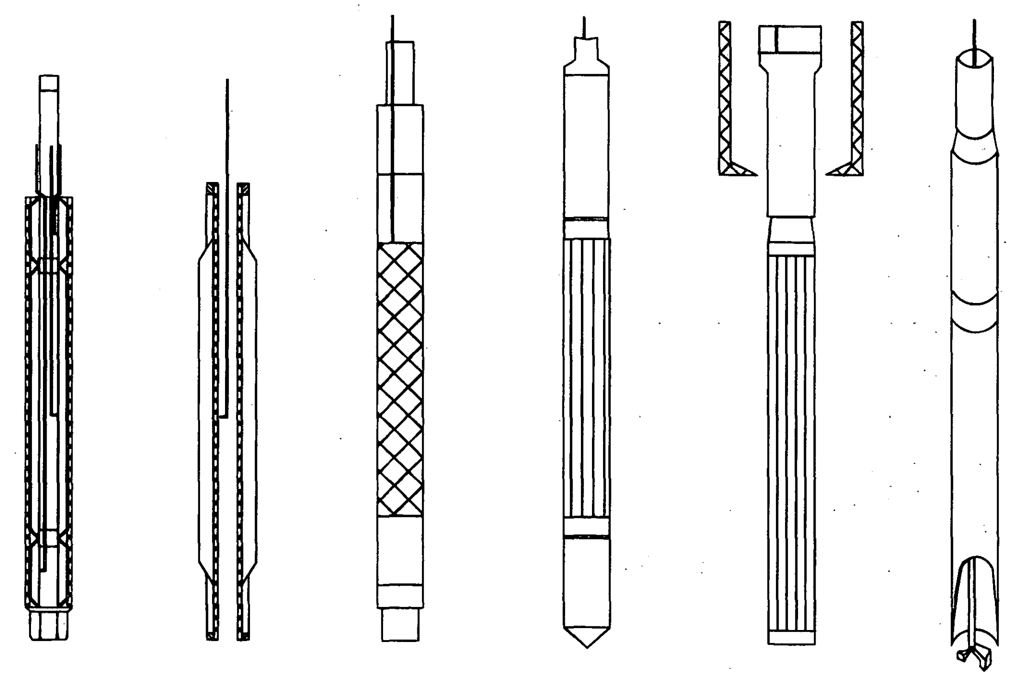


FIGURE 3 Menard and other pressuremeter probes (*17*).

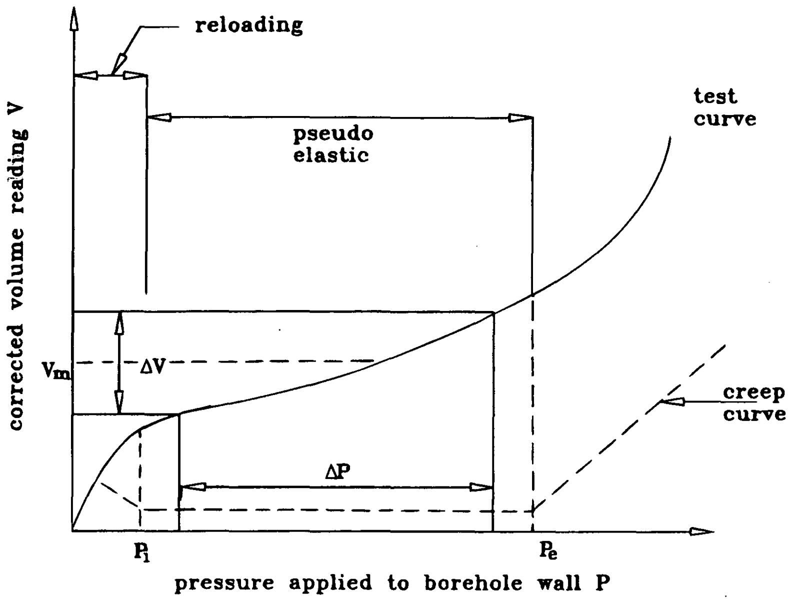


FIGURE 4 Typical pressuremeter expansion curve.

hole, some probe expansion occurs before the probe pressure reaches the lateral stress present in the soil mass ("reloading" in Figure 4). The inflection point corresponds to a point where creep volume reduces to a minimum, defined as the break point in the expansion curve where reloading ends and loading starts. The probe pressure at this inflection point (P_i) is considered by many researchers (*17–19*) to be an estimate of the in situ lateral stress.

For a PMT in OC clay, there is usually a pseudoelastic part of the pressuremeter expansion curve following the P_i point. Within the pseudoelastic zone, the creep volume remains relatively constant

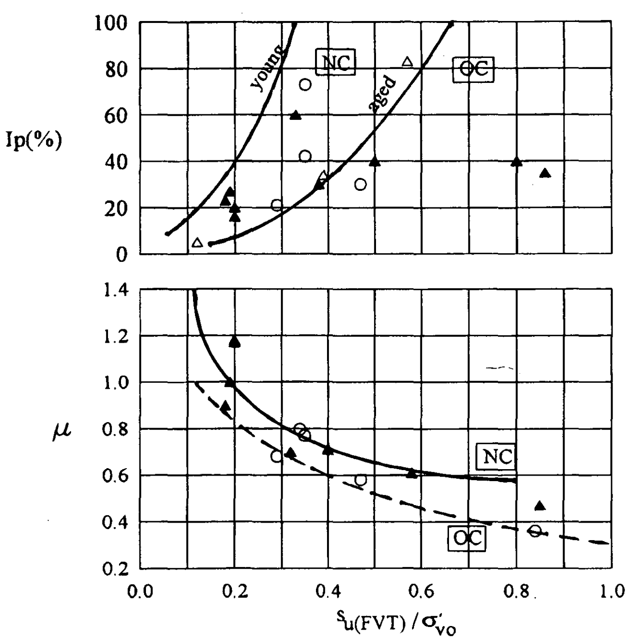


FIGURE 5 Determination of stress history and field vane correction factor (*41*).

and the expansion curve is close to a linear condition. Taking derivative within the linear part of the expansion curve, the pressuremeter modulus (E_p) is defined as

$$E_p = 2(1 + \nu)(V_o + V_m)\frac{\Delta P}{\Delta V} \tag{5}$$

where

ν = Poisson's ratio,
V_o = initial volume of the pressuremeter probe, and
V_m = volume reading in the center portion of the ΔV volume increase.

The break point in the expansion curve where creep volume starts increasing is referred to as the creep pressure (P_e). The limit pressure (P_l) is defined as the pressure where the probe volume reaches twice the original cavity volume and is usually obtained as an extrapolated value (*17*).

Baguelin et al. (*20*) presented empirical procedures that use PMT results directly in foundation designs. For axially loaded foundations the net ultimate bearing capacity q_{net} may be calculated as

$$q_{net} = k\,(P_l - P_i) \tag{6}$$

where k is the bearing capacity factor.

For OC clays, k values range from 0.8 to approximately 3.6. The foundation settlement w is related to PMT results as follows (*20*):

$$w = \frac{1.33}{3E}pR_0\left(\lambda_2\frac{R}{R_0}\right)^{\alpha} + \frac{\alpha}{4.5E}p\lambda_3R \tag{7}$$

where

p = net bearing pressure,
R_0 = reference length equal to 30 cm,
R = radius or half-width of the foundation,
λ_2,λ_3 = shape factors (see Table 1),

TABLE 1 Shape Factors (*20*)

$L/2R$	Circle	Square	2	3	5	20
λ_2	1	1.12	1.53	1.78	2.14	2.65
λ_3	1	1.10	1.20	1.30	1.40	1.50

α = structure coefficient, equal to 1 for OC clays, and
E = essentially the harmonic mean of E_p values for soils below the foundation level.

For the analysis of a long, laterally loaded pile, Gambin (*21*) proposed that the lateral soil reaction modulus k be calculated as

$$\frac{1}{k} = \frac{1.33}{3E} R_0 \left(2.65 \frac{R}{R_0}\right)^{\alpha} + \frac{\alpha}{3E} R \tag{8}$$

Methods of establishing p-y curves for the analysis of laterally loaded piles have been proposed (*21–23*). Briaud and Cosentino (*24*) suggested the use of PMT results in pavement designs.

The PMT can be reasonably considered as a cylindrical cavity expansion. That simplification enables interpretation of the test results in a more rigorous manner. If the PMT expansion curve is shifted so that P_i corresponds to zero radial strain, a stress-strain relationship may be derived by taking derivatives of the expansion curve (*25–27*) as follows:

$$\sigma_r - \sigma_\theta = 2\epsilon_{ro} \frac{dP}{d\epsilon_{ro}} \tag{9}$$

where σ_r is the radial stress, and σ_θ is the circumferential stress.

The shear modulus G can be computed as

$$G = 2 \frac{dP}{d\epsilon_{ro}} \tag{10}$$

Other graphical or curve-fitting techniques (*28*) have also been proposed to obtain stress-strain relationships from PMT results. Because of the soil disturbance and relaxation during borehole preparation, the stress-strain relationships obtained from these rigorous procedures are not always reliable (*29*). As an alternative, it is more desirable to empirically relate soil parameters to PMT results. Consider the PMT in OC clay as an undrained test ($\nu = 0.5$), and according to Hill (*30*),

$$P_l - P_i = \beta s_u$$

$$\beta = 1 + \ln \frac{E_u}{3s_u} \tag{11}$$

where E_u is the undrained modulus.

Data collected by Holtz and Kovacs (*31*) show that E_u/s_u values range from 200 to 1,800, which would give β values of 5.2 to 7.4. Baguelin et al. (*20*) indicated that for stiff to very stiff clays, β has an average value of 9. Lukas and De Bussy (*32*) reported a β value of 5.1 for cohesive tills and hardpan in Chicago.

The ratio of E_p/P_l relates to soil properties and may be used to classify soils. Baguelin et al. (*20*) and Gambin (*21*) showed that for OC clay, E_p/P_l is greater than 16. Lukas and Seiler (*16*) showed that E_p/P_l varies from 4 to 11 for low-plasticity clays. For high-plasticity clays, E_p/P_l ranges from 8 to well over 25.

FIELD VANE SHEAR TEST

Apparatus and Test Procedure

The field vane test (FVT) has been used extensively for the in situ determination of the undrained strength of soft clays. For practical purposes, the FVT may be used in clays with OCRs less than 10. Beyond that, the excessive torque may cause distortion or even breakage to the vane blades, unless an unusually small vane is used.

A standard field vane (ASTM D2573-72) has four blades. The height of the vane (H) should be twice the diameter (D). The rod friction should be accounted for with the use of sleeved rods or a slip coupling. The vane should be inserted to a depth that is at least five times the diameter of the borehole or that of the vane housing before testing. There is usually a "rest period" of not more than 5 min following vane insertion. The vane should be rotated at a rate not exceeding 0.1 degree/sec (ASTM D2573-72). Following the determination of the maximum torque, the vane is rotated rapidly for 10 revolutions. The test is then repeated to determine the remolded strength. The ratio of peak to remolded strength is referred to as sensitivity.

Vanes with different dimensions are allowed by the ASTM standard. An advantage of allowing different vane dimensions is that the accuracy of the torque measuring system maybe optimized. However, for a given rate of rotation, the strain rate at the tip of the vane blade is proportional to the vane diameter. Studies (*33–35*) have shown that the strain-rate effects are important for the FVT. To minimize the strain rate effects, it is beneficial to restrict the vane dimensions. Chandler (*36*) suggests that since the most widely used dimensions are H = 130 mm and D = 65 mm, these would seem to be the most appropriate for standardization.

Although not specified in the ASTM standard, there seems to be a general agreement that the vane blade thickness should be approximately 2 mm, and the area ratio (the ratio of the volume of soil displaced by the vane to the soil volume swept by the rotated vane) should be less than 12 percent.

Interpretation

Assuming that the clay is isotropic and shear stress is uniformly distributed along the edge of the vane blades, then for $H/D = 2$, the undrained shear strength is

$$s_u = 0.86T/\pi D^3 \tag{12}$$

where T is the maximum recorded torque. Wroth (*37*) concluded that the shear stress distribution at the top and bottom of the vane blades should be described by a polynomial. In that case

$$s_u = 0.94T/\pi D^3 \tag{13}$$

Using Wroth's approach, the vertical surfaces contribute 94 percent of the resistance to the total torque, not 86 percent according to Equation 12, and the shear strength will be dominantly that exhibited by the vertical planes. Consequently, Equation 12 would underestimate s_u and the FVT is not likely to reveal the strength anisotropy by changing H/D ratios.

Due to strain rate effects and soil anisotropy, Bjerrum (*38,39*) pointed out that there is a discrepancy between the shear strength from the FVT ($s_{u(FVT)}$) and that backcalculated from embankment and excavation failures and proposed that

$$s_{u\,\text{corrected}} = \mu s_{u(FVT)} \tag{14}$$

where μ is a correction factor based on the plasticity index, I_p. The validity of Bjerrum's approach was seriously questioned by many researchers [e.g. Schmertmann (*40*)] because of significant scatter of the data that Bjerrum used to establish μ values. Aas et al. (*41*) attributed that scatter to the lack of consideration of soil stress history and aging. A set of modified or renewed correction curves as shown in Figure 5 was proposed by Aas et al. (*41*). These curves consider both aging and stress history of clays. To use Figure 5, the $s_{u(FVT)}$ values should be calculated using Equation 12.

MARCHETTI DILATOMETER TEST

Apparatus and Test Procedure

The Marchetti dilatometer (*42*) consists of a stainless steel blade with a circular, expandable diaphragm on one side. The dimensions and geometry of the blade are shown in Figure 6. The Marchetti dilatometer test (DMT) involves the penetration of the blade followed by expansion of the diaphragm. A recommended DMT test procedure has been proposed by Schmertmann (*43*). Upon penetration, the diaphragm is expanded slowly by air pressure. A pressure gauge in the control console monitors the air pressure being applied behind the diaphragm. The console gives an electric signal when the diaphragm moves 0.05 mm horizontally off the vertical blade and when the central diaphragm expansion reaches 1.1 mm. The two corresponding pressures are referred to as the A and B reading, respectively. These pressures are corrected for diaphragm stiffness such that

$$P_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B) \tag{15}$$

$$P_1 = B - Z_M - \Delta B \tag{16}$$

where

P_0 = net soil pressure against the membrane immediately before its expansion into the soil,
P_1 = net soil pressure at 1.1 mm membrane expansion, and
Z_M = gauge pressure deviation from zero when vented at atmospheric pressure.

The tests are repeated at intervals of approximately 20 cm, thus resulting in a large number of data for a given location.

Interpretation

The P_0 and P_1 pressures along with an estimate of the effective vertical stress σ'_{vo} and hydrostatic pressure u_0 at the test level are used to provide three indices:

Material index:

$$I_D = \frac{P_1 - P_0}{P_0 - u_0} \tag{17}$$

Horizontal stress index:

$$K_D = \frac{P_0 - u_0}{\sigma'_{vo}} \tag{18}$$

Dilatometer modulus:

$$E_D = 34.7(P_1 - P_0) \tag{19}$$

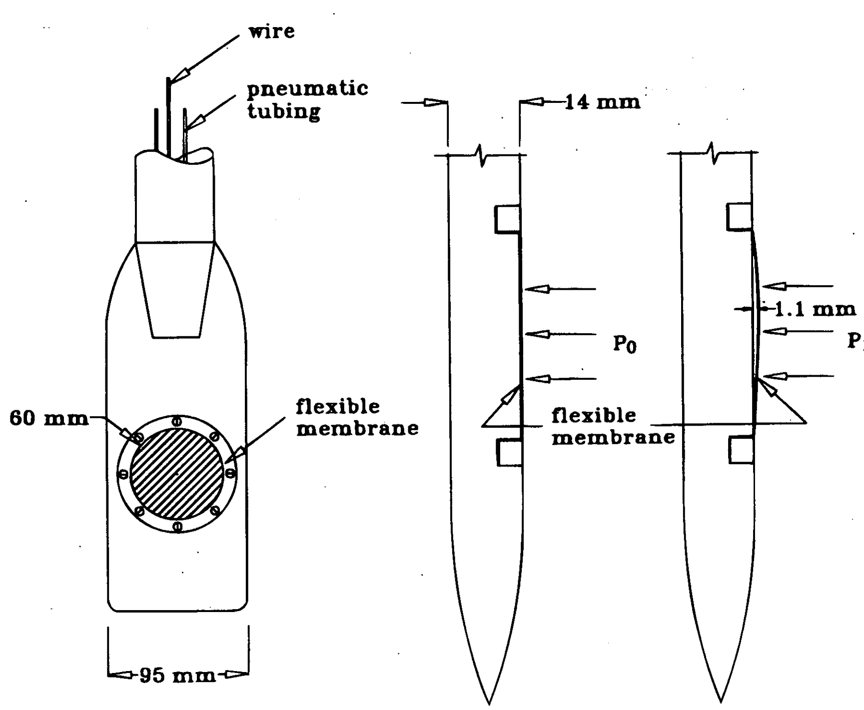


FIGURE 6 Marchetti dilatometer (*47*).

TABLE 2 Soil Classification Based on I_D Values (48)

	Soil Classification					
	Clay		Silt		Sand	
	Clay	Silty clay	Clayey silt	Silt	Silty sand	Sand
I_D value	0.10	0.35	0.6	0.9	1.8	3.3

Table 2 shows the soil classification according to I_D (42). Marchetti and many other researchers have proposed a series of empirical equations to relate soil parameters to the DMT indices. These soil properties include OCR, s_u, at-rest lateral earth pressure coefficient (K_0), constrained modulus (M), and initial modulus (E_i).

For uncemented OC clays ($I_D < 1.2$), resulted from simple unloading, Marchetti (42) proposed that

$$K_0 = (K_D/1.5)^{0.47} - 0.6 \tag{20}$$

and

$$OCR = (0.5\,K_D)^{1.56} \tag{21}$$

Following the concept of Ladd et al. (44), Marchetti (42) suggested estimating s_u based on

$$\frac{s_u}{\sigma'_{vo}} = 0.22(0.5K_D)^{1.25} \tag{22}$$

There is no unique relationship between M and E_D. If $R_M = M/E_D$ is considered, Marchetti (42) suggested that R_M increases with K_D, and proposed a series of empirical equations that relate these two parameters. However, because of the scatter of data, the validity of those equations is questionable.

More recent studies have proposed a linear relationship between E_i and E_D:

$$E_i = FE_D \tag{23}$$

For highly overconsolidated clays, Davidson and Boghrat (45) suggested that $F = 1.4$. For laterally loaded pile design, Robertson et al. (46) recommended that $F = 10$ for cohesive soils ($I_D < 1.0$). Lutenegger (47) suggested that F should decrease with I_D for clays.

Concluding Remarks

The CPT or CPTU is an efficient tool in establishing soil profiles and stratigraphy. Soil layers as thin as 5 mm could be identified with the help of pore pressure measurements in the CPTU (2).

The PMT is one of the few, if not the only, in situ testing method that measures a soil stress-strain curve. It has the potential of being very useful in predicting the performance of both the axially or laterally loaded foundations. The results of the PMT are sensitive to the quality of the borehole and the skills of the operator. It is thus imperative to follow the standard procedure as closely as possible and report details of the test method.

The FVT is a very useful tool in establishing the undrained shear strength profile of a clay deposit. Experience has indicated (35,36) that design of both the vane and test procedure can have significant effects on the test results. For the FVT results to be interpreted with meaning, it is important to report details of the test equipment and procedure utilized in the field.

Cases of foundation design or predicting field performance using DMT results have been reported (46). Most of these cases used conventional soil parameters derived from DMT data. Because of its efficiency and unique capability of measuring stress/stiffness in lateral direction, the DMT can be a very useful quality assurance tool for soil improvement operations (48).

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Profiling Yield Stresses in Clays by In Situ Tests

PAUL W. MAYNE

A unified approach for profiling the effective yield stress (σ_p') of natural clays by in situ tests is presented. The empirical methodology is developed from statistical regression analyses of databases involving soft-to-firm normally consolidated clays and stiff-to-hard overconsolidated clay deposits that have been tested using the cone penetration, piezocone, dilatometer, vane shear, pressuremeter, and standard penetration tests. Similar trends are established for field test results in intact clays, whereas deviations occur for fissured materials. The interpreted profiles of σ_p' from in situ tests are quick, economical, and continuous, yet should be verified by companion sets of laboratory oedometer tests on high-quality specimens. The procedures are applied to two case studies involving natural clay deposits near Washington, D.C., where reference consolidation test data were available.

The approximate magnitude of the effective yield stress or preconsolidation pressure ($\sigma_p' = \sigma_{v\max}' = P_c'$) of natural clays can be inferred from the results of in situ tests. This is advantageous because routine tube sampling and one-dimensional consolidation testing on retrieved specimens are often difficult, time-consuming, and expensive and suffer from sampling disturbance effects. However in situ tests are relatively fast and economical, and they test the soil in its natural environment and under its actual anisotropic geostatic stress state. Several theoretical premises for relating σ_p' to in situ test data have been postulated and are reviewed briefly in this paper. From a more practical vantage, first-order statistical relationships are presented for profiling the yield stresses of intact clays from field data obtained from cone, piezocone, vane, pressuremeter, dilatometer, and standard penetration tests. Similar trends are observed for intact clays, whereas fissured clay results are also affected by their macrostructure.

PRECONSOLIDATION STRESS

The maximum past vertical stress or effective preconsolidation pressure (P_c', σ_p', or $\sigma_{v\max}'$) is an important parameter defining the state of stress of clay deposits. Conventionally, this parameter is determined from standard one-dimensional oedometer tests on small specimens trimmed from samples taken from the field. The characteristic e-log σ_v' graphs from consolidation testing show a change in slope at a yield point termed the yield stress, henceforth designated σ_p'. The value of σ_p' separates overconsolidated states (elastic response) from the normally consolidated region (plastic response). Figure 1 shows results from a consolidation test on an overconsolidated sandy clay from Surry, Virginia, and the interpreted yield stress is 900 kN/m^2 at the reported depth of 27 m. It is important to impart sufficiently high stress levels during consolidation loading to define fully the normally consolidated region and magnitude of yield stress. Unfortunately, many commercial laboratories simply run oedometer tests using a standard set of stress increments regardless of the consistency and hardness of the clay, and therefore the tests do not completely reach the virgin compression line. In addition, sampling disturbance effects typically lower the overall e-log σ_v' curve from field conditions. Consequently, the value of σ_p' is often underestimated in routine testing and interpretation (*1*).

If the current state of vertical effective stress (σ_{vo}') is known, then the difference between the yield stress and current stress is referred to as the prestress ($\sigma_p' - \sigma_{vo}'$). Almost all natural soils have been prestressed to some degree because of geologic, environmental, or climatic processes that occur over long periods of time. Erosion, glaciation, desiccation, secondary compression, cyclic loading, groundwater fluctuations, and geochemical changes are typical mechanisms causing preconsolidation effects. In terms of dimensionless parameters, it is common to express the ratio of vertical stresses in normalized form, known as the overconsolidation ratio, $OCR = \sigma_p'/\sigma_{vo}'$. The advantages of this format are that no units are specified and the scaling laws of continuum mechanics can be used (*2*). For the data shown in Figure 1, the current $\sigma_{vo}' = 295$ kN/m^2 and therefore the in situ $OCR \approx 3$ for this marine clay from Virginia.

It should be noted that the stress state of soil is not merely a one-dimensional phenomenon. The use of routine consolidation tests with rigid lateral constraint and incremental application of vertical loads has proliferated because of the simplicity of equipment and test procedures. In reality, the stress history of natural materials is controlled, as a minimum, by a four-dimensional condition involving σ_x', σ_y', σ_z', and time (t). Series of extensive triaxial testing programs have found yield stresses associated with all types of stress paths. Consequently, recent studies in understanding soil behavior have developed the concept of a yield surface. That is, the stress history of natural materials is best characterized by a three-dimensional yield envelope that is rheological and changes as a function of time (age, creep, and strain rate). Figure 2 illustrates the yield surface for the Saint Alban Clay in Quebec (*3*), where typical index properties of the clay are liquid limit (LL) = 45, plasticity index (PI) = 20, water content (W_n) = 75, sensitivity (S_t) = 18, and OCR = 2.2. Here, the yield surface is presented in a Cambridge q-p' diagram, where $q = (\sigma_1 - \sigma_3)$ is the principal stress difference and $p' = \frac{1}{3}(\sigma_1' + \sigma_2' + \sigma_3')$ is the mean effective stress.

A review of yield surfaces from clays worldwide suggests that the effective frictional properties (ϕ') of the clay primarily govern the actual shape of the envelope (*4*). The well-known preconsolidation pressure or yield stress (σ_p') is but one point on the yield surface where the locus crosses the K_{0NC}-line corresponding to normally consolidated conditions. Available stress path and effective

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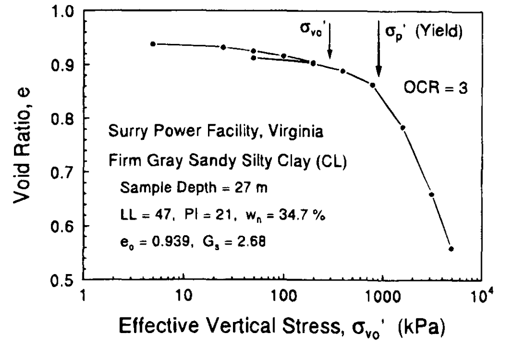

FIGURE 1 Yield stress observed in one-dimensional consolidation test on Surry Clay.

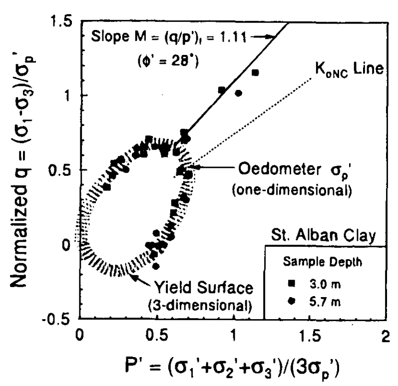

FIGURE 2 Three-dimensional yield surface for St. Alban Clay [after Leroueil et al. (3)].

stress strength data suggest that the shape of the yield surface is actually best represented as a rotated ellipse in MIT q-p' space, where the same terms imply different parameters: $q = \frac{1}{2}(\sigma_1' - \sigma_3')$ and $p' = \frac{1}{2}(\sigma_1' + \sigma_3')$. The yield surface concept is particularly useful in explaining the nonuniqueness of obtaining Mohr-Coulomb parameters (c' and ϕ') from limited numbers of strength tests (*5*), as well as the observed degradation of effective cohesion intercept (c') with time (*6*).

IN SITU TESTS

Traditionally, in situ test measurements in clay are used toward evaluating a value of undrained shear strength (s_u) of the deposit. The undrained strength, however, is a nonunique behavioral response of the material that, for a given stress state, depends on strain rate, boundary conditions, and direction of loading. Each particular test method (triaxial, plane strain, simple shear, vane, etc.) must therefore provide a different value of s_u for the same clay (*2*). In past correlative studies, s_u values from various tests often were used interchangeably without due regard to their differences. As a consequence, inconsistent interpretations of s_u have arisen in comparing clays of varied origins and backgrounds. As an alternative to the approach of evaluating s_u, it is suggested that field test data be used to infer the yield stress state of the clay, specifically the uniquely defined σ_p' obtained from one-dimensional oedometer tests.

The detailed profiling of yield stresses from consolidation tests at a particular site is often expensive and can require weeks or months of testing. High-quality samples must be retrieved from the field, carefully transported and stored, and subjected to incremental-load oedometer or constant rate-of-strain consolidation tests at approximately $400/test. Consequently, it is of interest to use in situ tests for this purpose because they are quick and inexpensive and provide nearly continuous and immediate results for analysis. However, the use of in situ tests for this purpose is not without other difficulties. For example, the actual stress path followed during the test is not known, rates of strain are very high, destructuration occurs, and the field measurements are affected by a variety of other soil conditions (fabric, sensitivity, mineralogy, plasticity, etc.) in addition to stress history. Nevertheless, merit in the use of in situ tests for evaluating σ_p' offers an expedient and economical approach that may supplement the results of conventional consolidation testing.

As early as 1957, Hansbo (*7*) suggested the use of field vane measurements for determining σ_p' in the very sensitive clays of Scandinavia. Later in 1979, Tavenas and Leroueil (*6*) indicated that the cone tip resistance (q_c) could be used reasonably to map values of σ_p' with depth in the sensitive clays of Eastern Canada. In 1980 the dilatometer test was introduced for profiling OCRs (*8*). Since that time a number of data bases involving a variety of field tests in clays were compiled where reference profiles of yield stresses were available from companion series of oedometer tests performed on undisturbed samples. The clay sites, sources of data, and specific details on the compilation of information have been presented elsewhere (*9,10*) and include normally consolidated to overconsolidated clays that range from soft to stiff to hard, intact to fissured materials. Many worldwide locations of diverse geologic origins are contained in the collections: marine, glacial, deltaic, lacustrine, alluvial, and diluvial. Differences in origin, plasticity, sensitivity, and age are also likely to be notable factors, but they are not discussed here.

THEORETICAL BASIS

Simple analytical models, as well as complex numerical simulations, have proved useful in relating σ_p' to the results of in situ measurements. The axisymmetric geometry of the cone penetrometer has permitted analyses interrelating OCR and normalized cone resistance via cavity expansion models (*11*), finite elements (*2*), and strain path methods (*12,13,17*). Piezocone penetration has been evaluated by effective stress analysis (*14*), limit plasticity (*15*), cavity expansion (*16*), and strain path methods (*17*). The dilatometer test (DMT) has also been evaluated by strain path techniques (*17,18*). An approximate approach for interrelating σ_p' and the results of piezocone, dilatometer, and pressuremeter in terms of concepts of cavity expansion versus critical state has been proposed (*19*).

The theoretical relationships for cone, piezocone, pressuremeter, and dilatometer are supported by statistical regression analyses conducted on specific data bases compiled for each in situ test. Similarly, trends for evaluating σ_p' from the vane shear test and standard penetration test are also available, albeit only on an empirical basis. The following section describes the observed average trends for these data bases.

STATISTICAL RELATIONSHIPS

Regression analyses have been conducted on data bases from intact clays only and were performed using two separate procedures. First, an assumed arithmetic-arithmetic pattern between independent and dependent variables was assessed. In these cases, only a small intercept value was determined from linear regressions. Therefore, a best-fit line ($b = 0$; $y = mx$) from least-squares analysis was conducted so that only one variable was obtained in the formulation (Table 1). This facilitates a direct comparison among the various in situ tests because σ_p' is always the dependent variable in this study. The number of data sets (n) and coefficient of determination (r^2) from the regression analysis are reported in each subsequent graph as well as in Table 1. The standard error of the dependent variable (equivalent standard deviation, or SD) for each regression is also given in the figures. Second, an assumed log-log relation was analyzed to give a power function format. That is, if b = intercept and a = slope, then a natural log base regression gives $y = e^b x^a$. In these cases, it was observed that $a \approx 1$ and the results could be adjusted to force the format $y = mx$, where $m = e^b$ and $a = 1$. This best-fit line approach permits a direct comparison between the arithmetic and logarithmic regression models, and an examination of the expressions in Table 1 indicates remarkable similarity between the expressions obtained from both types of regression studies.

In the following figures, log-log axes have been presented to show the vast range of data spanning nearly three orders of magnitude for each variable. If the data were shown arithmetically, several figures would be required to present all of the data. Both the ordinate and abscissa have been made dimensionless by normalization to a reference value equal to atmospheric pressure (p_a = 1 bar $\approx$ 100 kPa $\approx$ 1 tsf $\approx$ 1 kg/cm^2). The dependent variable (dimensionless yield stress or σ_p'/P_a) shown on the ordinate has a significant range that appears to be best represented as log normally distributed ($0.2 \leq \sigma_p'/P_a \leq 50$). Because the arithmetic-arithmetic and log-log relationships generally are similar, the simpler arithmetic statistical expression has been presented in each figure. Statistical results given are for intact clays only.

TABLE 1 Statistical Trends Between σ_p' and In-Situ Tests for Intact Clays

TEST	n	Best Fit Line (b = 0) Arithmetic Relation	r^2	Best Fit Line (m = 1) Log-Log Relation	r^2
VST	205	$\sigma_p' = 3.54\ s_{uv}$	0.832	$\sigma_p' = 4.14\ s_{uv}$	0.759
CPT*	113	$\sigma_p' = 0.287\ q_c$	0.858	$\sigma_p' = 0.240\ q_c$	0.863
ECPT	74	$\sigma_p' = 0.323(q_T - \sigma_{vo})$	0.904	$\sigma_p' = 0.342(q_T - \sigma_{vo})$	0.875
PCPT	77	$\sigma_p' = 0.467\ \Delta u_t$	0.838	$\sigma_p' = 0.461\ \Delta u_t$	0.884
PCPT	68	$\sigma_p' = 0.537\ \Delta u_{bt}$	0.827	$\sigma_p' = 0.474\ \Delta u_{bt}$	0.816
DMT	76	$\sigma_p' = 0.509(p_o - u_o)$	0.896	$\sigma_p' = 0.574(p_o - u_o)$	0.901
SPT	126	$\sigma_p' = 0.468\ N_{60} p_a$	0.699	$\sigma_p' = 0.517\ N_{60} p_a$	0.804
PMT	89	$\sigma_p' = 0.454\ p_L$	0.908	$\sigma_p' = 0.343\ p_L$	0.797
PMT	105	$\sigma_p' = 0.755 s_{up} \ln I_r$	0.895	$\sigma_p' = 0.595 s_{up} \ln I_r$	0.873

NOTES:

VST	= vane shear test	σ_p'	= effective yield stress
CPT	= cone penetration test	s_{uv}	= vane shear strength
ECPT	= electric cone	q_c	= measure cone tip resistance
PCPT	= piezocone test	q_T	= corrected cone tip resistance
DMT	= dilatometer test	σ_{vo}	= total overburden stress
SPT	= standard penetration test	u_t	= pore pressure on cone face
PMT	= pressuremeter test	u_{bt}	= pore pressure behind cone tip
p_a	= atmospheric pressure	p_o	= DMT contact pressure
n	= number of data sets	N_{60}	= energy-corrected N-value
r^2	= coefficient of determination	p_L	= limit pressure
*	= data includes mechanical and electrical cones	s_{up}	= pressuremeter undrained strength
		I_r	= G/s_u = rigidity index
b	= regression intercept	G	= shear modulus
m	= regression slope	Δu	= excess pore water pressure

Cone Penetration Tests

For cone penetration test (CPT), Figure 3 indicates the observed direct trend between σ_p' and net cone tip resistance ($q_T - \sigma_{vo}$). The specific symbols shown refer to the individual sites listed in the compiled data base (*20,21*). All measured cone tip resistances ($q_c \rightarrow q_T$) have been corrected for pore-water pressure effects acting on unequal areas of the cone tip (*20,21*). As noted previously, data are presented in log-log format to show that the relationship covers almost three orders of magnitude in the full range of values both for the ordinate (σ_p'/P_a) and for the independent variable plotted on the abscissa, $1 \leq (q_T - \sigma_{vo})/p_a \leq 60$. It is clear that a well-defined trend occurs between σ_p' and ($q_T - \sigma_{vo}$) for intact clays, but the data for fissured clays fall above this relationship. This is important since a macrofabric of discontinuities exists because of high desiccation effects or passive failure of heavily overconsolidated materials.

Variations are evident in the trend between σ_p' and net cone resistance and, undoubtedly, additional factors play a significant role in the relationship. For an initial first-order estimate, the regression studies for the CPT data indicate the following average trend for intact clays:

$$\sigma_p' = 0.33(q_T - \sigma_{vo}) \tag{1}$$

This is comparable with the originally proposed relationship ($\sigma_p' \approx q_c/3$) observed for structured natural clays in Quebec (*7*) and is consistent with a more recent evaluation of sensitive Swedish clays (*23*) where $\sigma_p' = 0.29\ (q_T - \sigma_{vo})$. The latter suggests further that clay plasticity also affects the interrelationship for normally consolidated clays. For fissured clays, Equation 1 provides a conservative estimate of preconsolidation, and the appropriate coefficient linking σ_p' and ($q_T - \sigma_{vo}$) may be dictated by other factors, such as degree of fissuring, age, and mechanism of overconsolidation.

Piezocone Tests

The magnitude of pore-water pressures measured by piezocone (PCPT) depends upon the specific position of the porous element on the cone geometry. For simplicity, Type 1 piezocones are classified as having the pore pressures measured on the cone tip/face, whereas the elements on Type 2 piezocones are located just behind the tip (shoulder). The noted trend between σ_p' and measured excess pore-water pressures (Δu_t) from Type 1 piezocones is shown in Figure 4 (*a*). The solid dots correspond to clay sites and sources of data cited by Mayne et al. (*20,21*). It is observed repeatedly that intact and fissured clays show distinct behavioral patterns in response. Note that all pore pressures are positive for Type 1 piezocones regardless of whether the clays are soft or hard. The average relationship for intact clays gives

$$\sigma_p' = 0.47\Delta u_t \tag{2}$$

A similar format was investigated by Larsson and Mulabdic (*23*) for Scandinavian clays that indicated an average coefficient term of 0.29 in very sensitive clays of high plasticity and about 0.21 in gyttja (highly organic clays). For Type 2 piezocones, which measure penetration pore-water pressures behind the cone tip (U_{bt}), the trend for evaluating σ_p' is shown in Figure 4(*b*) and indicates for intact clays:

$$\sigma_p' = 0.54\Delta u_{bt} \tag{3}$$

Considering both intact and fissured clays, Type 2 piezocones show a nonunique pattern of excess pore pressure development. At low to moderate OCRs, σ_p' increases with Δu, whereas at higher OCRs in fissured clays, piezocones typically measure zero or negative excess pore pressures during penetration. Thus, although Type 2 PCPTs are necessary for cone tip corrections, they often provide little stratigraphic detail in overconsolidated fissured clays. In this regard, dual- and triple-element piezocones can provide all of the essential information for detailed logging of strata.

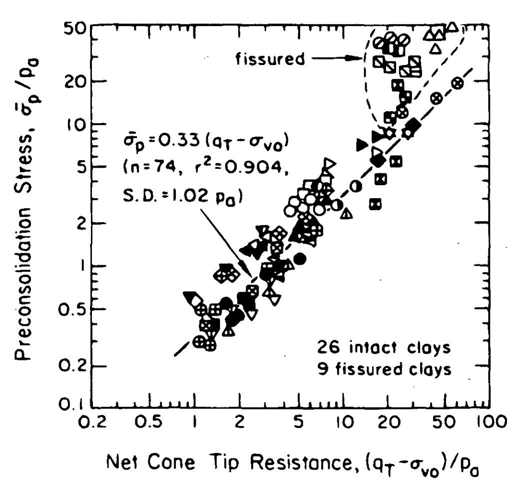

FIGURE 3 Graphical log relationship between σ_p' and ($q_T - \sigma_{vo}$) for cone penetration tests. (Note: arithmetic statistical trend given for intact clays.)

Dilatometer Tests

The dilatometer test (DMT) provides direct measurements of total lateral stress immediately after insertion of a flat steel blade. The normal procedure for estimating yield stress in clays relies on the original correlation proposed by Marchetti (*8*) between OCR and DMT horizontal stress index, $K_D = (p_o - u_o)/\sigma_{vo}'$, where P_o is the corrected DMT contact pressure or A-reading and u_o is the hydrostatic water pressure. The correlation is based on the results of data from only five clays. The magnitudes of induced total stress and pore pressure at this instant of penetration by a probe are quite similar both theoretically (cavity expansion) and experimentally (*24*). Consequently, a relationship for the DMT is expected to be similar to that from PCPT. The statistical trend for the DMT data base compiled from 24 different intact natural clays is

$$\sigma_p' = 0.51(p_o - u_o) \tag{4}$$

The relationship is presented in Figure 5(*a*) and suggests a generalized form:

$$\sigma_p' = (p_o - u_o)/\delta \tag{5}$$

where $\delta = 2$ corresponds to the mean trend. However, the observed variation noted by Mayne and Bachus (*19*) indicates a typical range

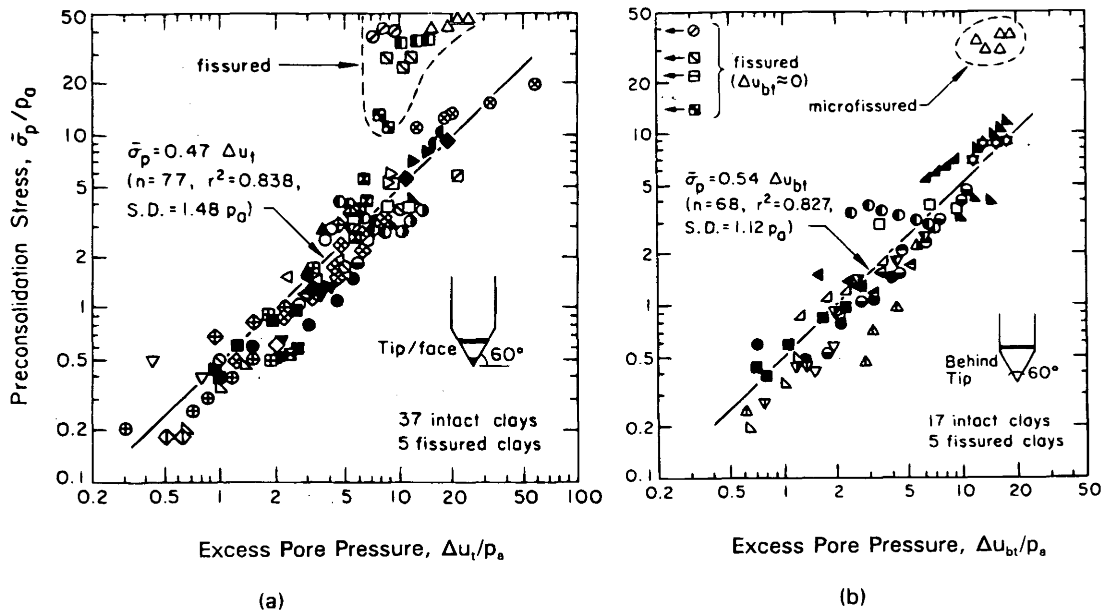

FIGURE 4 Graphical log relationships between σ_p' and Δu for piezocone penetration tests with (*a*) face and (*b*) shoulder pore pressure measurements. (Note: arithmetic statistics given for intact clays.)

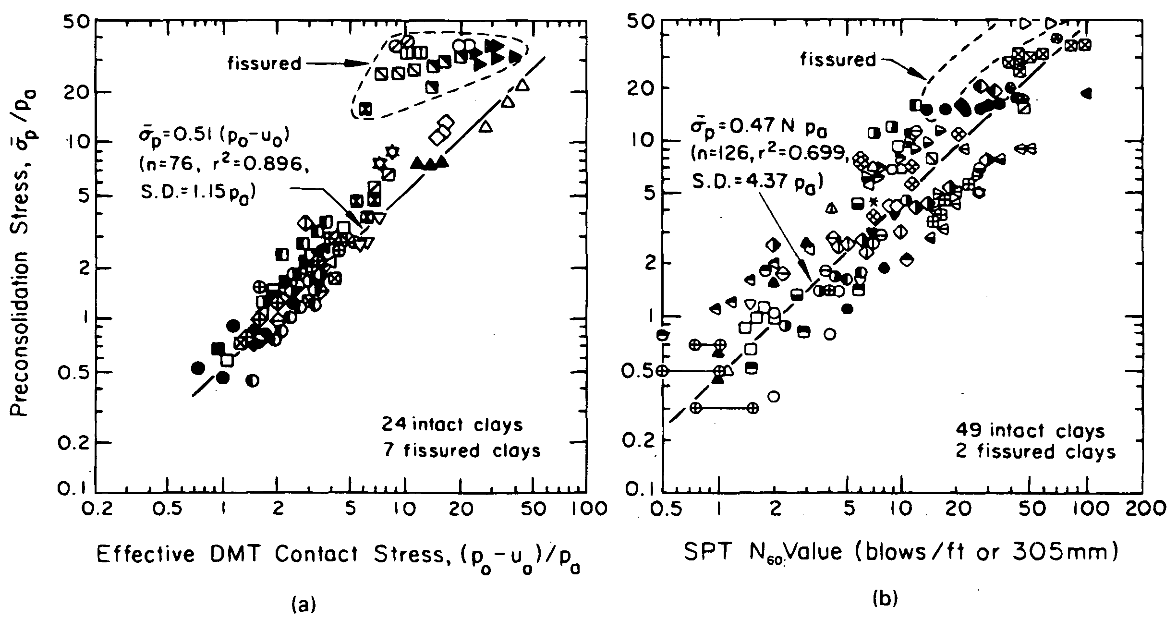

FIGURE 5 Graphical log relationships between σ_p' and (*a*) effective dilatometer contact stress ($p_o - u_o$) and (*b*) energy-corrected N_{60} from standard penetration test. (Note: arithmetic statistics given for intact clays.)

of $1.5 \leq \delta \leq 2.5$ for intact clays, whereas the data for heavily OC fissured clays suggest an average $\delta = 0.50 \pm 0.25$. The generalized form appears to be applicable to other in situ tests as well.

Pressuremeter Tests

In routine practice, pressuremeter tests (PMT) are used to better define soil parameters for analysis and to provide reference values for other field tests. Detailed profiling of properties with depth is usually neither cost-effective nor production-oriented. The PMT is useful in overconsolidated clays because it can provide independent evaluations of total horizontal stress ($P_{ho} = \sigma_{ho}$), shear modulus (G), undrained shear strength (s_u), and limit pressure (p_L). The PMT mimics an expanding cylindrical cavity, and the equivalent excess pore pressure calculated from this test can be related to yield stress (*19*). A review of data from self-boring pressuremeter tests in 44 different clays (*10*) indicated

$$\sigma_p' = 0.755 s_u \ln(G/s_u) \qquad (6)$$

Alternatively, the limit pressure (p_L) may be related to the yield stress of the deposit, as noted in Table 1. Associated graphs for these data trends have been presented elsewhere (*9*).

Vane Shear Tests

In Sweden the vane shear test (VST) is routinely used for evaluating profiles of σ_p' in soft sensitive clays (*6*). Using a data base derived from 96 different clay sites (*25*), the average regression relationship between σ_p' and measured s_{uv} from vane tests is

$$\sigma_p' = 3.54 s_{uv} \tag{7}$$

The data trend is in agreement with a recent statistical study of VSTs in four types of sensitive clays in eastern Canada (*26*). The aforementioned relationship is improved if the effects of plasticity are considered in the following empirical format:

$$\sigma_p' = 22 s_{uv} / \mathrm{PI}^{0.48} \tag{8}$$

which is consistent with a state-of-the-art and independent review on the VST by Chandler (*27*). Graphical presentations of the σ_p' versus s_{uv} trends have been made (*9,25*).

Standard Penetration Test

The variation in energy efficiency accounts for much of the observed widespread differences in standard penetration test (SPT) results. Even though the SPT remains a common in situ test in U.S. practice, a majority of testing firms and consulting engineers fail to calibrate the device for energy measurements. The SPT-N value from the test is severely altered by hammer type (pinhead, safety, or donut), drop height, number of rope turns, lifting system (cathead versus automatic trip), age of rope, driller, and other factors. Correlations over the past 40 years have been based on an average energy efficiency of 60 percent, and the energy-corrected SPT resistance is designated N_{60}. Until drillers and engineers adopt a calibration program for each rig, the SPT will remain a relatively uncertain and unreliable test for accurate measurements.

In this study, SPT data from 51 different clays were reviewed (*28*). The trend for σ_p' is shown in Figure 5(*b*) and regression analyses gave the poorest statistics (highest SD and lowest r^2) of any of the in situ test relationships:

$$\sigma_p' = 0.468 N_{60} p_a \tag{9}$$

where N_{60} assumes average U.S. practice. It is noted, unfortunately, that few of the N values were actually corrected for energy efficiency. Recently Decourt (*29*) proposed a similar format between σ_p' and N for Brazilian clays, where the coefficient term equals 0.28. In this manner, each of the aforementioned correlations could be adjusted to site-specific conditions, as calibrated against oedometer results on high-quality tube or block samples.

CASE STUDY APPLICATIONS

The aforementioned methodology has been used to profile σ_p' at two sites where in situ testing was conducted for site characterization of clay deposits in the Washington, D.C., area.

Suitland, Maryland

The Smithsonian Support Center in Suitland, Maryland, required detailed analysis of the yield stress profile of the subsurface soils for supporting several large warehouses, offices, and showrooms on shallow foundations (*30*). The site is underlain by an interesting geology of layered and varied sedimentary strata formed during different epochs. Figure 6(*a*) illustrates the sequence of strata and typical index properties of the clays. The uppermost 6 m of very dense sands and gravels required preboring. Below these terrace deposits, soft sandy clays of Miocene and Eocene age were encountered that were underlain by very stiff overconsolidated clays of Cretaceous age. Groundwater is approximately 6 m deep at the site. Typical SPT resistances were 4 to 6 blows/300 mm in the Calvert and Aquia

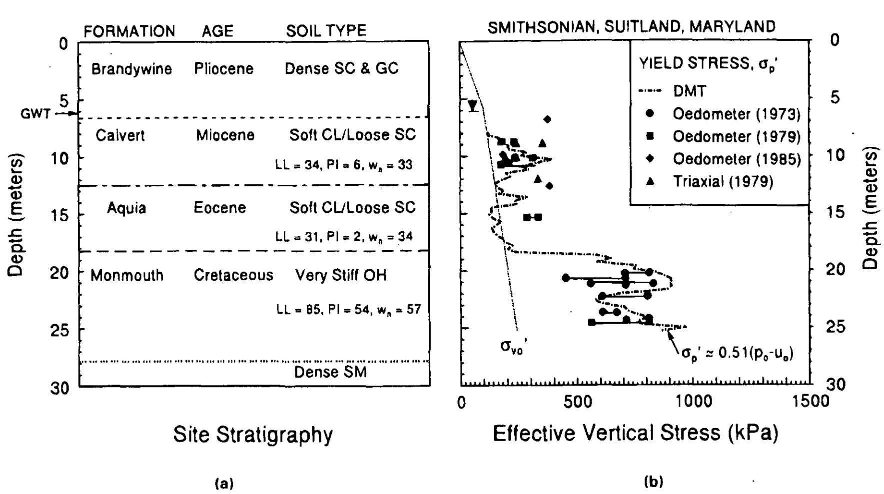

FIGURE 6 Field data at Smithsonian Center, Suitland, Md.: (*a*) general stratigraphy; and (*b*) yield stresses from oedometer and dilatometer tests.

formations and about 12 to 16 blows/300 mm in the underlying Monmouth formation. The reference preconsolidation stresses of the clays were reconstructed using several series of oedometer tests made on recovered specimens taken during different exploration programs at the site. These were supplemented by interpretations of σ_p' using CIUC triaxial test data (*31*). Figure 6(*b*) illustrates the use of DMT data via Equation 4 for estimating the yield stresses at the site. The DMT clearly indicates the dramatic changes in σ_p' between different strata. It is also noted that the original correlation (*8*) gives estimated σ_p' that are twice that of Equation 4 in the lower Cretaceous unit. In the original expression, OCR was correlated with the dilatometer index $K_D = (p_o - u_o)/\sigma_{vo}'$, based on data from only eight clays. The direct expression for σ_p' in terms of net contact stress $(p_o - u_o)$ given herein is based on data from 31 sites, which is statistically specific to intact clays.

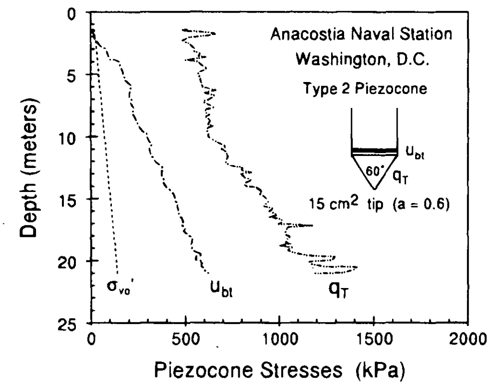

FIGURE 7 Piezocone data in alluvial clay at Anacostia Naval Air Station, Washington, D.C.

Anacostia, Washington, D.C.

The construction of a new helicopter hangar and communications facility at the Anacostia Naval Air Station in Washington, D.C., required additional characterization of the lightly overconsolidated alluvial organic clayey silts (LL = 83; PI = 37, w_n = 68) that extend up to 30 m in depth. The site is located at the confluence of the Potomac and Anacostia rivers, and the results of in situ dilatometer tests and laboratory consolidation tests were reported previously (*24*). Cone tip resistances and penetration pore pressures from Type 2 piezocone soundings with a 15-cm^2 tip are presented in Figure 7. These readings can be used via Equations 1 and 3 to provide estimated profiles of σ_p' with depth, as shown in Figure 8, and the clayey silts have corresponding overconsolidation ratios in the range of 1.5 to about 4 (*32*). An examination of the data in Figure 8 shows that the tip resistance slightly overpredicted σ_p', and the pore pressure data slightly underpredicted σ_p'. A partial explanation is the uncertainty associated with the relatively large correction (*22*) based on the net area ratio for this cone (a = 0.60); thus, q_T may be a bit high. Also, saturation was accomplished with water (although glycerine has proven better) and may have been less complete because of the larger diameter cone, thus yielding a lower registration with the Δu readings. In any event the estimates from the two independent piezocone readings are in relative agreement with the reference oedometer results.

DISCUSSION OF RESULTS

Statistical relationships have been presented between σ_p' and each of the measured in situ test parameters. Variance in the trends is evident for these relationships, as noted in the original database sources (10, 11, 19–21, 24, 25, 28). One intent of this study was to acknowledge that a simple unified approach exists that is common to several types of in situ tests. Care should be taken in the use of any empirical relationships, however, and site-specific calibration of in situ tests with oedometer test results is warranted and recommended. In this regard, a more generalized format may be more useful and prudent for interrelating σ_p' to in situ parameters:

$$\begin{array}{ccccc} & \text{PCPT1} & \text{PCPT2} & \text{DMT} & \text{PMT} \\ \sigma_p' \approx & \dfrac{\Delta u_t}{\delta} \approx & \dfrac{1.2\Delta u_{bt}}{\delta} \approx & \dfrac{1.1(p_o - u_o)}{\delta} \approx & \dfrac{1.6 s_u \ln I_r}{\delta} \end{array}$$

$$\begin{array}{cccc} & \text{CPT} & \text{VST} & \text{SPT} \\ \approx & \dfrac{0.7(q_T - \sigma_{vo})}{\delta} \approx & \dfrac{7.6 s_{uv}}{\delta} \approx & \dfrac{N_{60} p_a}{\delta} \end{array} \quad (10)$$

where experimentally observed values are typically in the range $1.5 < \delta < 2.5$ for intact clays, whereas for fissured materials the range is generally $0.4 \leq \delta \leq 0.8$. From a theoretical consideration of cavity expansion and critical state concepts (*19*), the value of δ for intact soils can be approximately obtained as

$$\delta = (4/3)(\phi'/100)\ln(I_r) \quad (11)$$

where ϕ' = effective stress friction angle and $I_r = G/s_u$ = rigidity index of the clay. However, the relevant values of these parameters often are not known a priori during in situ testing, and therefore the mean statistical trends presented may be useful. More rigorous and fundamental relationships have been developed for certain tests, such as the piezocone (*17,32,33*) and dilatometer (*10,18*), and these

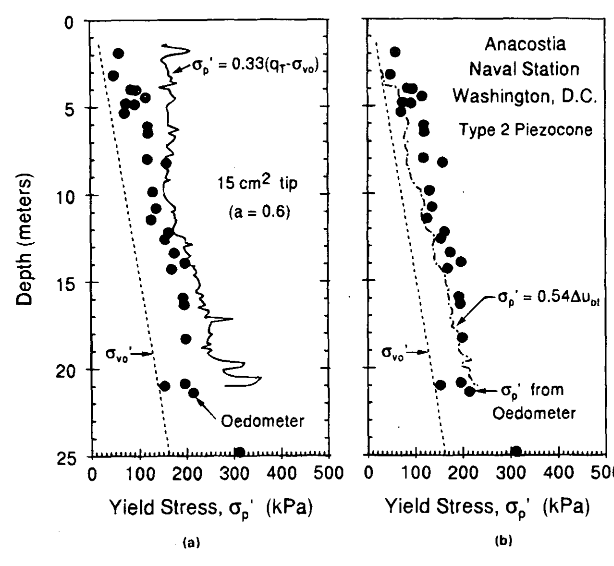

FIGURE 8 Comparison of measured and interpreted yield stresses from (*a*) cone tip resistance and (*b*) penetration pore pressures.

use the normalized parameter OCR for more proper representation of stress history effects (*2*).

CONCLUSIONS

In situ tests may be used to profile the approximate value of effective yield stresses (σ_p') in natural clay deposits. Statistical relationships are presented for evaluating σ_p' from cone, piezocone, dilatometer, pressuremeter, vane, and standard penetration test data in intact clays. The trends illustrate the existence of a unified approach common to all tests. Conservative estimates are obtained in fissured deposits because the discontinuities affect the in situ measurements. Advantages of the approach include the immediate, continuous, and economical evaluation of the stress state of cohesive materials, in contrast to difficulties associated with sampling, disturbance, handling, and long testing times for laboratory testing. However, reference values of σ_p' should always be obtained from conventional consolidation tests to verify that the empirical procedures are valid. A generalized format is outlined to permit site-specific calibration with in situ test data.

ACKNOWLEDGMENTS

Funding for this research on in situ testing was provided by the National Science Foundation. The author gratefully acknowledges the support of Mehmet T. Tumay, currently director of the Louisiana Transportation Research Center. Appreciation is given to Fred H. Kulhawy of Cornell University for helping to develop the statistical relationships.

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Overview of State-of-the-Practice Modeling of Overconsolidated Soils

EMIR JOSE MACARI AND PEDRO ARDUINO

Numerical methods currently used in practice to predict the behavior of overconsolidated clays are described. The paper is not intended to be a state-of-the-art report but rather a state-of-the-practice report on techniques that have been used in practice. The discussion focuses on the Modified Cam Clay (MCC) model, which is widely accepted because of its practicality and simplicity. However, there are instances when more sophisticated overconsolidated soil models have been used. The paper presents an overview of the developments of soil elasto-plasticity followed by a detailed description of the MCC model and derivation of its incremental formulation. In addition, an example describing the calibration of the MCC parameters is presented. Step-by-step procedures are developed for drained and undrained predictions. A generalized form of the MCC model is described in which a third stress invariant is included in the formulation. A constitutive driver code is then implemented to allow for the numerical simulation of three-dimensional stress states. Finally, a brief description of the implementation of a constitutive driver into a finite element formulation is presented.

Traditionally, problems of soil mechanics have been divided into two groups: deformation problems and stability problems. The first group deals with the stress-strain, or load-deformation, of a soil mass before failure. Some of the problems that are considered in this category are stresses at a point in a soil mass under a structure, excavations, and settlement problems. Solutions to these problems have been obtained with the aid of the theory of linear elasticity because it has been assumed that small deformations produce a nearly linear elastic response of the soil mass. Stability problems, on the other hand, deal with the conditions of ultimate failure of a soil mass, and among these, one could list such problems as stability of slopes, earth pressure against lateral support, and bearing capacity of footings. The main issue related to stability problems is the determination of the loads that will cause failure of the structure or soil mass. In classical soil mechanics these problems have been solved by what is known as the theory of perfect plasticity or ultimate strength.

In the past few decades a third category of problems has emerged that is a combination of the first two problems, referred to as "progressive yielding." These problems deal with the theory of elasto-plasticity as soils deform from an initial elastic limit to an ultimate or critical state.

Initial attempts to model the mechanical behavior of soils have been attributed to Drucker et al. (*1*). This approach described the behavior of soil with constitutive equations using the framework of continuum mechanics. Researchers at the University of Cambridge (*2–4*) continued these efforts during the 1960s, leading to the framework of critical state soil mechanics (CSSM). Constitutive models such as the Cam Clay model (*3*) and the Modified Cam Clay (MCC) model (*5*) were developed on the basis of plasticity theory within the framework of CSSM. These models have been widely used because of their simplicity; however, there are many aspects of soil behavior that they fail to capture.

Over the last 20 years a great deal of effort has been dedicated to the development of more realistic constitutive models for soils. This emphasis is evident by the number of specialty conferences and prediction workshops that have been dedicated to modeling issues. There is a great deal of knowledge in the area of constitutive modeling of soils. However, as the models have become more and more advanced, so has the level of complexity increased. As a result these developments have had very limited impact on the practicing geotechnical engineering profession. In recent years these trends have reversed and there have been some efforts to incorporate the knowledge on constitutive modeling of soils into the actual practice of geotechnical engineering. Therefore emphasis has been placed on simplicity. The relationships between strain and stress (constitutive relations) have been implemented into finite element programs to solve engineering problems formulated as boundary-value problems.

Duncan (*6*) presented a summary of 100 publications in which advanced numerical models were used in practice for the analysis of the response of geotechnical structures. The models discussed in Duncan's paper ranged in complexity from very simple hyperbolic model formulations to more advanced models such as the MCC model and finally to more sophisticated models that incorporate failure theory into the analysis (shear band or bifurcation analysis). In addition, powerful pre- and postprocessors are being developed to aid the practicing engineer in gaining a better understanding of the response of earth structures as they are subjected to external loads or deformations within the context of finite element analysis. With these issues in mind, an attempt is made to demonstrate the need for advanced numerical models when dealing with overconsolidated soils. The main intent is to present some commonly used analytical methods and to explain how numerical (elastoplastic) soil models, specifically the MCC model, are implemented into displacement-based finite element techniques that may be used for the analysis of the response of earth structures. In so doing, an attempt is made to close the gap between research-driven developments in computational geomechanics and practical engineering.

SOIL MODELING

Although the number of soil models is too large to be described in detail here, the basic theories on which they are founded may be listed as follows (*7*):

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- Elasticity,
- Hyperbolic,
- Rate-type,
- Plasticity (single/multiyield surface),
- Plasticity (bounding surface), and
- Endochronic.

The formal mathematical theory of plasticity, originally presented by Hill (*8*), Prager and Hodge (*9*), Drucker (*10*), and others was developed to describe the mechanical response of metals. This theory has a tangible physical meaning for metals. However, as the theory was adapted to soils, which are pressure-sensitive materials because of the presence of voids, yielding was related to both mean effective stress as well as shear stresses, always including the effects of volume change.

However, even though there is no true physical definition for the theory of plasticity for soils, there is plenty of evidence to suggest that a reasonably good representation of the response of soils is obtained for normally consolidated clays (*11*). Hence, plasticity has become a widely used theory for modeling soil behavior. One of the most widely used plasticity models has been the MCC model (*5,12*). This model was originally developed for normally consolidated clays; however, it has also been used for overconsolidated clays. The response of highly overconsolidated soils predicted by MCC is governed by elastic behavior in the prepeak regime followed by a slight elastoplastic softening branch until the critical state is attained. This is a major shortcoming of this model. However, as stated earlier, the MCC model is one that is well understood and easily implemented.

DESCRIPTION OF MCC MODEL

Some important parameters used in the development of the Cam-Clay and MCC models are p', the mean effective stress; q, the stress difference (related to the octahedral shear stress or the second invariant of the deviatoric stress tensor); and e, the void ratio, or $v = 1 + e$, the specific volume. In a triaxial stress space one may express p' and q as $p' = (\sigma_1' + 2\sigma_3')/3$, $q = \sigma_1' - \sigma_3'$.

In an attempt to study the yielding behavior of normally consolidated clays, Roscoe et al. conducted tests on samples of saturated clays. The effective stress paths for several undrained tests were geometrically similar, and their ultimate stress states were observed to be on a straight line in a $q - p'$ space.

When a saturated soil sample is sheared, it experiences progressive states of yielding before reaching a state of collapse. That is, the stress path passes through several yield surfaces (hardening caps), causing plastic deformations. The yielding continues to occur until the material reaches a critical void ratio, after which it remains constant during subsequent deformations. That is, the material will pass through a state in which the arrangement of the particles is such that no volume change takes place during shearing. This particular void ratio is called the *critical void ratio* and is considered the ultimate state of the material. It has been observed that a soil with a void ratio lower than the critical value will deform in such a manner as to increase its volume, whereas at a void ratio higher than the critical value, the deformations will decrease in volume.

The MCC model is founded on the incremental plasticity theory, which provides stress-strain relationships that can be obtained by defining the four essential components of an elastic-plastic model: (*a*) Elastic properties; (*b*) yield surface (criterion); (*c*) plastic potential; (*d*) hardening rule.

Unloading and reloading of a soil is assumed to be elastic. That is, there is a linear relation between the specific volume (or void ratio) and the logarithm of the effective mean stress p', such that

$$\delta\epsilon_p^e = \kappa \frac{\delta p'}{vp'} \tag{1}$$

where κ is a model parameter similar to the swelling index and v is the specific volume ($e + 1$).

Also, it assumed that the elastic shear strains result from any change in the deviator stress q such that

$$\delta\epsilon_q^e = \frac{\delta q}{3G} \tag{2}$$

where G is a constant shear modulus.

The yield criterion defines the limit of purely elastic behavior. When the state of stress comes in contact with the current yield surface, the material undergoes elastic-plastic deformations.

The MCC model yield surface is represented by an ellipse (Figure 1) given by

$$\frac{p'}{p_o'} = \frac{M^2}{M^2 + \eta^2} \tag{3}$$

where $\eta = q/p'$, p_o' is the isotropic preconsolidation stress, and M is the slope of the critical state line (failure envelope). As the soil yields p_o' increases (expanding the yield surface) and this increase is linked with changes in the effective stresses p' and q through the differential form of the yield function. The yield function can also be rewritten as

$$f = q^2 - M^2[p'(p_o' - p')] = 0 \tag{4}$$

The MCC model assumes that the soil obeys the normality condition (which essentially describes the ratio of shear to volumetric plastic strain increment as the soil yields). Therefore, the flow rule is associative and mathematically is simply given by the slope of a normal to the yield surface at the present stress state (shown in Figure 1) as

$$\frac{\delta\epsilon_p^p}{\delta\epsilon_q^p} = \frac{\partial g/\partial p'}{\partial g/\partial q} = \frac{M^2(2p' - p_o')}{2q} = \frac{M^2 - \eta^2}{2\eta} \tag{5}$$

as plastic deformations occur.

As the soil yields, it hardens, and this hardening is linked to the increase of the isotropic preconsolidation stress p_o'. This hardening relationship is assumed to be linear such that

$$v = N - \lambda \ln p_o' \tag{6}$$

where N is a model parameter that indicates the location of the isotropic compression in the $p' - v$ space (N is the value of v for the value of $\ln(p') = 0$ or $p' = 1$). Therefore, the magnitude of the plastic volumetric strains is given as

$$\delta\epsilon_p^p = [(\lambda - \kappa)/v] \frac{\delta p_o'}{p_o'} \tag{7}$$

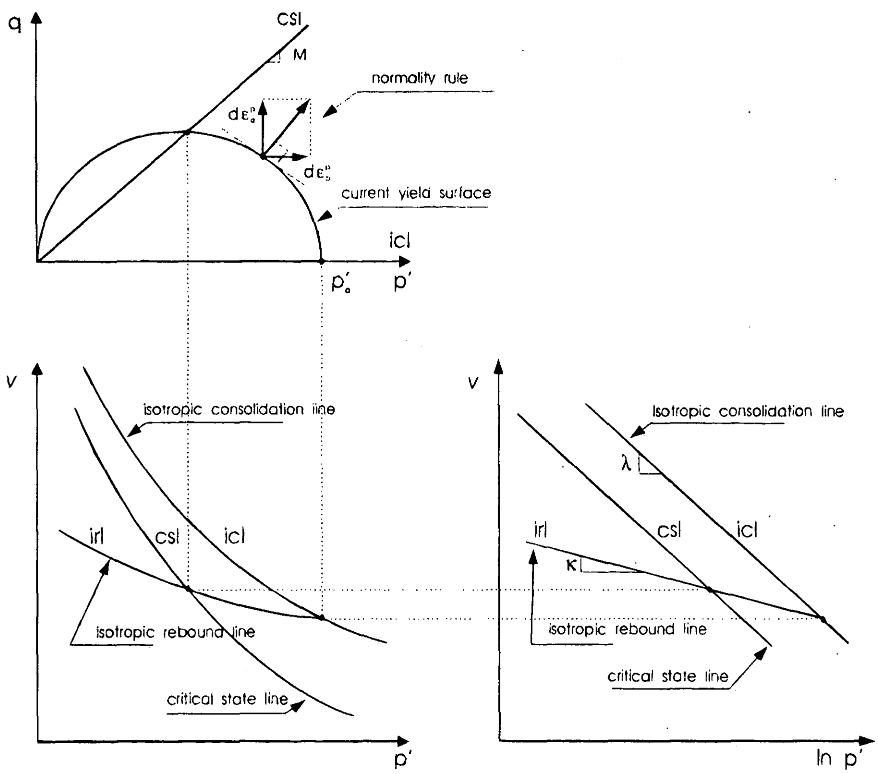

FIGURE 1 **MCC yield and ultimate surfaces $p' - q$ and $p' - v$ spaces.**

CALIBRATION OF MCC MODEL PARAMETERS AND PREDICTION

The calibration of the appropriate model parameters requires at least three undrained conventional triaxial compression (CTC) tests and one isotropic consolidation (IC) test. In addition, if one is interested in simulating the response of the soils in extension, one must also perform three undrained conventional triaxial compression (CTE) tests. As a way to illustrate the procedure, a testing program was designed to test a Speswhite clay under the above-mentioned conditions. The CTC and CTE tests were performed at confining pressures ranging from 50 to 800 kPa. Figure 2 presents the results of an isotropic consolidation test and superimposed are the values of λ, κ, and N. The results of the triaxial test program are presented in Figure 3, along with the predictions obtained from the MCC model. Figure 4 presents the test results in a $q - p'$ space for the 10 undrained shear tests. Note that the values of M_c and M_e can be readily obtained from the test data. Figure 4 also shows the response of the MCC model for each of the 10 test conditions.

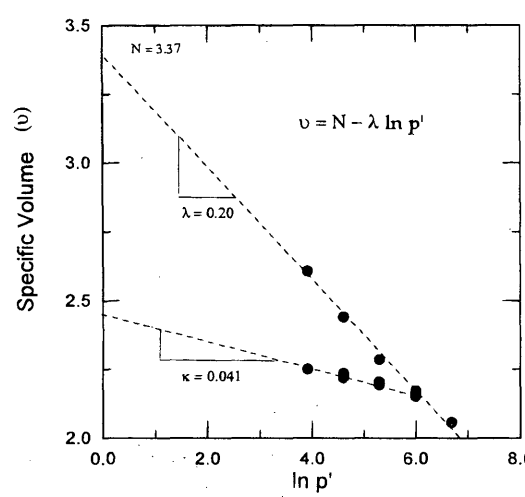

FIGURE 2 **Results of isotropic consolidation test.**

COMPUTATIONAL SEQUENCE FOR MCC MODEL

Drained Triaxial Test on Lightly Overconsolidated Clay

Given: The critical state parameters and the initial conditions: M, e_o, p_o', p', G, K', κ, and λ; step-by-step procedure (Figure 5) is as follows:

1. Compute or note e_o on the Normally Consolidated Line (NC-Line) corresponding to p_o'.
2. Compute or note p_X', e_X, and q_x for point X; the current Critical State Line (CSL)-Yield Surface intersection point.
3. Compute or note e_D for initial point D of test.
4. Compute e_U, q_U, and p_U' for the ultimate point U of the test.
5. Compute q_F and p_F' for the yield point F on the current yield locus.
6. Compute elastic strain components for path DF:

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G} \qquad \Delta\epsilon_p^e = \frac{\kappa}{1+e_o}\frac{\Delta p'}{p'} = \kappa'\frac{\Delta p'}{p'}$$

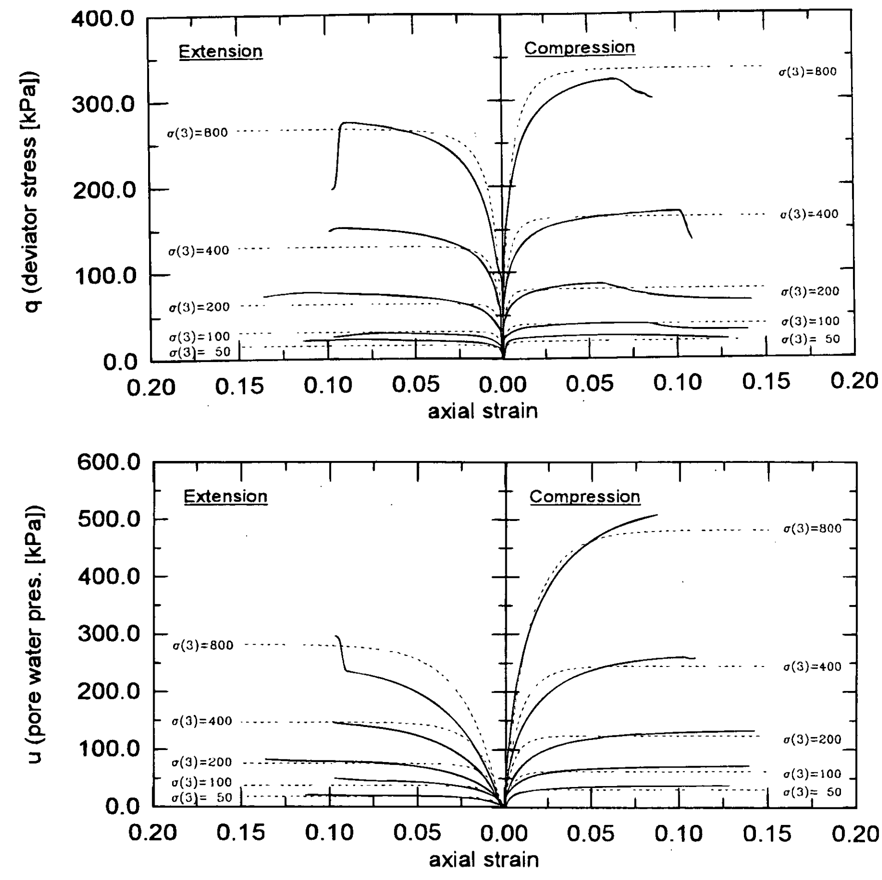

FIGURE 3 Stress-strain (*top*) and pore-water pressure (*bottom*) results of CTC and CTE tests and MCC model predictions.

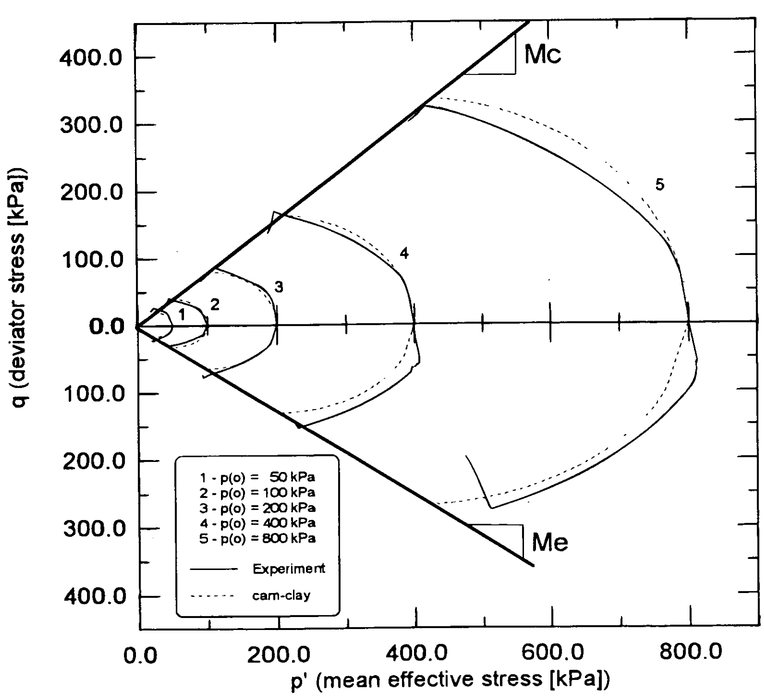

FIGURE 4 Results of CTC and CTE tests and MCC model predictions in $q - p'$ space.

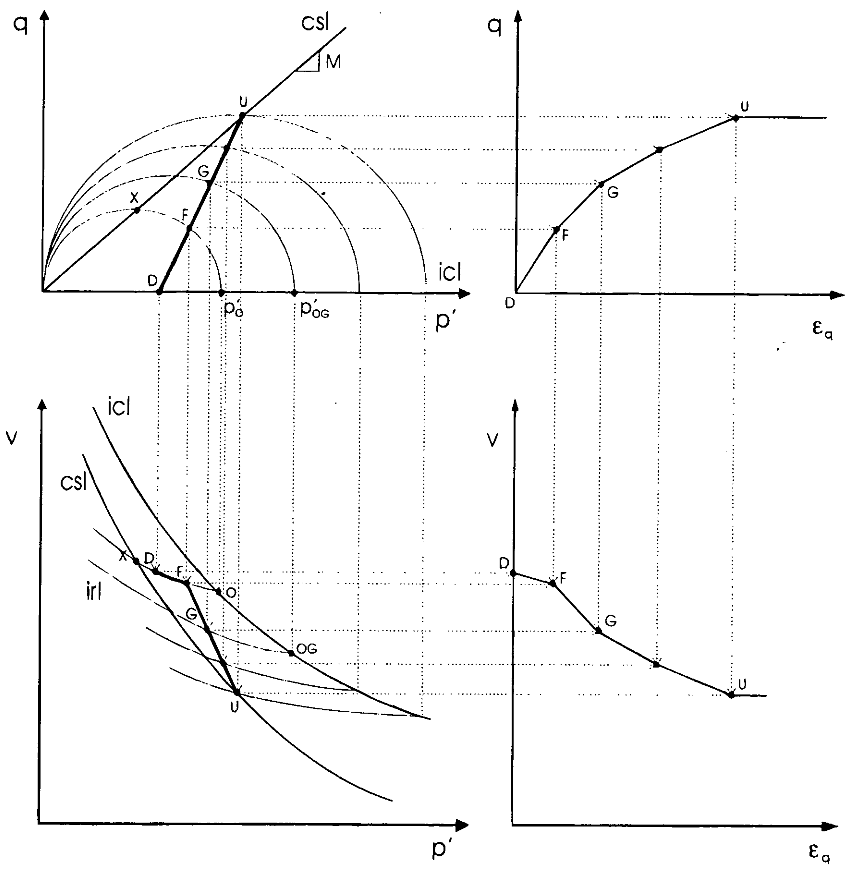

FIGURE 5 Stress paths for MCC model prediction of drained triaxial test on lightly overconsolidated clay.

7. Select several equal stress increments (three or more) along the stress path between F and U.

8. Compute coordinates p_G' and q_G of intermediate point G.

9. Compute coordinates p_{OG}' corresponding to new yield locus through G:

$$p_{OG}' = p_G'\left[\left(\frac{qG}{Mp_G'}\right)^2 + 1\right]$$

which comes from $M^2 p_{OG}' p_G' = p_G^{2'} M^2 + q_G^2$.

10. Find e_{OG} corresponding to p_{OG}' on the NC-Line.

11. Find e_G on the recompression line through OG.

12. Compute

$$\Delta\epsilon_p^{\text{total}} = \frac{\Delta e}{1 + e_o}$$

13. Compute

$$\Delta\epsilon_p^e = \kappa' \frac{\Delta p'}{p'}$$

14. Compute

$$\Delta\epsilon_p^p = \Delta\epsilon_p - \Delta\epsilon_p^e$$

15. Compute plastic shear strain increment $\Delta\epsilon_q^p$ from the Normality Law:

$$\Delta\epsilon_q^p = \Delta\epsilon_p^p\left[\frac{qG}{M^2\left(p_G' - \dfrac{p_{OG}'}{2}\right)}\right]$$

16. Compute elastic shear strain increment as in Step 6:

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G}$$

17. Then $\Delta\epsilon_q^{\text{total}} = \Delta\epsilon_q^e + \Delta\epsilon_q^p$.

18. Repeat Steps 8–15 for point H and any subsequent or intermediate points before U is reached.

19. Plot q versus $\Delta\epsilon_q^{\text{total}}$ and e versus $\epsilon_q^{\text{total}}$.

Undrained Triaxial Test on Lightly Overconsolidated Clay

Given: The critical state parameters and the initial conditions: M, e_o, p_o', p', G, K', κ, and λ; step-by-step procedure (Figure 6) is as follows:

1. Compute or note initial yield surface from p_o'.

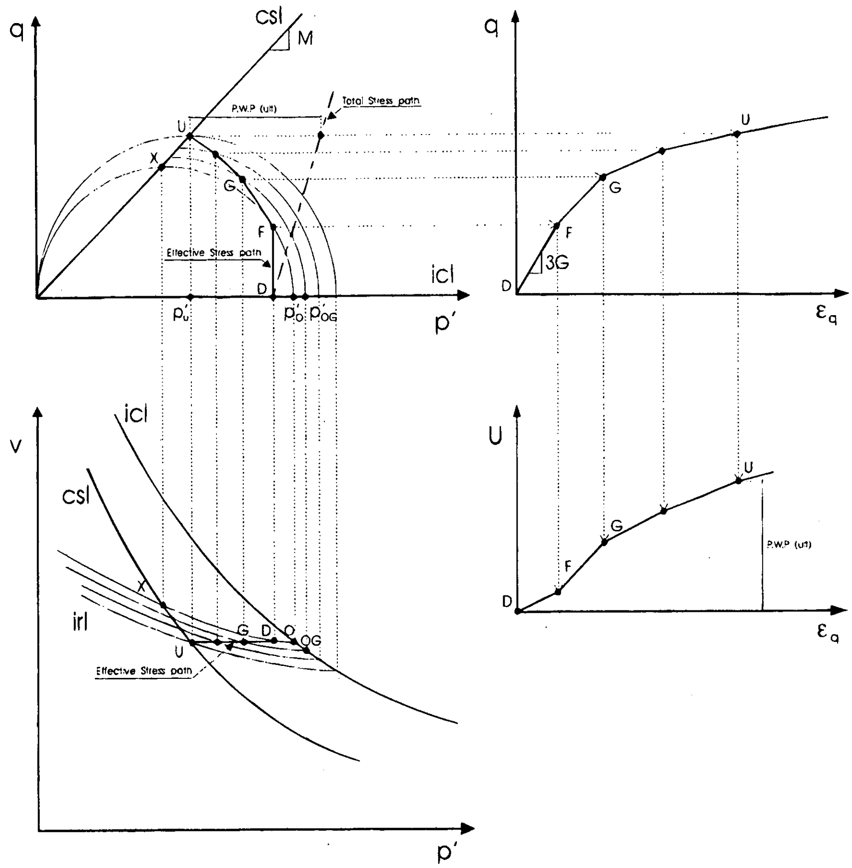

FIGURE 6 Stress paths for MCC model prediction of undrained triaxial test on lightly overconsolidated clay.

2. Compute e_o for initial point D of the test.
3. Compute or note p'_U corresponding to $e_D = e_D$ on CSL; also note q_U.
4. Compute q_F, and p'_F for the yield point F on the current yield locus. Since the behavior is purely elastic and $\Delta V = \Delta\epsilon_p = 0$, the $\Delta p' = 0$ and DF is vertical.
5. Compute

$$\Delta\epsilon_q^e = \frac{\Delta q}{3G} \qquad \Delta u = \Delta p \; (\Delta p' = 0)$$

6. Construct the new yield surface through a selected intermediate point G.
7. From

$$p'_{OG} = p'_G\left[\left(\frac{qG}{Mp'_G}\right)^2 + 1\right]$$

find the new OG at the intersection of the recompression line through G and the NC-Line. Construct new yield locus. Compute qG corresponding to G.
8. Compute

$$\epsilon_p^p = -\epsilon_p^e = -\kappa' \frac{\Delta_p'}{p'}, \text{ since } \Delta\epsilon_p = 0$$

9. Use the Normality Law to compute $\Delta\epsilon_q^p$:

$$\Delta\epsilon_q^p = \Delta\epsilon_p^p\left[\frac{qG}{M^2\left(p'_G - \dfrac{p'_{OG}}{2}\right)}\right]$$

10. Use: $\Delta\epsilon_q^{\text{total}} = \Delta\epsilon_q^e + \Delta\epsilon_q^p$.
11. Compute the increment of the pore pressure as $\Delta u = \Delta p - \Delta p'$.
12. Repeat Steps 7–11 for additional increments.
13. Plot q versus $\epsilon_q^{\text{total}}$ and u versus $\epsilon_q^{\text{total}}$.

IMPLEMENTATION OF THIRD STRESS INVARIANT

To properly account for the three-dimensionality of the stress space, one should include a third stress invariant in the formulation in addition to the two (p' and q) mentioned previously. This third invariant will account for the nonsymmetric shape of the observed yield function in the principal (or octahedral) stress space. One convenient form of the third stress invariant proposed is the so-called lode angle (θ), which gives the angle between the principal stress direction and the current stress path. If one maintains that $\sigma_1 \geq \sigma_2 \geq \sigma_3$, it will suffice to describe the lode angle θ from 0 to 60 degrees, which can be expressed as

$$\cos(3\theta) = \frac{9}{2}\frac{\mathrm{tr}(\sigma_d^3)}{q^3}$$

The MCC model yield function (Equation 4) takes the form

$$f = q^2g^2(\theta) + M^2p'^2 - M^2p'p_o' = 0 \tag{8a}$$

where $g(\theta)$ is a function that defines the shape of the yield function in the deviatoric plane and was originally proposed by Willam and Warnke (*13*):

$$g(\theta) = \frac{4(1-e^2)\cos^2\left(\frac{\pi}{3}-\theta\right) + (2e-1)^2}{2(1-e^2)\cos\left(\frac{\pi}{3}-\theta\right) + (2e-1)\left[4(1-e^2)\cos^2\left(\frac{\pi}{3}-\theta\right) + 5e^2 - 4e\right]^{1/2}} \tag{8b}$$

The eccentricity parameter e must satisfy the condition $\frac{1}{2} \leq e \leq 1$ in order to maintain a convex yield surface. One may define e as the ratio of the shear strength in extension to that in compression. For $e = 1$, i.e., $g(\theta) = 1$, the influence of the third stress invariant via θ is dropped and the now-conical surface becomes a circle in the deviatoric plane. As the value of e approaches $\frac{1}{2}$, the shape of the yield surface becomes more triangular (*14*).

Within the context of the finite element method, it is often convenient to utilize implicit integration techniques (as opposed to the above-mentioned explicit integration) for the solution of unknown stress paths that might develop in a boundary value problem. Implicit integration techniques do not restrict the size of the integration step resulting in a more robust algorithm that can account for larger deformations as compared with the explicit integration techniques (*14*). One implicit integration technique that has gained wide acceptance is that referred to as the Closest-Point-Projection Method (CPPM). For a detailed presentation of this technique, the reader is referred to work by Alawaji et al. (*14*).

The CPPM algorithm was implemented in a mixed-control (stress- and strain-controlled) driver computer code in conjunction with a generalized three-stress invariant MCC model (described previously). Several examples of triaxial stress paths were selected to illustrate the simulation of CTC and CTE tests on normally and overconsolidated clay specimens, as shown in Figures 7–9. The simulations performed represent undrained test conditions. The undrained condition is obtained by subjecting the analysis to an incompressibility constraint in addition to the imposed stress equilibrium conditions.

Figures 9 and 10 present the results of a simulation of specimens tested under CTC conditions under identical confining stress level (20 kPa). Figure 10 shows the results in a $q - p'$ space and presents how the MCC model simulates the overconsolidation of soil specimens. That is, for the normally consolidated state the specimen undergoes plastic strain from the onset of shearing; however, the overconsolidated specimens undergo initial elastic deformation until they reach a stress state compatible with their original yield surface (governed by their preconsolidation stress). It is interesting to note that the specimen having an overconsolidation ratio of 2 undergoes only elastic deformation until it reaches the critical state and then undergoes continuous plastic flow. Figure 10 presents the same results as deviator stress-axial strain and pore-water pressure-axial strain.

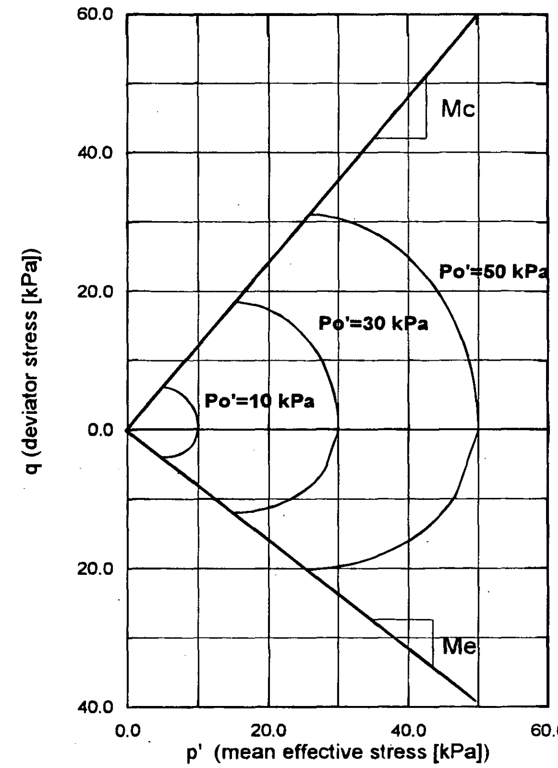

FIGURE 7 Simulation of CTC and CTE tests on normally consolidated clay specimens.

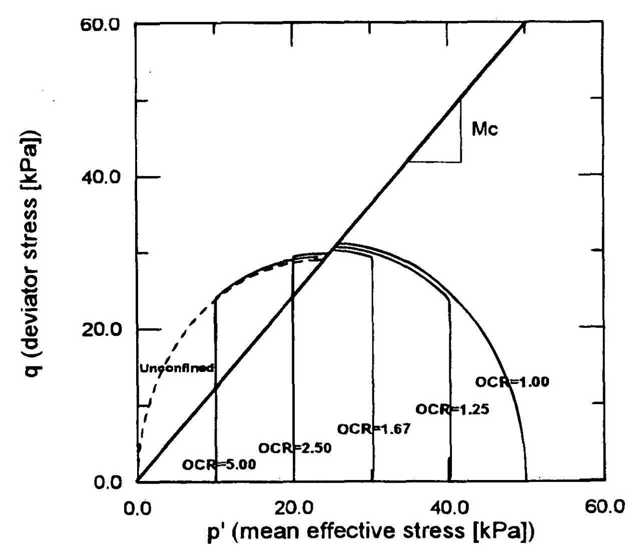

FIGURE 8 Simulation of CTC tests on overconsolidated clay specimens.

FINITE ELEMENT IMPLEMENTATION

In many geotechnical problems it is important to determine the distribution of stresses and displacements throughout the soil structure. In the determination of a system of stresses and displacements for a given problem, one must first define the corresponding governing equations that should satisfy the conditions of equilibrium and com-

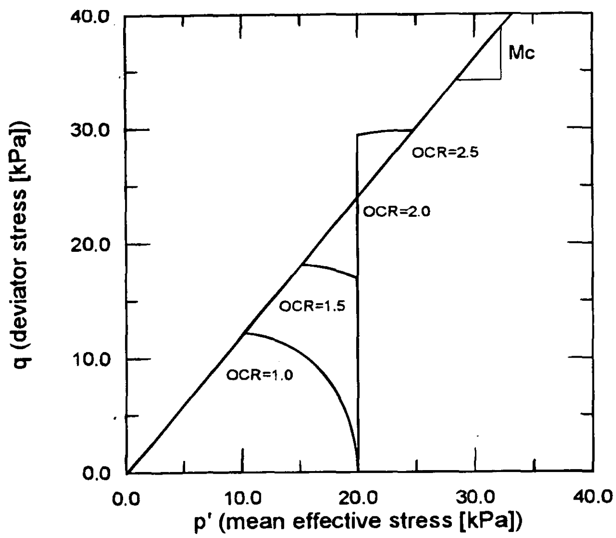

FIGURE 9 Simulation of CTC tests on normally and overconsolidated clay specimens subjected to confining levels of 20 kPa.

patibility. A basic difficulty in this regard, quite different from the solvability of the governing equations, is their ability to represent in situ conditions. Complications in geometry, loading, and material properties contribute to the problem.

The exact solution of the resulting equations, which in general are partial differential equations that satisfy all boundary conditions, is only possible for relatively simple systems, and numerical procedures are commonly employed to predict the system response.

The most common numerical procedure used to address these problems is the finite element method. This consists of the subdivision of the continuum into regions (finite elements) for which the behavior is described by a separate set of assumed functions representing stresses or displacements (*15,16*). These sets of functions are often chosen in a form that ensures continuity of the described behavior throughout the complete continuum. Certain types of finite element algorithms require no knowledge of the material model being used, or the constitutive strategy at the constitutive level. The constitutive formulation must only update stresses and state variables to the finite element level, given the current stresses, state variables, and strain increment. This makes the incorporation of additional constitutive models into the finite element formulation easy.

CONCLUSIONS

This paper has focused on the MCC model, because it is one that has been widely accepted in practice due to its simplicity and "real" physical representation. A generalized form of the MCC model was described where a third stress invariant was included in the formulation. A brief parametric study was presented to show how the MCC model may simulate the response of normally, lightly and highly overconsolidated clays. However, as previously mentioned, there are instances when more sophisticated models may be warranted, especially for highly overconsolidated soil models. From the examples presented in this paper it is evident that the MCC model cannot properly account for the essential characteristics of highly overconsolidated clays. Some of the limitations of the MCC model are as follows.

- During undrained loading shearing of overconsolidated clays, the MCC model predicts linear elastic response for stress states within the current yield surface. In the case of highly overconsolidated clays this linear elastic response is up to peak. The yield point is marked by a sharp change in the tangential stiffness, and the critical state is tangentially approached.
- The MCC model describes uncoupled behavior which result in no shear-induced pore pressure response predictions during

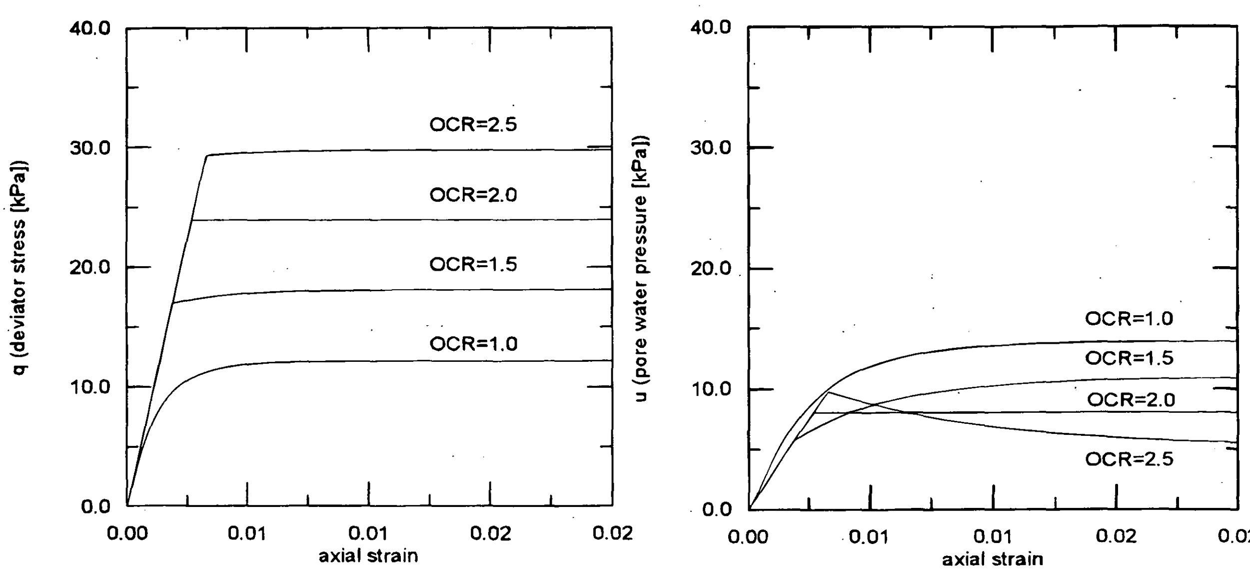

FIGURE 10 Stress-strain-pore-water pressure response of normally and overconsolidated clay specimens subjected to confining levels of 20 kPa.

undrained loading. Therefore it is very difficult for the MCC model to predict the negative pore pressures that highly overconsolidated clays may exhibit during undrained shear.

In recent years there have been some innovative models developed that are capable of simulating the dilatant tendency of overconsolidated clays. However, the complexities in these newer formulations have also made them less desirable to the practicing engineer and will continue to do so until constitutive modelers return to the ideology that simplicity is better. A good compromise between sophistication and simplicity will result in the closure of the gap that today exists between the state-of-the-art and the state-of-the-practice in constitutive modeling of soils. In recent years, researchers have attempted to develop a unified approach to constitutive modeling (i.e., the development of models that may be suitable for different soil types under a variety of stress states and history). This shift in philosophy will surely improve the chances that engineers may again view developments in constitutive modeling as something they should consider for the analysis of practical problems and not as yet another constitutive model.

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APPENDIX

Incremental Relations for MCC Model

During the process of yielding, the material hardens and the yield surface expands to a new position. In order to describe the elastoplastic response of the soil, it is essential that one develop explicit relations in an incremental (flow) fashion. These formulations will then be used to predict the response of a soil as the stresses or strains are incrementally increased or decreased. Hence, the objective is to develop explicit incremental stress-strain relations of the form

$$\tilde{\boldsymbol{\sigma}} = \boldsymbol{C}^{ep}\,\tilde{\boldsymbol{\epsilon}} \tag{A1a}$$

or

$$\begin{Bmatrix} dq \\ dp' \end{Bmatrix} = \begin{bmatrix} C \end{bmatrix}^{ep} \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix} = \left(\begin{bmatrix} C \end{bmatrix}^{e} - \begin{bmatrix} C \end{bmatrix}^{p} \right) \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix}$$

$$= \begin{bmatrix} C_{11} & C_{12} \\ C_{21} & C_{22} \end{bmatrix} \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix} \tag{A1b}$$

From the additive strain decomposition:

$$\epsilon_q = \epsilon_q^e + \epsilon_q^p \qquad d\epsilon_q = d\epsilon_q^e + d\epsilon_q^p \tag{A2}$$

$$\epsilon_p = \epsilon_p^e + \epsilon_p^p \qquad d\epsilon_p = d\epsilon_p^e + d\epsilon_p^p \tag{A3}$$

Hooke's law for isotropic and linear elastic materials gives

$$dq = 3G d\epsilon_q^e \qquad dp' = K' d\epsilon_p^e \tag{A4}$$

$$\tilde{\boldsymbol{\sigma}} = \boldsymbol{C}^e\,\tilde{\boldsymbol{\epsilon}}^e \tag{A5}$$

or

$$\begin{Bmatrix} dq \\ dp' \end{Bmatrix} = \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} d\epsilon_q^e \\ d\epsilon_p^e \end{Bmatrix} \tag{A6}$$

Flow Rule: Associated Flow (the plastic potential is the same as the yield surface $f = g$)

$$d\epsilon_q^p = d\Lambda \frac{\partial f}{\partial q} \tag{A7a}$$

$$d\epsilon_p^p = d\Lambda \frac{\partial f}{\partial p'} \tag{A7b}$$

Yield Function and Consistency Condition:

$$f = f(\sigma_{ij}, \kappa) \leq 0 \tag{A8a}$$

Here

$$df = \frac{\partial f}{\partial q} dq + \frac{\partial f}{\partial p'} dp' + \frac{\partial f}{\partial p_o'} dp_o' = 0 \tag{A8b}$$

$$f = f(q, p', p_o') \leq 0 \tag{A8c}$$

Hardening Rule:

$$d\epsilon_p^p = d\epsilon_p - d\epsilon_p^e = \frac{\lambda dp_o'}{p_o'(1 + e_o)} - \frac{\kappa dp_o'}{p_o'(1 + e_o)} \tag{A9a}$$

or

$$d\epsilon_p^p = \frac{(\lambda - \kappa)dp_o'}{p_o'(1 + e_o)} \tag{A9b}$$

$$dp_o' = \frac{p_o'(1 + e_o)}{(\lambda - \kappa)} d\epsilon_p^p \tag{A9c}$$

From Equations A3, A4, and A6 we obtain

$$\begin{Bmatrix} dq \\ dp' \end{Bmatrix} = \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \left(\begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix} - \begin{Bmatrix} d\epsilon_q^p \\ d\epsilon_p^p \end{Bmatrix} \right) \tag{A10a}$$

or

$$dq = 3G(d\epsilon_q - d\epsilon_q^p) \tag{A10b}$$

and

$$dp' = K'(d\epsilon_q - d\epsilon_p^p) \tag{A10c}$$

From Equations A7a and A7b

$$dq = 3G\left(d\epsilon_q - d\Lambda\frac{\partial f}{\partial q}\right) \qquad dp' = K'\left(d\epsilon_p - d\Lambda\frac{\partial f}{\partial p'}\right) \tag{A11}$$

This expression can then be substituted into Equations A6 and A11 to obtain

$$\tilde{\sigma} = [\mathbf{C}^e - \mathbf{C}^p]\,\tilde{\epsilon} \tag{A14a}$$

$$\begin{Bmatrix} dp \\ dp' \end{Bmatrix} = \left(\left[C^e \right] - \left[C^p \right] \right) \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix} \tag{A14b}$$

where

$$\left[C^e \right] = \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \tag{A14c}$$

and

$$\left[C^p \right] = \frac{\begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} \frac{\partial f}{\partial q} \\ \frac{\partial f}{\partial p'} \end{Bmatrix} \begin{bmatrix} \frac{\partial f}{\partial q'} & \frac{\partial f}{\partial p'} \end{bmatrix} \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix}}{\begin{bmatrix} \frac{\partial f}{\partial q'} & \frac{\partial f}{\partial p'} \end{bmatrix} \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} \frac{\partial f}{\partial q} \\ \frac{\partial f}{\partial p'} \end{Bmatrix} - \frac{\partial f}{\partial p_o'} \frac{p_o(1 + e_o)}{(\lambda - \kappa)} \frac{\partial f}{\partial p'}} \tag{A14d}$$

Knowing the yield function f from Equation 4 one can differentiate to obtain the appropriate ratios:

$$\frac{\partial f}{\partial q} = 2q, \qquad \frac{\partial f}{\partial p'} = 2M^2p' - M^2p_o', \qquad \frac{\partial f}{\partial p_o'} = -M^2p' \tag{A15}$$

Substituting into Equation A14d results in

$$\left[C^p \right] = \frac{\begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} 2q \\ 2M^2p' - M^2p_o' \end{Bmatrix} [2q,\ 2M^2p' - M^2p_o'] \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix}}{[2q,\ 2M^2p' - M^2p_o'] \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} 2q \\ 2M^2p' - M^2p_o' \end{Bmatrix} - \frac{-M^2p'p_o'(1 + e_o)}{(\lambda - \kappa)} 2M^2p' - M^2p_o'} \tag{A16}$$

Substituting Equation A11 and Equation A9c into Equation 14b one obtains

$$\frac{\partial f}{\partial q}3G\left(d\epsilon_q - d\Lambda\frac{\partial f}{\partial q}\right) + \frac{\partial f}{\partial p'}K'\left(d\epsilon_p - d\Lambda\frac{\partial f}{\partial p'}\right) + \frac{\partial f}{\partial p_o'} \frac{p_o'(1 + e_o)}{(\lambda - \kappa)} d\Lambda\frac{\partial f}{\partial p'} = 0 \tag{A12}$$

Rearranging Equation A12 and solving for $d\Lambda$ results in

$$d\Lambda = \frac{\frac{\partial f}{\partial q}3Gd\epsilon_q + \frac{\partial f}{\partial p'}K'd\epsilon_p}{\frac{\partial f}{\partial q}3G\frac{\partial f}{\partial q} + \frac{\partial f}{\partial p'}K'\frac{\partial f}{\partial p'} - \frac{\partial f}{\partial p_o'}\frac{p_o'(1 + e_o)}{(\lambda - \kappa)}\frac{\partial f}{\partial p'}} \tag{A13a}$$

or

$$d\Lambda = \frac{\begin{bmatrix} \frac{\partial f}{\partial q'} & \frac{\partial f}{\partial p'} \end{bmatrix} \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix}}{\begin{bmatrix} \frac{\partial f}{\partial q'} & \frac{\partial f}{\partial p'} \end{bmatrix} \begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} \begin{Bmatrix} \frac{\partial f}{\partial q} \\ \frac{\partial f}{\partial p'} \end{Bmatrix} - \frac{\partial f}{\partial p_o'}\frac{p_o'(1 + e_o)}{(\lambda - \kappa)}\frac{\partial f}{\partial p'}} \tag{A13b}$$

Finally, if one executes the matrix multiplication and expand its terms, one obtains

$$\begin{Bmatrix} dq \\ dp' \end{Bmatrix} = \left(\begin{bmatrix} 3G & 0 \\ 0 & K' \end{bmatrix} - \left[C^p \right] \right) \begin{Bmatrix} d\epsilon_q \\ d\epsilon_p \end{Bmatrix} \tag{A17a}$$

where

$$\left[C^p \right] = \frac{1}{h} \begin{bmatrix} 36G^2q^2 & 6GK'M^2q(2p' - p_o') \\ 6GK'M^2q(2p' - p_o') & K'^2M^4(2p' - p_o')^2 \end{bmatrix} \tag{A17b}$$

and where

$$h = 12Gq^2 + K'M^4(2p' - p_o')^2 + \frac{M^4p'p_o'(1 + e_o)(2p' - p_o')}{(\lambda - \kappa)} \tag{A17c}$$

Equations A17a, A17b, and A17c represent the explicit integration formulation of the MCC model.

Geotechnical Behavior of Overconsolidated Surficial Clay Crusts

ALAN J. LUTENEGGER

The overconsolidated crust of fine-grained sedimentary deposits may exert a significant influence on the performance of structures located in the near surface. The degree of influence depends on the thickness of the crust and its degree of development in comparison with the underlying deposit. The development of a crust is a result of the combined effect of chemical and physical processes acting in place. In this paper, the principal factors responsible for the development of surficial crusts and the resulting geotechnical characteristics of surficial clay crusts are described. Changes in intrinsic soil properties and the variable nature of surficial crusts produced are discussed. Examples from several sites are presented that illustrate the important consequences of crust development on the resulting geotechnical properties, and a description of the implications of the presence of a crust on design practice is given.

Most surficial fine-grained sedimentary deposits exhibit an upper stiff overconsolidated zone that represents a crust developed largely as a result of in situ modification after deposition of the original material. Correct recognition of the extent of the crust and accurate characterization of its engineering properties are of considerable practical significance to geotechnical engineers. Common design situations that may be influenced to some degree by a surficial crust include bearing capacity and settlement of shallow foundations, embankment stability, retaining wall behavior, and stability of slopes. Other problems involving foundation elements in tension, such as pullout or uplift behavior of vertical and inclined pile anchors, screw anchors, or plate anchors, actually may derive a majority of support from the crust.

In the northeast and mid-Atlantic states of the eastern United States and in southern Canada, weathered surficial crusts have been described in a number of areas, generally associated with marine clay deposits, glacial lake deposits, and flood plain deposits. This area includes the Hackensack Meadows of New Jersey, glacial Lake Warren in western New York, glacial Lake Hitchcock of western Massachusetts and Connecticut, and the Atlantic Coast areas of Portsmouth, New Hampshire; Portland, Maine; and the Boston Basin. Other significant fine-grained deposits exhibiting an upper surficial crust include glacial Lake Hudson around Albany, New York, glacial Lake Champlain, certain areas around metropolitan New York, and the Champlain Sea clays of southern Ontario and northern New York.

This paper presents a review of the principal mechanisms involved in the development of overconsolidated surficial clay crusts and discusses the impact of the various mechanisms on the geotechnical behavior of the resulting deposit. Examples of typical soil properties from several sites that contain a crust are presented.

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DEVELOPMENT OF SURFICIAL CRUSTS

Most sedimentary deposits resulting from erosion and redeposition of individual soil grains or grain assemblages develop a surficial weathered crust as a result of postdepositional changes or in-place weathering of the parent material. The degree of formation of a crust depends on the weathering mechanisms that operate through time following deposition of the parent material and therefore is dependent in part on the geologic age of the original deposit. Kenney (*1*) defined *weathering* as follows:

> Weathering is those processes which cause structural disintegration and decomposition of geological materials under the direct influence of the hydrosphere and atmosphere. Disintegration is physical breakdown of the structure of a material, and decomposition is chemical alteration of the constituent minerals and matrix materials.

Hence, weathering involves both physical and chemical processes.

The most common types of physical and chemical weathering mechanisms that produce weathered crusts in sedimentary clay deposits include

1. Seasonal fluctuations in the groundwater level; alternating wetting/drying cycles and translocation of materials;
2. Seasonal temperature changes; alternating freeze/thaw cycles;
3. Erosion or other removal of overburden stress or unloading;
4. Oxidation;
5. Leaching; and
6. Cementation.

These and other mechanisms act in varying degrees to alter the parent deposit and produce materials that exhibit behavior that often does not follow traditional soil mechanics theories or fit typical models of mechanically overconsolidated soil behavior (i.e., overconsolidation resulting from simple unloading).

Geochemical Weathering

The development of a surficial crust in an unaltered sedimentary deposit is largely the result of in-place weathering. Therefore, it is important to have an understanding of the complexity of weathering processes to have a better appreciation for the resulting complexity of the weathering product, that is, the crust. Weathering of diagenetic sedimentary deposits can be thought of as the chemical and physical decomposition of individual particles or particle assemblages. This decomposition may involve both disintegration and alteration of minerals and other constituents within the deposit.

This weathering occurs on two scales: (a) a macroscale in which the processes occur beneath the developed soil profile or solum and would take place without the solum; and (b) a microscale in which the processes only act on the immediate near-surface material or soil solum, often referred to as the A and B horizons by soil scientists or agronomists. Although the latter may have limited importance in geotechnical engineering, it is the former that is of most significance to geotechnical performance of the various design problems previously described. Weathering that is a continuous process occurring below the soil solum is sometimes referred to as *geochemical weathering* (*2*). The most common reactions associated with geochemical weathering include oxidation, reduction, alternating cycles of these reactions, hydration, solution, and hydrolysis. Other processes, including cation exchange and carbonation, also may operate to weather materials in place (*3*).

Oxidation

Oxidation is an important reaction that occurs in well-aerated environments where the oxygen supply is high and the biological demand for it is low. Normally, oxidation is thought of as occurring in soil zones above a permanent water level where the void space is only partly filled with water, that is, in the vadose zone. The most important reaction is the alteration of ferrous iron to ferric iron:

$$Fe^{2+} \rightarrow Fe^{3+} + e^- \qquad (1)$$

The oxidation of iron as described by this reaction disrupts the electrostatic neutrality of the crystal lattice, allowing collapse of the crystal lattice, and can promote additional weathering in the presence of oxygenated water; it allows for the formation of an oxide, Fe_2O_3, or hydrous oxides such as $Fe_2O_3 \cdot H_2O$ (goethite) and $2Fe_2O_3 \cdot 3H_2O$ (limonite). Manganese compounds within the soil are also affected by oxidation. Oxidation of pyrite is also a common reaction during weathering.

Reduction

Reduction occurs in the portion of a soil profile that is saturated or near saturated, such as below the water table, where oxygen supply is low and biological oxygen demand is high. In this case the iron is reduced to the ferrous state, which is highly mobile and can be lost from the system if there is sufficient groundwater movement. If the ferrous iron is retained in the system, it may move into fissures or channels within the sediment and be oxidized or remain in the soil matrix and react to form sulfides and other compounds. If the deposit remains in a reducing environment and a state of saturation or near saturation throughout its geologic history, and it is not subjected to alterations produced by oxidation, the sediment is often referred to as "unoxidized."

Oxidation-Reduction

In zones of transition between a fully aerated environment and a fully saturated environment, groundwater generally fluctuates as a result of seasonal fluctuations in precipitation. In these transition zones, alternating cycles of oxidation and reduction will occur depending on the biological oxygen demand and the availability of oxygen.

Hydration

Hydration is the surface adsorption or association of water molecules or hydroxyl groups with minerals. Hydration usually occurs at the surface or edge of mineral grains. An example is the formation of gypsum crystals:

$$CaSO_4 \text{ (anhydrite)} + H_2O \rightarrow CaSO_4 + 2H_2O \text{ (gypsum)} \qquad (2)$$

Solution

Solution involves the dissolving of simple salt compounds such as carbonates and chlorides that may be present as mineral grains in some soils. An example is the dissolving of calcium carbonate in calcareous deposits:

$$CaCO_3 + 2H^+ \rightarrow H_2CO_3 + Ca^{2+} \qquad (3)$$

One result of solution reactions can be the leaching of minerals from the system if there is sufficient groundwater movement or the precipitation and redeposition of minerals into segregated zones. The leaching of carbonates and other minerals is a common result of solution activity. The degree of leaching depends on groundwater chemistry and fluctuations, infiltration, time, and initial mineral composition.

Hydrolysis

Hydrolysis normally refers to the attack of hydrogen on the crystal structure of certain minerals. The result is often a replacement of the basic ion composition by the hydrogen. An example of hydrolysis is the attack of hydrogen on the interlayer potassium of micas to produce illite (by partial K removal) or vermiculite (by full K removal). Hydrolysis is an important process that can result in partial or complete modification of weatherable primary minerals and the production of mixed layer minerals by cation replacement.

Cementation

Cementation may play a role in the behavior of surficial crusts, but its influence often may be overshadowed by more dominant processes. Cementation bonds in soils can develop by translocation of various cementing agents that then precipitate between particles or particle assemblages. Among the more common cementing agents are carbonates (calcium and magnesium), iron oxides, silica, and amorphous compounds. It is sometimes difficult to identify cementing agents in soil samples; however, there are well-documented studies of the influence of cementation on soil behavior such as stress history, compressibility, and shear strength (*4,5*). The presence of carbonates as a form of cementation may have a significant influence on geotechnical properties, for example, as illustrated by Burghignoli et al. (*6*).

In general, the combined activity of these and other geochemical weathering processes can be considered in a simplistic model that is often used to illustrate the pedochemical changes that take place to produce the soil solum. Because these processes operate at varying rates depending on ground temperature, topography, hydrology, initial soil mineralogy and composition, and groundwater chemistry, many resulting profiles are possible containing a wide number of end products.

A diagram illustrating the combined action of these processes is shown in Figure 1. All of the reactions discussed can be categorized as additions, transformations, transfers, or removals. Unfortunately, the effect of some developmental mechanisms such as leaching of carbonates or oxidation on specific geotechnical behavior have not been studied in any detail or systematic manner and therefore are unknown or poorly understood.

Physical Weathering

Physical processes that modify massive sedimentary fine-grained deposits can act in varying degrees of intensity and for varying durations to create postdepositional modifications of the deposit, which can be just as dramatic as those caused by chemical processes. The most important physical weathering processes include groundwater fluctuations, desiccation, frost action (or freeze-thaw cycles), and unloading. Other physical processes, including drained creep and organic activity of plants and animals, can also influence crust formation.

Groundwater Fluctuations

Significant changes in groundwater levels can occur in shallow depths leading to substantial changes in effective stresses. The actual magnitude and frequency of fluctuations depend on a number of factors, including site topography, regional hydrogeology, local hydrogeology, surface drainage characteristics, rainfall and other seasonal precipitation, and surface characteristics that may control runoff and infiltration. The development of overconsolidation at the surface of a soft clay by changing groundwater levels has been described by Parry (*7*), who noted that within the crust the effective stresses are caused by not only the weight of the soil but also by negative pore water stresses induced by desiccation.

An example of typical groundwater fluctuations in a clay crust is shown in Figure 2, which presents piezometer observations taken over several years at a number of elevations at the National Geotechnical Experimentation Site (NGES) at the University of Massachusetts-Amherst (UMass). The zone of active modern groundwater fluctuations is within the upper 4 to 5 m. Within this zone it can be seen that groundwater fluctuations are generally seasonal and coincide with seasonal variations in precipitation, but they are also influenced by single rainfall events that can affect daily groundwater levels in the near surface. For example, variations are greatest in the shallowest piezometer at a depth of 1.52 m, which

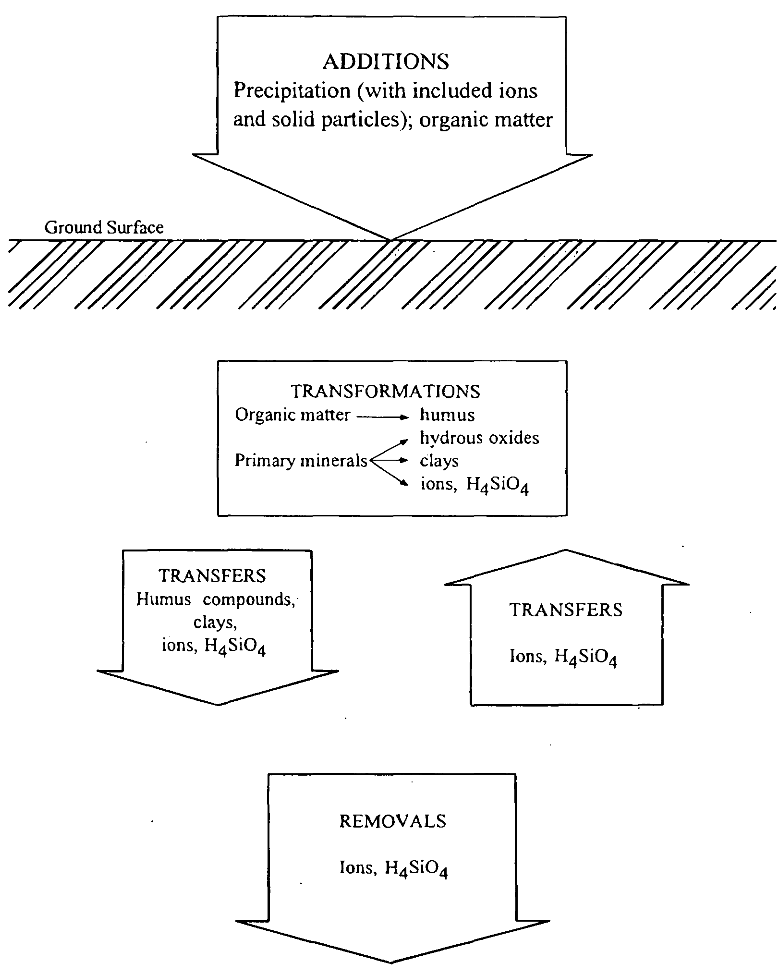

FIGURE 1 Flowchart of major geochemical processes in crust formation.

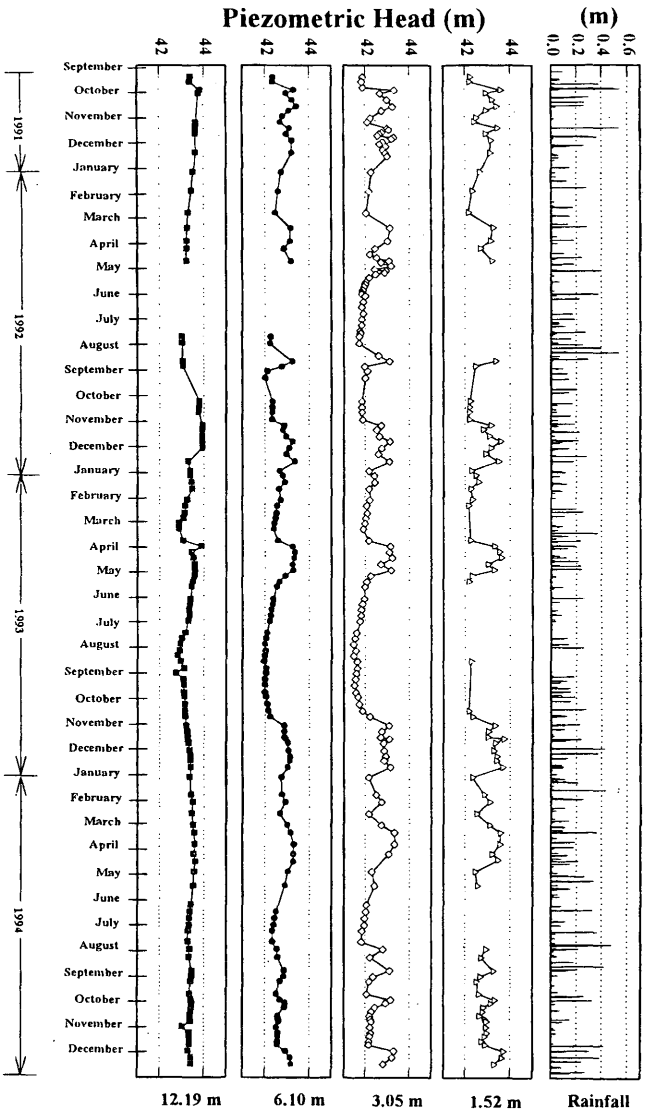

FIGURE 2 Groundwater fluctuations at NGES UMass-Amherst.

actually goes dry in the summer months. Below the active groundwater zone, it can be seen that the amplitude and frequency of groundwater fluctuations are less affected by single events and even show less pronounced influence for seasonal variations in precipitation. The maximum observed fluctuation in static groundwater level over a 3-year period has been about 2.5 m.

What are the consequences of fluctuations in the static groundwater level to the development of a surface crust? Consider a site in which the groundwater table fluctuates from the ground surface to a depth of 3 m. If the total unit weight of the soil is taken as 1.9 Mg/m^3 and a constant preconsolidation stress (σ_p') of 143 kPa is assumed throughout the profile, the stress history fluctuates within the upper 10 m as shown in Figure 3 simply from the change in groundwater position. As can be seen, the impact on the overconsolidation ratio ($OCR = \sigma_p'/\sigma_{vo}'$, where σ_{vo}' equals the in situ vertical effective stress) within the upper 5 m is dramatic with the *OCR* at a depth of 1.5 m varying from about 5 to 10. Below a depth of about 5 m, where the soil is normally consolidated, the magnitude of difference between the stress history at the two different times is probably within the measurement error of the laboratory determination of the preconsolidation stress.

This simplistic example illustrates the importance of having reliable measurements of in situ pore-water pressures throughout a site profile to evaluate soil behavior. For example, if correlations are being developed between laboratory tests and in situ tests, it is imperative to have pore pressure measurements at the time the in situ tests are performed to determine effective stress accurately.

Desiccation

The surface of a sedimentary deposit is susceptible to drying out as a result of contact and exposure to the atmosphere. In the zone immediately beneath the surface, water is lost by evaporation at different times of the year as a result of climatic changes. This reduction in water content can result in strong capillary action, resulting in the development of negative pore-water pressures, which in turn results in an increase in effective stress. This in effect can produce a preconsolidation effect in the soil; the development of high lateral stresses may also produce fracturing or fissuring of the soil. The significance of negative pore-water pressures or matric soil suction in the zone of capillary saturation and in the vadose zone has been presented in detail by Fredlund and Rahardjo (*8*).

Over time, and with multiple cycles of wetting and drying, an extensive fracture pattern can develop. Infilling of the open fissures with washed material can help produce coatings on the face of fracture surfaces and can also help reduce crack closure during wetting. In soils composed of expansive clay minerals, the cyclic wetting and drying may produce slickensided surfaces as a result of development of passive failure planes from expansion. Desiccation by surface drying may also produce an increase in soil unit weight resulting from a reduction in void ratio from the consolidating effect of an increase in effective stress. The water content in this part of the crust may be near or below the plastic limit.

The thickness of the desiccated zone of a crust depends on climatic conditions and the seasonal fluctuations in the groundwater table. Even below the desiccated crust, other weathering processes produce an altered zone that is still considered, along with the drying crust, as part of the crust. The lower extent of the crust in most clay deposits is generally taken as the depth at which the undrained shear strength exhibits a minimum value.

Desiccation may also be produced by vegetation, especially large trees, which can also produce fissuring of clays. Large reductions in water content can occur in the upper few meters of soil as a result of root penetration and water removal by trees. This reduction in water content can produce consolidation of the soil leading to enhanced settlements of shallow foundations. A number of cases of this type have been reported previously (*9,10*). When trees are removed, the groundwater level may recover (*11*), producing a reduction in effective stress. A general rule of thumb regarding the influence of trees appears to be that the root penetration of trees is approximately equal to the height of the tree. The zone of influence of water removal by large trees may extend as much as 20 m beyond the base of the tree.

Frost

Seasonal fluctuations in the maximum depth of frost penetration can produce results in the soil similar to desiccation. Frost action is

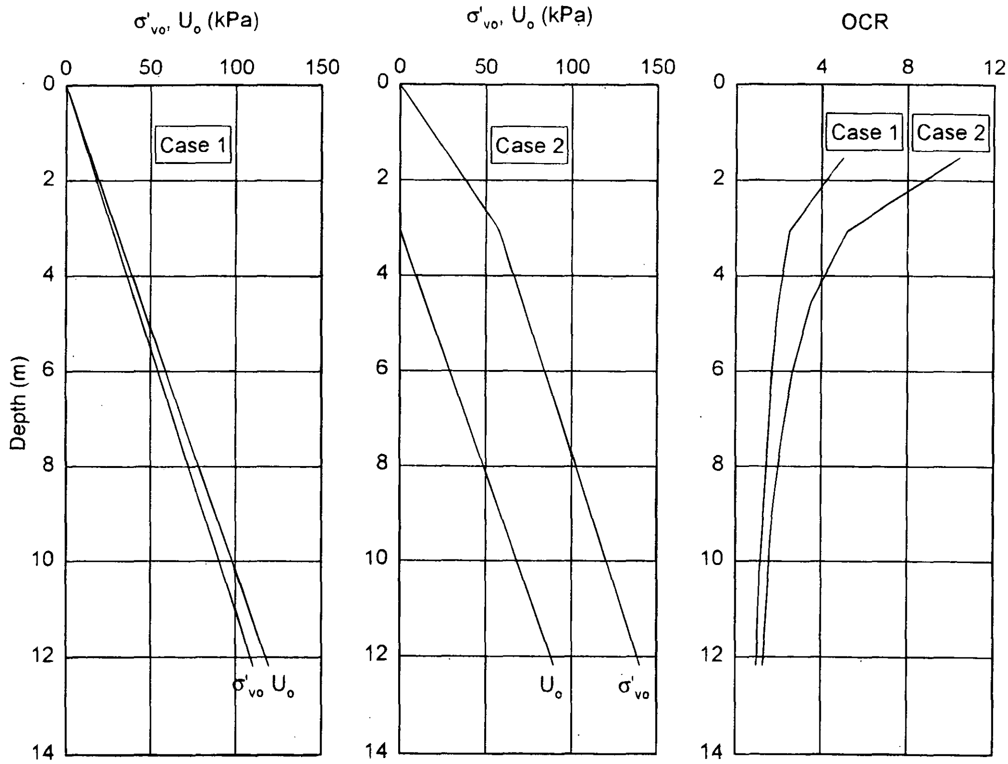

FIGURE 3 Changes in stress history resulting from groundwater fluctuations.

obviously more important in northern latitudes but can still produce modifying surficial effects in other areas from occasional climatic changes. For example, Ladd (*12*) described the formation of an overconsolidated freeze-thaw crust in marine deposits at James Bay in northern Quebec, Canada.

The effect of soil freezing on overconsolidation has been described in detail by Chamberlain (*13*), who found that preconsolidation stresses in plastic soils could be induced by freezing and could greatly exceed in situ prefrozen stresses because of large increases in pore-water tension during freezing. The formation of a preconsolidated frost crust has also been described by Vahaaho (*14*) as a means of stabilizing road beds in Finland. The increase in preconsolidation stress of soft, normally consolidated clays resulting from a decrease in temperature is also well documented. Alternating freeze-thaw cycles can result in the development of jointing or fissuring and may also cause water migration.

Unloading

Reduction in effective confining stress resulting from uplift, erosion, or changes in pore pressure can also produce cracks and joints within an otherwise massive deposit. Joints in surficial crusts are typically produced by elastic rebound coupled with alternating shrinking and swelling or freeze-thaw cycles. Most unloading in surface crusts is generally considered to take place as a result of the removal of overburden accompanied by physical erosion. The degree of removal may vary dramatically from a few meters to several tens of meters depending on the geologic setting. In most young sedimentary deposits, unloading may have a minor effect on the formation of crusts.

EFFECTS OF CRUST-FORMING PROCESSES

What are the overall consequences of these and other processes on the resulting extent and geotechnical behavior of a crust? Bjerrum (*15*) has shown that the thickness of a crust may range from as little as 1 to 3 m to as much as 6 to 8 m depending on landscape position (i.e., well-drained versus poorly drained topographic position) and site hydrogeology. Table 1, taken from Brenner et al. (*16*), summarizes the effect of a number of the mechanisms discussed on the geotechnical properties of marine clays.

Alteration of Deposit

The overall result of the development of a surficial crust as a result of in-place weathering is that the material in the crust often shows little resemblance to the underlying material. It should not be surprising, then, that in many cases the observed behavior, such as undrained shear strength or in situ stress state, cannot be fully explained by stress history. In these cases stress history may be considered to be the result of both chemical and physical phenomena.

The most obvious consequence of crustal development is the modification of the original geologic deposit. The extent of the development in terms of both degree of modification and depth depends on a number of factors. Processes of crustal development, both physical and chemical, require time to operate and are affected

TABLE 1 Effect of Postdepositional Processes on Geotechnical Properties of Marine Clays [from Brenner et al. (*16*)]

Process	Geotechnical property								
	Water content	Liquid limit	Plasticity index	Liquidity index	Preconsolidation pressure	Compressibility	Undrained strength		Sensitivity
							Undisturbed	Remoulded	
Desiccation	−		±	−	+	−	+		
Chemical weathering	±	+	+	−	+	−	+	+ or −[1]	− or +[1]
Leaching	±	−	−	+	−	+	−	−	+
Cementation		+	+		+	−	+	+[2]	+

+ Increase; − Decrease; ± little or no change
[1] Depends on type of clay mineral
[2] Depends on amorphous content

locally at any given site by climate (rainfall and temperature fluctuations), vegetation, topography (degree of slope and relative landscape position), material (original mineralogic composition at time zero), and time (geologic age of the deposits).

The most obvious and significant results of the alteration or modification of the virgin deposit as a consequence of the formation of a surface crust are

1. Changes in soil color,
2. Changes in soil structure,
3. Changes in mineralogic composition,
4. Changes in intrinsic properties, and
5. Increase in soil variability.

It is because of the fundamental changes that take place on a small scale that the geotechnical behavior changes on a large scale. Table 2 presents a classification of soft clay proposed by Bjerrum (*17*) that compares weathered clays in the crust to unweathered clays on the basis of water content, Atterberg limits, shear strength, and compressibility.

Soil Color

The matrix colors of sediments sometimes have been related to the state of oxidation and the chemical status and distribution of iron. The oxides of iron have visual properties that may be determined by the distance between iron atoms. For example, hematite, Fe_2O_3, has an iron-iron distant of 2.88 Å and a red color. The hydrated iron oxides, such as goethite and limonite, tend to be lighter in color. A reduced form of iron, iron sulfide, has an iron-sulfide distance of 2.27 Å and a very light color.

Colors observed for the unoxidized matrix in which the iron occurs in the ferrous state include dark gray, dark greenish-gray, greenish-gray, green, blue, and bluish-gray. Soil color ranges for the oxidized zone of most sediments include reddish-brown, yellow-brown, and olive-brown. The change in soil color from those of the unoxidized to those of the oxidized state can occur rapidly upon exposure to air (*18*).

In the transition zone between unoxidized and oxidized zones, where groundwater fluctuates, soil colors will reflect characteristics of both an oxidizing and a reducing environment. Background base color may appear as brown, whereas distinct "blotches" of gray or blue-gray are present or as gray with distinct blotches of brown. The thickness of this mottled zone depends on the mineral composition of the soil, the degree of groundwater fluctuations, and the chemical composition of the groundwater.

Mineral Composition

In some cases, changes in mineralogic composition will also accompany weathering. An example of such alteration is shown in Figure 4, which shows carbonate profiles obtained at three sensitive marine clay (Leda) sites in northern New York. The first site, IDA, occupies a geomorphic low position in the landscape that does not allow groundwater fluctuations to go much below a depth of 0.6 m, even during extended dry periods. The site is capped by a surficial sand deposit about 1 m thick. It is suspected that this site has undergone very minimal modification since deposition, and it exhibits no significant surficial crust. The carbonate composition shown in Figure 4 shows relatively small modification near the surface relative to the underlying material. In contrast, the other two sites sit on more well-drained geomorphic positions about 1.2 km from the first site. The groundwater table fluctuates as a result of seasonal precipitation, and the lowest level historically may have been on the order of 4 to 5 m below ground surface. These sites display a substantial surficial crust that contains distinct blocky soil structure and common fissures. The degree of alteration of the carbonate mineralogy at both of these sites is pronounced down to a depth of about 3 m. Such obvious changes in composition, brought about by postdepositional changes, may help explain differences in soil behavior that may not be explained only by differences in stress history.

Scale Effects

It has long been recognized that significant scale effects can be present in fine-grained deposits that exhibit secondary structure mainly in the form of microcracks, discontinuities, fissures, joints, and other macroscale features. Weathered crusts developed in sedimentary clay deposits often display a blocky fissured soil structure, with the individual frequency of fissures or joints related to the degree of structural formation. Even laboratory shear strength tests performed on larger-than-normal specimens may be adversely affected by the frequency and orientation of fissures. Lo et al. (*19*), Garga (*20*), and others have shown clearly that the undrained shear strength of stiff fissured clays and other structurally dependent soils is related to the size of the specimen and that field tests that include a larger volume of soil, such as plate loading tests or large scale field shear box tests, generally give

TABLE 2 Classification of Soft Clays [from Bjerrum (*17*)]

	Classification	Water Content	Shear Strength	Compressibility
Weathered clays in upper crust	Frost treated, dried-out clays	$w_n \approx w_p$	Very stiff, fissured, with open cracks	-
	Dried-out clays	$w_n \approx w_p$	Very stiff, fissured	Low compressibility
	Weathered clays	$w_p < w_n < w_L$	Shear strength decreases with depth	Low compressibility curved e-log σ'_v curve
Unweathered clays	Young normally consolidated clays	$w_n \approx w_L$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc} \approx \sigma'_{vo}$
	Aged normally consolidated clays	$w_n \approx w_p$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc}/\sigma'_{vo}$ constant with depth
	Young normally consolidated quick clays	$w_L < w_n$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc} \approx \sigma'_{vo}$
	Aged normally consolidated quick clays	$w_L < w_n$	s_u/σ'_{vo} constant with depth	$\sigma'_{vc}/\sigma'_{vo}$ constant with depth

w_n = water content
w_p = plastic limit
w_L = liquid limit
s_o = undrained shear strength
σ'_{vo} = in situ vertical effective stress
σ'_{vc} = preconsolidation stress

lower shear strengths than laboratory tests because of the greater probability of including macrofeatures within the test specimen.

Results from undisturbed samples and even field vane tests tend to give higher strength values because macrofeatures are not always present. In fact, the results of laboratory tests on undisturbed samples from normal size sampling tubes (e.g., 76 mm) actually may be inadvertently biased toward the high (unsafe) side because only those specimens that remain intact during preparation are tested. The remainder of the sample, which tends to fall apart, often cannot be trimmed and placed into a testing fixture and is discarded.

These scale effects may have serious implications in design relating to choosing a design shear strength value or strength profile for use in analysis. This issue was addressed by Meyerhof (*21*), who suggested that a strength reduction factor be used when the end-bearing capacity of bored and driven piles in stiff fissured clays is evaluated.

Bjerrum (*17*) and Pilot (*22*) noted that the thickness and strength of the crust may have an important role in defining the mode of failure of an embankment. The selection of the undrained shear strength profile in the weathered crust of an otherwise soft clay deposit may also have a strong influence on the stability analysis of embankments or footings as demonstrated by Sagaseta and Arroyo (*23*) and others (*24*). This consideration is important because a substantial portion of the failure surface under shallow footings or embankments may be located within the crust.

A suggestion for reducing the undrained shear strength profile obtained in the crust from the field vane has been presented by Tavenas and Leroueil (*25*). Other schemes for reducing the field vane strength in the crust have also been presented (*26*). One reason that field vane strength tests may show what appear to be unusually or abnormally high undrained shear strength values in the crust is that the test may not represent undrained loading conditions and therefore the response obtained may include a significant component of drained behavior. It has been suggested that the field vane be used to define the thickness of the crust by identifying the location of minimum strength in the profile rather than determining the absolute undrained strength.

Scale effects may also be manifested in surficial crusts and evidenced in the flow characteristics (i.e., hydraulic conductivity) of the deposit. Most surficial clay crusts have a significantly higher hydraulic conductivity than the underlying unweathered deposit. For example, results of in-place hydraulic conductivity tests presented by Lafleur et al. (*27*) indicated that the hydraulic conductivity of the brown oxidized crust of a marine clay may be two to three orders of magnitude higher than that of the underlying gray unoxidized zone. It has also been demonstrated (*28*) that

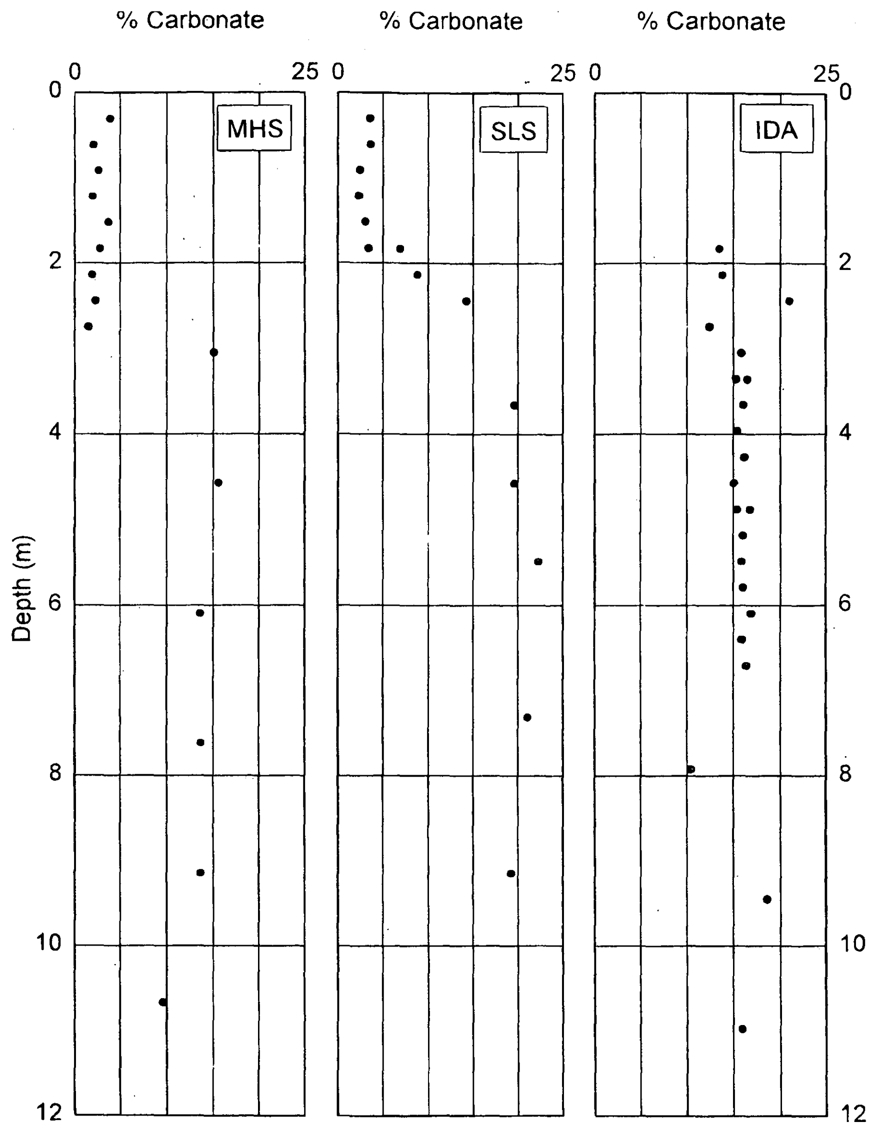

FIGURE 4 Carbonate composition at three marine clay sites.

there may be a significant scale effect when the results of laboratory and field hydraulic conductivity tests are compared with field results giving much higher values. This means that at some sites there is a high likelihood that the soil in the crust does not behave undrained during certain field-loading conditions (e.g., laterally loaded drill shafts).

Scale effects may also exert a significant influence on the results of in situ tests, such as the cone penetration and piezocone tests, which often only involve a small volume of soil. This has been illustrated by Marsland and Quarterman (*29*) and by Mayne et al. (*30*).

Anisotropy

Soil properties of overconsolidated clay crusts may exhibit characteristics that are directionally dependent (i.e., intrinsic anisotropy). For example, Ladd et al. (*31*) indicated that some stiff fissured clays exhibit pronounced undrained strength anisotropy. A number of studies in which samples of stiff, highly overconsolidated and often fissured clays have been trimmed in different directions have shown that shear strength under compressive loading is higher for horizontally trimmed samples than for vertically trimmed samples (*32,33*). Additionally, field investigations using the field vane test suggest that significant undrained strength anisotropy is also present in overconsolidated crusts (*34*). There is also evidence that suggests that some highly overconsolidated clays exhibit directionally dependent stress-strain behavior (*35,36*). Unfortunately, because of the difficulties in sampling, trimming, and testing natural clay crusts, there are very limited data on their anisotropic behavior.

Variability in Properties

One of the important consequences of the development processes of surficial crusts is the production of a highly variable deposit. It is expected that the properties will be more variable than those of the underlying unweathered section, and therefore more effort will be needed to define the engineering properties of the crust. Large variations in such properties as water content, shear strength, compressibility, stress history, and other compositional and structural properties can occur over relatively short distances. A few examples are presented to illustrate these variations.

Water Content and Unit Weight

Simple properties such as water content and soil unit weight often show larger variations in surficial crusts than in the underlying unaltered zone of the profile. The variation in water content obtained throughout the surficial crust and into the underlying unweathered zone at the UMass-Amherst NGES is shown in Figure 5. These data, taken from a combination of hand auger and tube samples, illustrate that large changes occur in both the lateral and vertical directions. In this case the systematic increase in water content with increasing depth helps to identify the base of the severely altered sediments as the water content approaches a relatively constant value. Some water content variations below the crust in Figure 5 represent individual silt and clay varves. Variations in unit weight of the soil at this site, obtained from individual trimmed specimens, are shown in Figure 6. Again the variation in both lateral and vertical directions of even a simple parameter is evident, and the effect of surface processes is a systematic increase in unit weight.

Water content and plasticity data obtained at two sensitive marine clay sites in northern New York are shown in Figure 7. As previously indicated, the IDA site does not exhibit a crust, whereas the MHS site exhibits a pronounced crust. These results illustrate a relatively common feature in the crust, that is, the water content is usually between the liquid and plastic limit and therefore the liquidity index is low in the crust and increases progressively into the lower unaltered zone.

Preconsolidation Stress

Variations in stress history at a given elevation within a clay crust is expected and may be considered the result of the combined effects of the chemical and physical weathering processes. Apart from the obvious problems of sampling disturbance that may

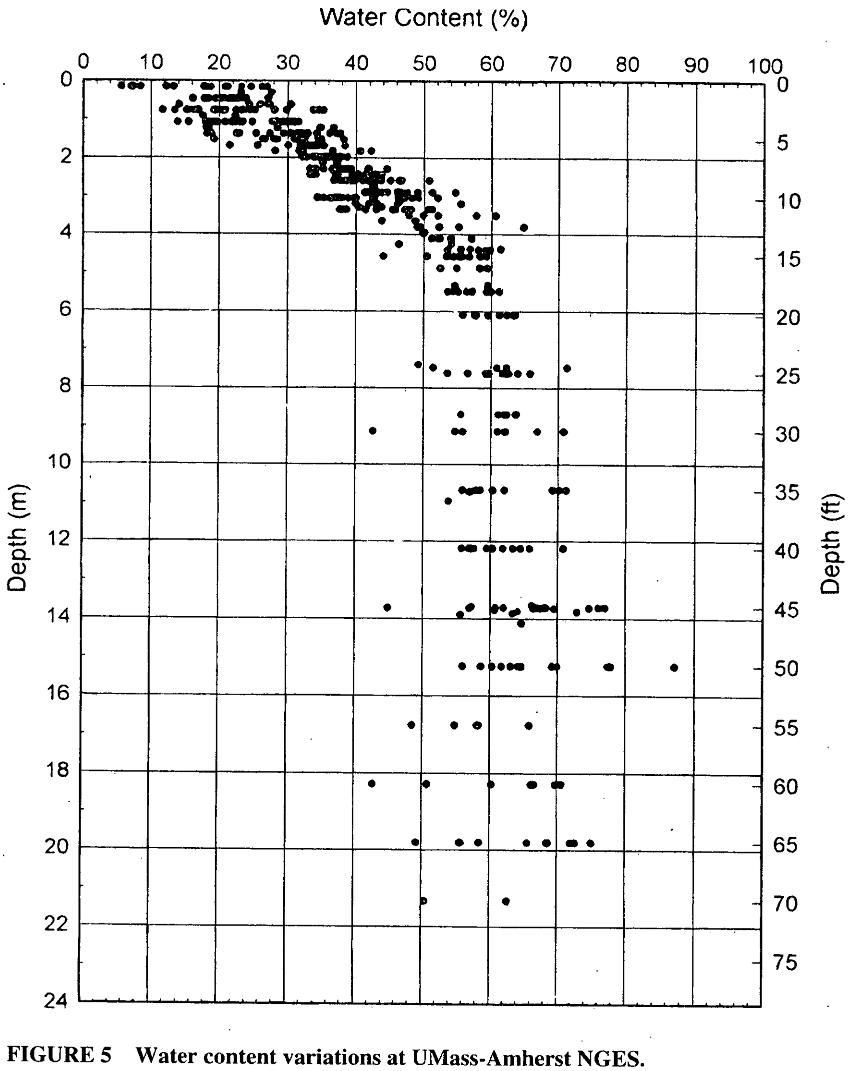

FIGURE 5 Water content variations at UMass-Amherst NGES.

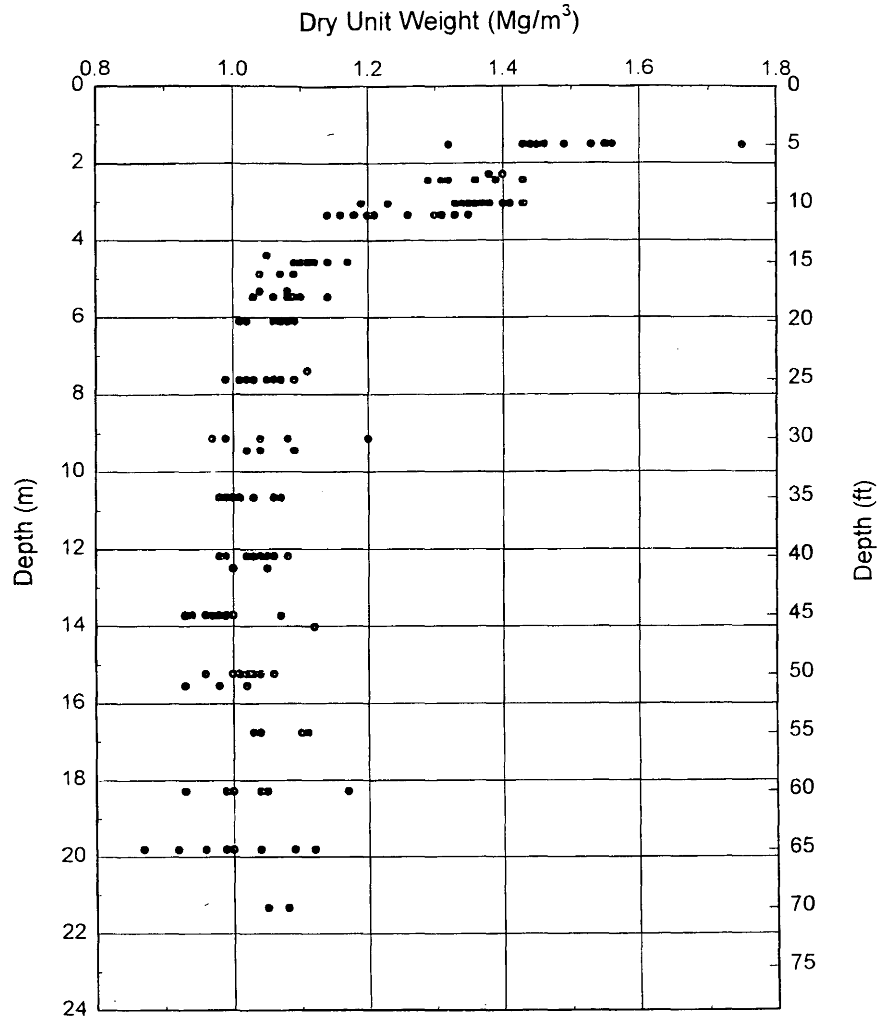

FIGURE 6 Unit weight variations at UMass-Amherst NGES.

accompany the determination of preconsolidation stress (yield stress) in the oedometer, such natural variations may be accentuated because small specimens, for example, on the order of 60 mm, are typically used in performing the test. The results of an initial series of oedometer tests performed at the UMass-Amherst NGES to evaluate stress history are shown in Figure 8. The range in interpreted preconsolidation stresses in the upper part of the profile illustrates the difficulties that can be encountered. What is the proper interpretation of the stress history profile for use in design? This variation in preconsolidation stress and its effect on settlement predictions have been recognized and discussed by Duncan et al. (*37*).

The results of Figure 8 also illustrate the difficulty in using singular values of a given property to correlate the results obtained between laboratory and in situ tests. For example, in this case, selecting which values of σ'_p should be used to develop correlations with the results of cone penetration, piezocone, dilatometer, or other in situ tests can have a significant effect on the resulting correlation. The author suspects that such natural variations are a significant source of errors encountered in the application of empirical correlations between in situ and laboratory tests for many overconsolidated clays.

In Situ Test Results

As mentioned previously, the development of a secondary soil structure from a massive deposit can affect the results of in situ tests, especially small-scale penetration tests. One expects that the variation in test results would decrease through the crust as the secondary structure diminishes with depth into the massive deposit. For example, results of prebored pressuremeter tests performed through the crust at the UMass-Amherst NGES indicate that the range of possible earth pressure coefficients that might be interpreted from the tests is very large in the crust but decreases with depth approaching the less altered zone, as shown in Figure 9. Values of $(K_o)_{min}$ and $(K_o)_{max}$ indicated in Figure 9 are obtained simply from the initial and final points on the straight-line portion of the pressuremeter test. In situ lateral stress ratios may approach limiting passive values; however, passive earth pressures in heavily overconsolidated (i.e., $OCR > 10$), near-surface soils may be much higher than previously reported, simply because a significant effective stress cohesion component has been ignored when limiting stress ratios are calculated and because stress ratios cannot be evaluated by the simple Rankine expression for cohesionless soil.

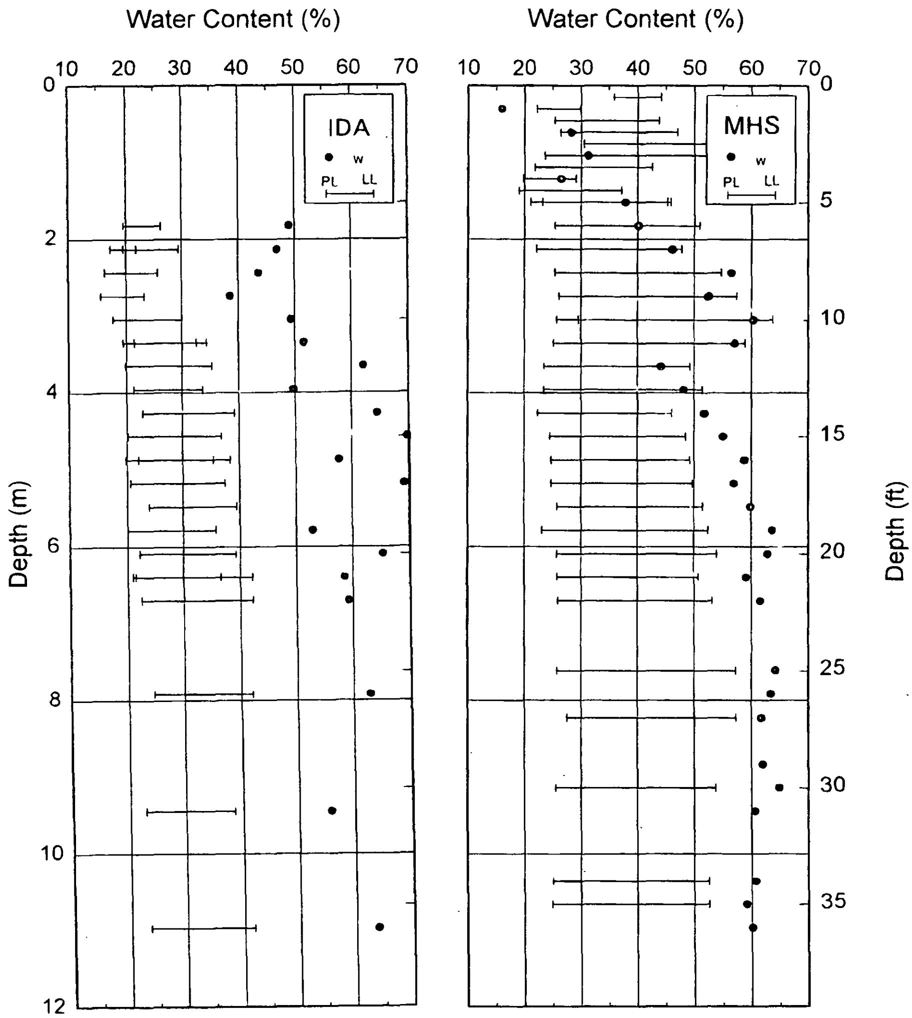

FIGURE 7 Plasticity and water content profiles in two marine clays.

The plate load test and screw plate test, which involve the response from a larger volume of soil have been applied successfully to evaluate the deformation modulus of surficial weathered clay crusts and other stiff clays by Bauer et al. (*38*) and Powell and Quarterman (*39*). Undrained shear strength values obtained from plate load tests also appear to be more applicable in clay crusts to evaluate the behavior of foundation and embankment performance.

DESIGN IMPLICATIONS

The existence of a surficial crust often is recognized and accounted for in analytical procedures for typical design problems. As previously indicated, a surficial crust may have a substantial influence on the performance of earth structures and foundations. The bearing capacity of shallow foundations in or on a surficial crust may be significantly affected by the properties of the crust. Several theories have been presented for estimating the bearing capacity of shallow foundations on a layered system (*40,41*) as well as for evaluating the contribution of a stiff crust to the settlement (*42,43*). For example, Raymond (*44*) indicated that the evaluation of properties of the crust was one of the major uncertainties in analyzing settlement of embankments on clays. The presence of a stiff crust overlying a softer material can change dramatically the distribution of vertical stress when compared with the Boussinesq pressure distribution for a uniform material. Stability of embankments also needs to consider the presence and properties of the crust (*22–25,45*). With only a few exceptions (*38,46*), there are relatively few well-documented field case histories of foundations involving surface crusts to verify the foundation performance and the use of various methods to predict performance. Engineers should take appropriate steps to acknowledge the occurrence of surficial crusts and seek reasonable solutions to design problems.

SUMMARY AND CONCLUSIONS

The development of a stiff, overconsolidated weathered clay crust at the surface of fine-grained sedimentary geologic deposits is relatively common and can have some important implications for geotechnical engineering practice. As a result of the wide range in both physical and chemical crust-forming processes, a complex and highly variable soil mass may result that can be difficult to characterize accurately. The following observations are applicable to the geotechnical behavior of surficial crusts:

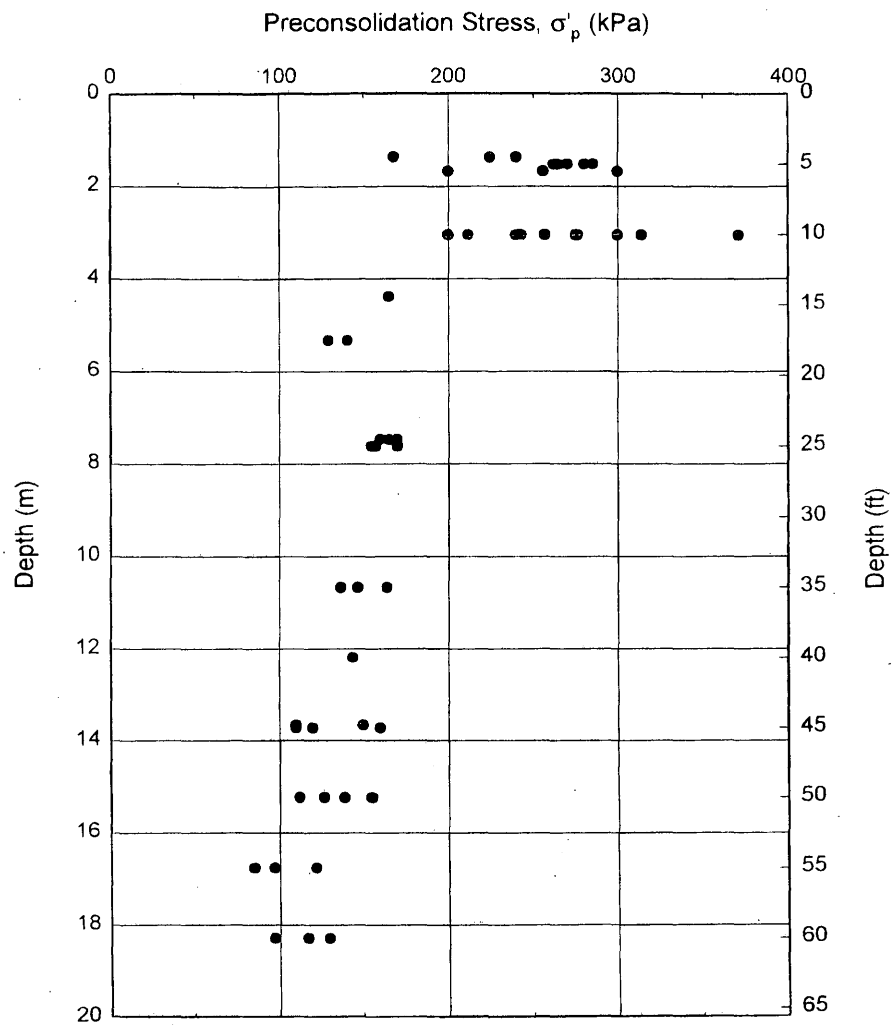

FIGURE 8 Variations in oedometric preconsolidation stress at UMass-Amherst NGES.

1. Crusts are more variable than the underlying unweathered parent deposit.

2. The extent of alteration may vary considerably over short distances and the thickness of a developed crust may be highly variable.

3. The location of the crust often coincides with the zone of maximum movement of the groundwater table, which often enhances the development of the crust. This means that large fluctuations in the magnitude and sign of pore-water pressures are common; therefore, temporal changes in soil effective stresses are common in the crust. Because the thickness of most crusts is limited to a few meters, the changes in effective stress at these shallow depths may be significant.

4. Because the water table fluctuates, the degree of soil saturation above the capillary fringe also fluctuates, and the crust often may be unsaturated.

5. Overconsolidation in the crust is often the result of processes other than simple mechanical unloading. This suggests that soil models that use normalized concepts and property relationships with stress history due to simple unloading from a normally consolidated state to predict such properties as undrained strength or coefficient of lateral stress may not be appropriate.

6. Because of the highly variable nature of the deposit, the geotechnical behavior is less predictable than that of unaltered sedimentary deposits. This means that more effort is required to characterize the properties for geotechnical designs.

7. Because of the developed structure of weathered clay crusts, soil sampling is often difficult, and the results of laboratory tests to predict structural properties such as shear strength may be unreliable and subject to significant scale effects. Therefore, like other significantly structured geologic materials such as residual soil profiles, field tests such as the plate load test or pressuremeter, which provide response of a large volume of soil, are preferred. Usually a larger number of tests is needed to accurately characterize the soil.

8. As a result of the development of secondary soil structure, the hydraulic conductivity of surficial crusts is usually controlled by secondary features such as fissures and joints and may show substantial scale effects.

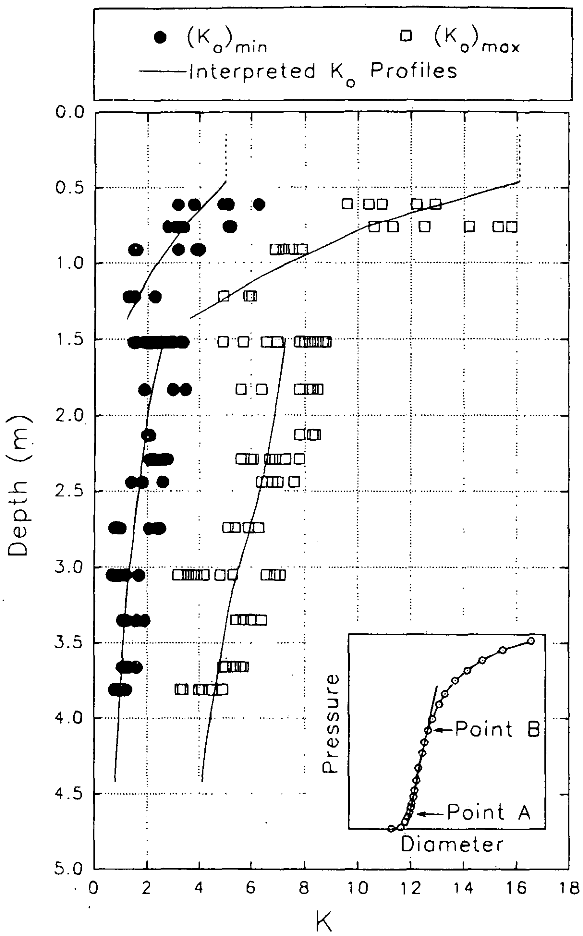

FIGURE 9 Results of interpreted in situ lateral stresses from pressuremeter tests.

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Design Practices in Overconsolidated Clays of New York

VERNE C. MCGUFFEY

The key design practices for construction in overconsolidated clay deposits in New York State are summarized. The differences in prediction technology for heavily overconsolidated soils as compared with normally consolidated soils are highlighted. Rules of thumb for design practices are also included. The design approach used is based on the stress history of the deposit. Overconsolidated clays subject to *overload*-type stress history perform as predicted using classical approaches to settlement and stability. Clay deposits subjected to *desiccated*-type preconsolidation, however, require different approaches in exploration and modeling to properly predict performance. Continuous undisturbed samples are desirable, and many consolidation tests are needed to describe the preconsolidation history accurately. Plots of moisture content versus depth from numerous disturbed samples in the deposit best reflect the type of stress history and therefore are used as a guide to process selection. It is difficult to predict the probability of a slope failure for cuts in natural slopes. The slope stability varies with the rate of shear stress release and rate of water table drawdown. Cutslope failures in overconsolidated clays in New York commonly occur about 7 years after construction. Predictions of time for settlement for desiccated clays overlying normally consolidated clays are difficult to make. Therefore, treatment (such as wick drains) is recommended if the performance objectives cannot be guaranteed should settlement occur in a manner not predicted.

Many design errors have been made and are continuing to be made by engineers inexperienced in prediction of performance of heavily overconsolidated clays. Some of the successful design approaches used in areas of heavily overconsolidated clays in New York State are summarized to help the inexperienced designer. Special features that must be looked into differently than would be done with normally consolidated or lightly overconsolidated clay deposits are also identified. Analysis techniques will not be discussed unless they are unique to overconsolidated clays. These methods and associated rules of thumb can be applied to most heavily overconsolidated clay deposits observed in geotechnical literature.

New York State experience indicates that much of the difficulty in making accurate predictions of soil performance appears to relate to lack of recognition of small variations in soil stratigraphy or parameters that produce a major change in the performance (e.g., major changes in shear strength and consolidation characteristics occur in short distances when the preconsolidation pressure changes from overconsolidated to normally consolidated).

BACKGROUND

Heavily overconsolidated clays in New York State are defined as those clay deposits with preconsolidation pressure (P_p) appreciably higher than the present overburden pressure (P_o). This type of deposit is defined further as having an overconsolidation ratio greater than 2 ($OCR > 2$) or a preconsolidation pressure greater than 7000 kPa (1,000 PSF) over the present overburden pressure. The distribution of overconsolidation commonly takes two forms (Figure 1) (*1*): (a) the overload preconsolidation pattern is identified by its relative straight line distribution from clay surface to bottom, usually paralleling the normal overburden pressure diagram; and (b) the desiccated preconsolidation pattern is identified by its very high preconsolidation level near the top of the layer, decreasing in a parabolic shape to the bottom of the layer or to an overload preconsolidation line.

It is common to find a desiccated pattern grading into an overload pattern in the same deposit. It is less common, but not unusual, to find a desiccated pattern underlain by a second or third desiccated pattern. Occasionally there will be an overload pattern over a desiccated pattern (this usually means that there are two separate geologic clay deposits that may have appreciably different characteristics).

Identification of the preconsolidation load pattern and quantification of the preconsolidation loads through the deposit are probably the two most important items in any design activity in overconsolidated clay deposits.

IDENTIFICATION OF PRECONSOLIDATION LOAD PATTERN

Identification of the preconsolidation load pattern early in the design process is very important so that sufficient sampling and testing can be done to quantify the critical values for the particular design. In New York State (and in technical literature) there is often a wide band of preconsolidation values obtained from tests on the same soil deposit (*1–3*). It is difficult, therefore, to identify clearly the pattern unless a large number of consolidation tests are available (which is seldom economically justified).

New York State uses natural water content tests as a surrogate method to identify the pattern to support the conclusions needed for the design. New York State and others (*1,2*) have observed that natural moisture content (W_n) is inversely related to the preconsolidation pressure in a natural soil deposit if the deposit does not change and therefore can be used as a surrogate to identify the shape of the preconsolidation curve for the deposit. Natural moisture content tests can be obtained from most types of disturbed samples with good results. A large number of disturbed samples can be obtained at reasonable cost, and the results can be used to plan a program to get the expensive undisturbed samples for detailed testing. An example of this type of data is shown in Figure 2 (*1,2*).

Knowing the expected overconsolidation load pattern from the plots of moisture content (W_n) versus depth, an effective undisturbed sampling and testing program can be planned to quantify the

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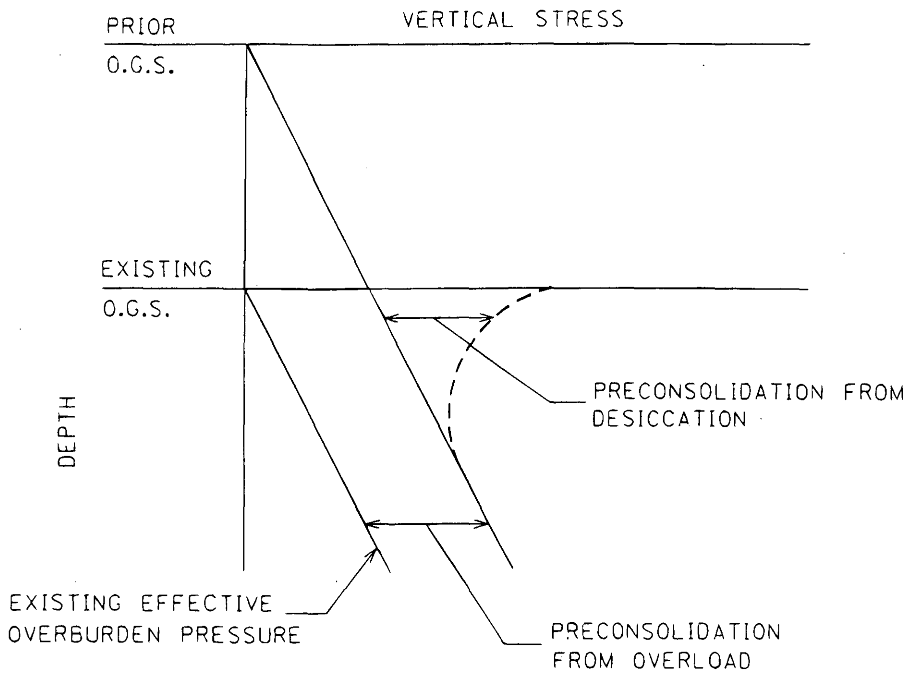

FIGURE 1 Definitions of overconsolidation patterns [after Ladd and Foott (*1*)].

preconsolidation pressure values and refine the shape of the curve. Depth versus W_n profiles also can identify where the most severe conditions can be expected so that the proper undisturbed sampling and testing can be done at the most critical depth. Figure 2 shows a typical plot for a desiccated pattern clay, where W_n increases with depth until the preconsolidation load curve changes from desiccated to overload shape; then W_n decreases with depth.

Sampling disturbance seriously affects the consolidation test values for preconsolidation load and therefore special efforts must be taken on critical projects to identify the possibility of sampling disturbance. The natural moisture content seldom is affected by normal sampling disturbance; therefore, the moisture content profile from numerous disturbed sample holes usually will identify clearly the shape of the preconsolidation curve of the deposit. Any consol-

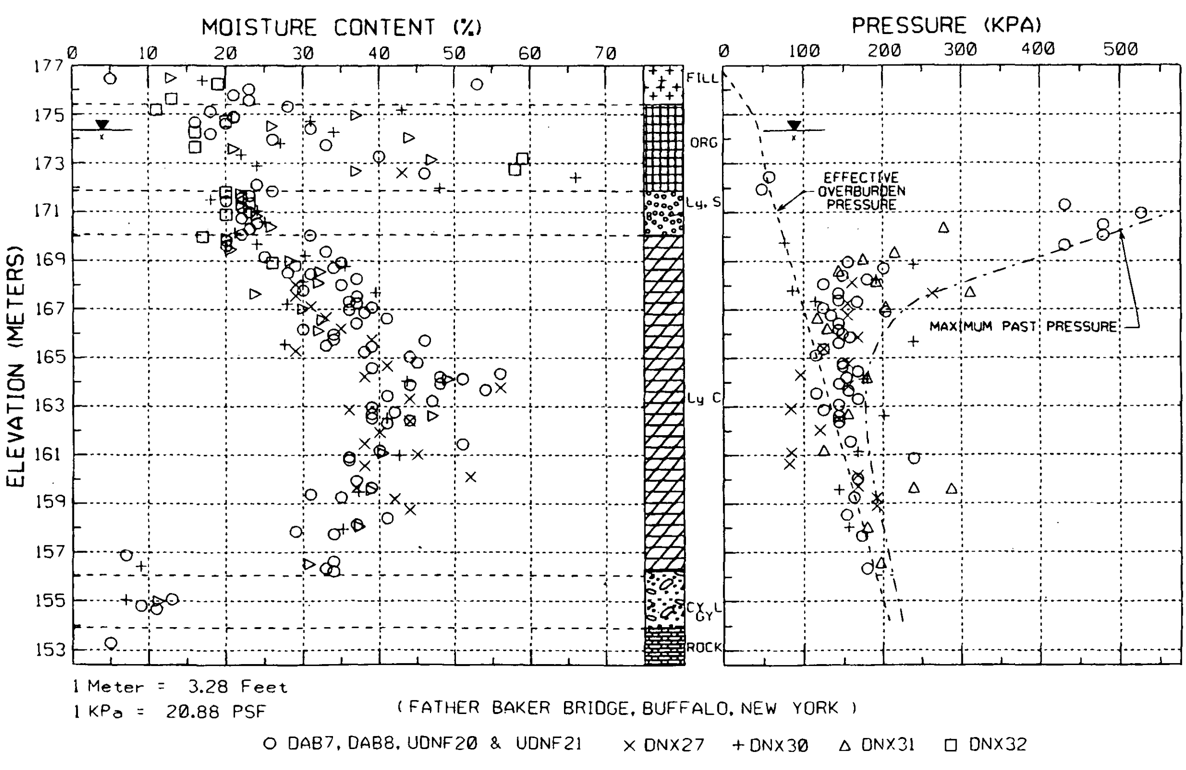

FIGURE 2 Moisture content and maximum past pressure versus elevation.

idation test data that do not agree with that shape of curve should be looked at as suspect in quality. Resampling may be needed if it is important to the design. Although a low value of preconsolidation pressure in an otherwise highly preconsolidated deposit is sometimes due to natural landslide disturbance, it is more commonly a direct result of sampling disturbance. The degree of sampling disturbance can usually be identified by X-raying undisturbed sample tubes before running consolidation tests (*4*).

STABILITY OF NATURAL OR CUT SLOPES

Short-Term Slope Stability

Most cuts in overload-type preconsolidated clays can be made quite steep for a short term because of the high undrained shear strength (S_u), which usually controls short-term cut slope stability. The stability may be estimated using the stability charts from Taylor (*5*) using the shear strength identified from routine testing [consolidated undrained triaxial (C_u), unconsolidated undrained triaxial (U_u), field or laboratory vane shear, cone penetrometer, or unconfined compression (*UC*)]. Short-term cuts in desiccated preconsolidated clays, however, may become unstable because of fracturing that took place during drying. The material in the fracture may control the stability instead of the average shearing strength from unfractured samples. Literature of successful predictions of cut-slope failures in fractured clay deposits is scant. It is advisable, therefore, to design temporary support or use long-term parameters (e.g., drained friction angle Φ_d) for design of cuts in desiccated deposits.

Long-Term Slope Stability

The long-term performance of cuts in natural slopes of overconsolidated clays (both overload and desiccated) is difficult to predict. The undrained shear strength at the time the cut is made may be very high, but this value reduces with time (apparently because of the stress relief) until the shear strength governing failure approaches the drained friction angle at the new overburden stress. This has been discussed more thoroughly previously (*6–8*).

Appropriate circular arc- or wedge-type stability analyses (e.g., Bishop's circular or NAVFAC wedge) (*9*) usually will be adequate to describe the condition of stability of an overconsolidated clay slope, provided that the drained strength (Φ_d) is used in the analysis. A usable value of the drained friction angle can be obtained from drained triaxial, drained direct shear, or consolidated undrained triaxial tests with pore pressure measurements. These values should be compared with existing charts of drained friction angle versus plasticity index (*7,8*). Backfigured data from failed slopes in overconsolidated clays demonstrate that using the drained friction angle (Φ_d) will usually give an appropriate expression of the stability at the time of failure ($FS = 1 \pm 0.05$). (Computerized slope stability analyses with automatic search patterns may produce artificially low factors of safety when using a zero cohesion input because the search moves to the skin of the slope. It may therefore be necessary to include a low value of cohesion in the computer stability analyses so that the computer will search at realistic locations where the observed failure occurs. Inserting 344 to 1033 kPa (50 to 150 psf) of cohesion allows the automated stability analyses to identify correctly the location of the failure surface without changing the factor of safety appreciably.)

The static groundwater table in the system is disrupted when a cut is made and the groundwater surface is lowered below the surface of the new cut slope. It is believed that the excavation produces a negative pore pressure below the cut slope surface, which contributes to the overall short-term stability of the slope. The negative pore pressures dissipate with time, resulting in a reduction of shear strength to a value approaching the drained value (Φ_d). Few clay slopes would remain stable in the long term in normal highway cuts of 1 on 2 (26.5 degrees) to 1 on 3 (18.5 degrees) if the groundwater table remained at the surface of the slope when the preconsolidated clays reach the drained shear strength condition (usually 20 to 27 degrees). Often preconsolidated clay cut slopes fail from 1 to 10 years after construction (an average of 7 years in New York State). One possible explanation, and a methodology to predict the time to failure for long-term stability of cuts and overconsolidated clays, has been given (*7*). This explanation assumes that the negative pore pressure dissipation is the reverse of the normal loading pore pressure dissipation and therefore that the same time factor curves apply. The time to failure then can be estimated from the coefficient of consolidation (c_v) of the soil and the depth to the probable failure plane [usually about 4.6 to 6.1 (15 to 20 ft) in New York State clay slopes]. For example $t = T \times H^2/c_v$, substituting typical numbers, $t_{90} = 0.848 \times$ [6.1 m (20 ft)]^2/0.0093 m^2 (0.1 ft^2)/day $\times$ 365 days/year = 9.3 years) (*7*).]

Care is needed to identify slopes that could have been subjected to old landslide activity. Large excess pore pressure may still exist along the original failure plane that may reduce shear strengths to values below that described by the drained friction angle and the present overburden. A conservative approach is to backfigure an equivalent shear strength along the failure plane assuming a factor of safety of 1 on the old failure surface, and then to complete the design assuming no increase in shear strength. A detailed investigation must be conducted to define clearly all necessary parameters if the conservative approach is not acceptable or cost-effective. This investigation might include extensive explorations and testing along with long-term pore pressure and movement measurements.

One way of reducing the risk of slope instability from a permanent high water table in overconsolidated layered silt and clay systems (which seldom exhibit much drawdown of the water table) is to excavate an additional 2 to 3 ft of the clay along the slope and replace it with an open stone fill slope protection (*10*), which has the effect of lowering the water table.

STABILITY OF EMBANKMENTS ON OVERCONSOLIDATED CLAYS

Expectation of very high in-situ undrained shear strength in heavily overconsolidated clay can lull the investigator into complacency because a stability situation seldom arises from embankments constructed on overconsolidated clays when undrained shear strength controls. There are situations in which an exception in the soil system controls the performance of the construction, and therefore some exceptions will be discussed.

- Even small embankments placed on slopes that have failed (old landslides) can set off new failures even though the average shear strength is very high.
- Some overconsolidated clays have been subjected to major previous failures resulting in micro shear planes (low-strength clays between blocks of very-high-strength overconsolidated clays) (New York State Department of Transportation, Morrows Corners, West Granville, 1958, unpublished data). A special sampling and testing program may be needed to identify the situation and obtain suitable

parameters for shear strength. Use of a drained strength parameter (Φ_d) is usually suitable if there is no residual pore pressure.

- Overconsolidated clays of the desiccated pattern usually have very high strength clays over much softer clays. It is usually not appropriate to use the very high shear strength of the surface soils because they are often fractured and fissured. Because there is no easy way to obtain quality test results for these fissures, a conservative approach is usually taken. One such approach is to assume that fissures are filled with sand and use a 35 degree drained friction angle in the heavily overconsolidated clay instead of the measured shear strength of the clay. The measured shear strength can be used if there is confidence that the data were obtained from soil below the zone of fissures.
- In urban areas, discontinuities should be expected in the stiff clay where old foundations or utility lines were excavated through the very strongest part of the desiccated clay. This often leaves areas of very low strength (sand, debris, etc.) in an otherwise high-shear-strength clay deposit. It is suggested that a conservative approach be used as in the previous paragraph.
- Strength gain from loading of soft clay beneath heavily overconsolidated clay in a desiccated clay system should not be depended on without extensive study. The heavily overconsolidated surface clay has a low permeability that may slow the vertically upward drainage from the soft clay, thereby making it very difficult to predict time for strength gain. The concepts of predicting time for drainage will be discussed further under the section "Settlement of Overconsolidated Clays."
- Some overconsolidated clays are very sensitive and may dramatically change characteristics under loading when overstressed (*11*). The St. Lawrence clays in New York State have natural water content more than 10 percent over the liquid limit and are very sensitive. Failures in sensitive clays may occur at the post peak or residual strength (which may be 20 to 70 percent of the peak natural strength) because of progressive failure. The lower value of shear strength may control fill stability if the embankment being constructed creates a condition of overstress at any place beneath the embankment (*11,12*). As a general rule any overconsolidated clay that has a natural moisture content 5 percent or more above the liquid limit is highly susceptible to this large, and sometimes rapid, loss of strength (*13*). A method of analyzing the stability of sensitive clay systems subjected to overstress is described by Gemme (*11*).

SETTLEMENTS OF OVERCONSOLIDATED CLAYS

The magnitude of settlement of overconsolidated clays usually can be predicted quite accurately using the standard consolidation equations developed by Terzaghi (*14*) using the recompression ratio (*RR*) up to preconsolidation pressure (P_p) and using the compression ratio (*CR*) above the preconsolidation pressure (*13*).

The time for consolidation to occur follows the standard consolidation equations using standard test parameters for the overload pattern preconsolidated system.

The time for consolidation to occur for the desiccated system, however, seldom can be predicted accurately. Often the surface soil is so heavily preconsolidated that it may effectively block the drainage of the underlying near-normally-consolidated clays (the coefficient of permeability may become very small—less than 10^{-7} cm/sec). If this occurs a vertically upward component of drainage may no longer occur in the underlying soft clay. If the bottom of the layer has an impermeable boundary such as rock, there may be nearly a zero rate of vertical drainage and most of the pore pressures must dissipate laterally. Rough estimates of the rate of lateral pore pressure dissipation can be made using the procedures described by Ladd and Foott (*15*) or an approximation from McGuffey (*16*). Some desiccated deposits have shrinkage cracks (fissures) filled with silt or sand, which can allow upward drainage. Unfortunately, it is nearly impossible to estimate the overall contribution of this effect to the rate of settlement expected. One project in Buffalo (Young Street Arterial) exhibited changing boundary conditions with time. The pore pressure measurements indicated primarily one-way downward drainage to a thin gravel layer over rock at the beginning of loading. As loading continued, the downward component stopped and all drainage appeared to be lateral with a small component vertically upward through the desiccated clay.

The above type of performance is nearly impossible to predict, and therefore it is often prudent to treat the area with sand drains or wick drains as an economical guarantee of performance if vertical drainage is uncertain in desiccated clay systems.

HEAVE EXPANSION OF OVERCONSOLIDATED CLAYS

Heave in excavations of overload preconsolidated clays in New York State is generally so small as to be neglected (*17,18*). To summarize New York State experience (*17*), "The swell potential is not considered to be large unless the soil is *desiccated*, the groundwater table at a considerable depth, and the soil contains clay mineral particles with expansive characteristics."

Heave in overconsolidated clay due to desiccation often occurs when excavations expose the clay to free moisture. This is most noticeable with high bentonite clay that has been dried back below the shrinkage limit during much of its previous history. Damage similar to frost heaves can occur unless special treatment is used such as allowing preexpansion before installation of the final roadway surface or designing so that the expansive clay is unable to obtain additional moisture.

STRUCTURE FOUNDATIONS

Shallow Foundations

Shallow foundations usually are no problem on either type of overconsolidated clay because the footing loads are very small compared with the previous loading of the clay system and therefore the clay has adequate strength and exhibits little or no compression from the structure loading system. A few situations that should be investigated in detail for shallow foundations on overconsolidated clays are given below:

- Preconsolidated clays with a natural moisture content below the plastic limit may heave if subjected to free water as a result of construction (e.g., inadequate footing drainage). Clays with natural moisture content below the shrinkage limit are highly susceptible to damaging heave when exposed to free moisture, and special precautions are required.
- Footing loads seldom will cause settlement of either overload or desiccated clay systems, but if fill is being placed around the structure foundation in a desiccated clay, the settlement of the softer underlying clay will have to be evaluated to determine the effect of grading settlements on the structure performance.

Deep Foundations

Deep foundations in overload pattern preconsolidated clays usually do not present problems and perform as predicted using standard design practices. Deep foundations through a desiccated pattern preconsolidated clay may be damaged by settlement of the underlying softer clay if settlement is not predicted accurately or accounted for. This requires careful investigations to determine whether any settlement will occur. The design may also have to consider large pile drag if there is grading around the structure that would cause consolidation of the underlying softer clay.

It may be more economical sometimes to redesign the structure for zero net load than to increase the number of piles to account for pile drag. The situation becomes more complex when settlement is still occurring from a previous load on the site. The net load must then be reduced to a value below the preconsolidation pressure curve or back to the previous original ground surface load. On one building project in New York City, the cost of the deep foundation was more than doubled to account for the settlement remaining from old fill.

Deep foundations should go completely through the softer material under the very stiff surface layers and not get founded in the stiff upper layer by those using designs based on pile-driving blow counts alone.

ADDITIONAL RULES OF THUMB

- Overload pattern preconsolidated clay deposits exhibit similar stress history over large areas and often can be related to a definable geologic deposit [e.g., glacial Lake Albany clay is not found above USGS elevation 70.15 m (230 ft), and therefore the preconsolidation load anywhere in the deposit can be closely estimated by subtracting the present ground elevation from elevation 70.15 m (230 ft) and multiplying this by the soil effective unit weight of 1.042 g/cm^3 (65 $\pm$ lb/ft^3)].
- Desiccated pattern overconsolidated clay deposits also cover wide areas, but local variations can have a major effect on performance. For example, the depth to rock on the Lockport Expressway Project in western New York State varied across the project. There were areas where rock was immediately under the stiff clay with no underlying soft clay adjacent to areas with appreciable amounts of soft clay underlying the stiff clay. This required berms next to moderate fills and no berms next to higher fills, and there were large differences in the settlement for the same height of fill. To prevent being surprised by these types of variations, a large number of subsurface explorations are desirable to define the controlling conditions.
- Samples taken at 5-ft or wider intervals in desiccated pattern soils can miss the most critical (weakest and most compressible) soil in the deposit. Therefore, continuous samples are essential in desiccated clay deposits.
- A small error in identifying and testing the weakest portion of a desiccated pattern deposit can result in shear failures in situations where the stiff surface layer has been removed (e.g., streams or canals—Figure 3). The lowest undrained shear strength strongly controls stability in these situations.

COMMON PROBLEMS

Three common errors that can be disastrous to the designer when working with overconsolidated clays are as follows:

1. Failure to recognize a highly sensitive overconsolidated clay that may be subjected to high shearing stresses where a part of the deposit is overstressed. This error can result in rapid loss of shearing strength and resultant shear failures under loading (and flow slides of cuts in natural slopes).
2. Failure to recognize a cutslope where the groundwater table will not draw down sufficiently (before shear stress relief occurs) to allow for long-term stability of the cutslope.
3. Failure to recognize the potential for heave in desiccated clay deposits.

The first situation usually is easily identified in New York State, and there has not been a failure of this type for over 20 years. The second situation is common in New York State, with failures occurring about once every 2 to 3 years. These failure can be expected to continue because the prediction technology is not dependable, and the cost of a detailed design often exceeds the cost of correction if there

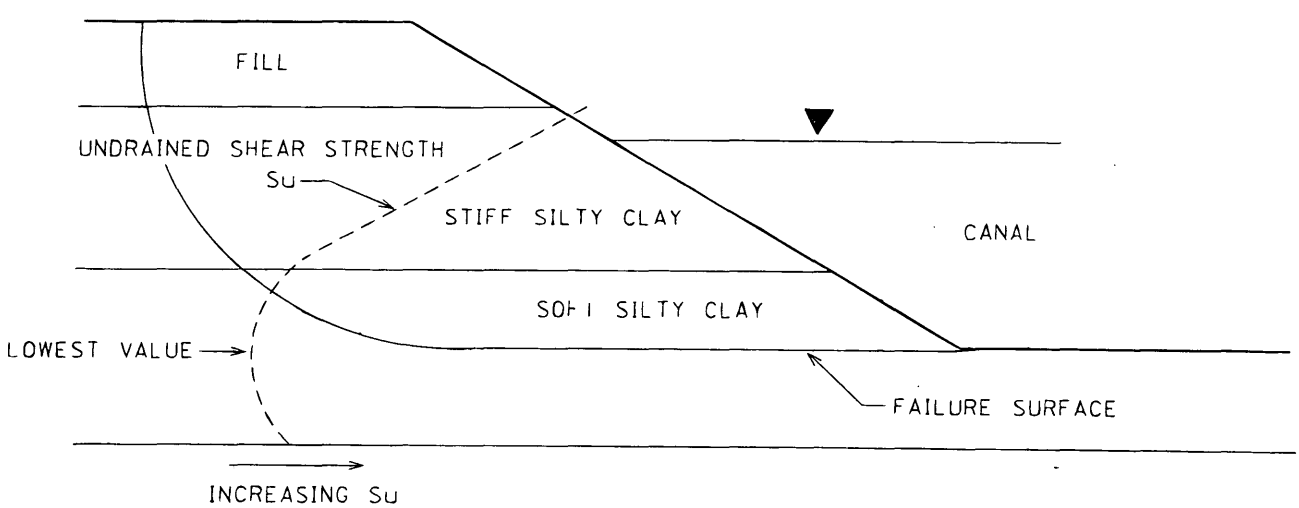

FIGURE 3 Example site where lowest value of undrained shear strength S_u controls performance.

is a failure. The third situation seldom is important for transportation facilities in New York State because the design requirements for frost protection also prevent heave problems. It is important for buildings, and the availability of moisture must be controlled to prevent costly damage.

CONCLUSIONS

- Identifying the pattern of preconsolidation load of the clay systems is one of the most important features of any investigation in overconsolidated clays.
- Identifying the pattern can best be made by a large number of plots of moisture content versus depth from numerous disturbed sample borings.
- The quantification of the preconsolidation stress level can be obtained by good-quality undisturbed samples and normal consolidation testing programs.
- The undrained shear strength of the deposit is satisfactory for determining stability under loading conditions on flat terrain.
- For unloading conditions (cuts) or loading on sloping terrain, it is essential to determine the drained friction angle of the deposit.
- It is nearly impossible to predict accurately the time for settlement to occur for the desiccated pattern systems because of the difficulty in defining the boundary conditions and the drainage parameters. Therefore, treatments such as sand drains are often preferable to attempting to get enough explorations to make accurate predictions.
- The potential for heave should be of concern on most desiccated pattern overconsolidated clay systems.

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Engineering Properties of Overconsolidated Pleistocene Soils of Texas Gulf Coast

MICHAEL W. O'NEILL AND GIL YOON

Engineering soil properties, including undrained shear strength, overconsolidation ratio (OCR), coefficient of earth pressure at rest, Young's modulus, and cyclic degradation factors, obtained by various in situ and laboratory testing methods are presented for two Texas Gulf Coast sites. Soil deposition was deltaic, and preconsolidation occurred as a result of desiccation, producing local variability, as well as variability from site to site. The most comprehensively studied property, OCR, is in the range of 3 to 7 at Site A below a depth of 3 m, in which the soils to a depth of 8 m were formed in a pro-delta environment. Site B, at which the soils to a depth of about 11 m were formed in a backswamp environment several kilometers from Site A, indicated that OCR values are two to three times as high. Properties at Site A are probably appropriate for conservative geotechnical design at most sites in the geographical area.

This paper is concerned with the engineering properties of two Pleistocene terrace formations found along the Gulf Coast, generally west of the Mississippi River and north of the Rio Grande, exposed at the surface to about 100 km inland from the present coastline. Both formations have similar depositional histories. The lower formation, termed the Upper Lissie formation or Montgomery formation (the latter designation will be used here), was deposited on a gentle slope on an older Pleistocene formation during the Sangamon Interglacial Stage by streams and rivers near the existing coast, where numerous large and small river deltas developed. After deposition, the nearby sea level was lowered during the first Wisconsin Glacial Stage, producing desiccation and consolidation of the Montgomery soils, which consisted primarily of clays and silts. At the beginning of the Peorian Interglacial Stage, as the glaciers were retreating, the sea level returned to its previous level, producing a preconsolidation effect within the Montgomery formation. At the same time, rivers and streams produced sedimentary deposits on top of the slightly seaward-sloping Montgomery from the existing coastline to about 60 km inland from the present coastline. The resulting new formation, primarily a fresh-water deposit sloping toward the Gulf of Mexico, has characteristics typical of deltaic environments, including point bar, natural levee, backswamp, and pro-delta deposits within, beside, and at the termination of distributary channels. This formation is known as the Beaumont formation in Texas. After deposition, the nearby Gulf of Mexico receded by about 125 m once more during the late Wisconsin Glacial Stage, inducing desiccation in the Beaumont and redesiccating the underlying Montgomery. Finally, with the recession of the late Wisconsin glaciers, the sea level returned to its present level, leaving both formations preconsolidated through desiccation (*1,2*). A map of Beaumont-aged distributary channels within the Houston, Texas, area is shown in Figure 1 (*3*). Most of these channels became inactive and were covered by a few meters of clay during Recent times. Because of the differing depositional processes, Williams (*3*) has predicated that clays, now overconsolidated, that were deposited as backswamp soils have higher overconsolidation ratios (OCRs) and therefore have different properties from those that were deposited within or in front of microdeltas. The depositional process left thin seams of fine sand or silt within the primary deposits of clay in the Beaumont. Weathering of the Montgomery formation before deposition of the Beaumont leached some of the clay from the soil, resulting in soils near the surface of the Montgomery that are more silty and sandy than the soils of the Beaumont. The Beaumont-Montgomery contact is unconformable, and rather significant changes in water content, Atterberg limits and strength properties often occur there.

Desiccation produced a complex network of joints in both formations that were filled with solids during succeeding flooding events. This infilling restricted the return of the soil, through swelling, to its state of strain before desiccation, resulting in high values of the effective coefficient of earth pressure at rest, K_0. Al-Layla (*2*) characterized the clays as existing in "lumps" with an average of 2 to 4 mm between closed joints in each direction. Evidence that this process produced a "gilgai" structure (surface waves produced by wetting due to horizontally and vertically varying density of joints and resulting chemistry changes) in the present Beaumont formation is presented by Georghiou et al. (*4*). Mahar and O'Neill (*5*) surmised that this joint structure resulted in spacewise variable preconsolidation pressures over a few tenths of millimeters, with the highest capillary stresses (highest preconsolidation pressures) near the joint surfaces and the lowest in the interior of the blocks. Higher preconsolidation pressures should also exist in gilgai mounds, spaced 15 to 40 m apart, rather than in the troughs between the mounds. Therefore, not only are the Beaumont and Montgomery formations variable macroscopically, depending on the location of the point of investigation relative to distributary channels (Figure 1), but they may also be variable on a typical site scale because of systematically varying joint patterns (gilgai) and, on a microscale, because of variable capillary stresses and the presence of heterogeneous materials deposited within open joints and horizontally as seams. These characteristics influence the engineering properties.

Capillary stresses in the Beaumont were high enough to preconsolidate it through its entire thickness. Vertical effective stresses produced within the Montgomery by overburden loading from the Beaumont, which is 8 to 12 m thick within Houston, are considerably less than the capillary stresses produced by desiccation and loss of buoyancy at the base of the Montgomery. The Montgomery therefore remains preconsolidated for its entire thickness, about

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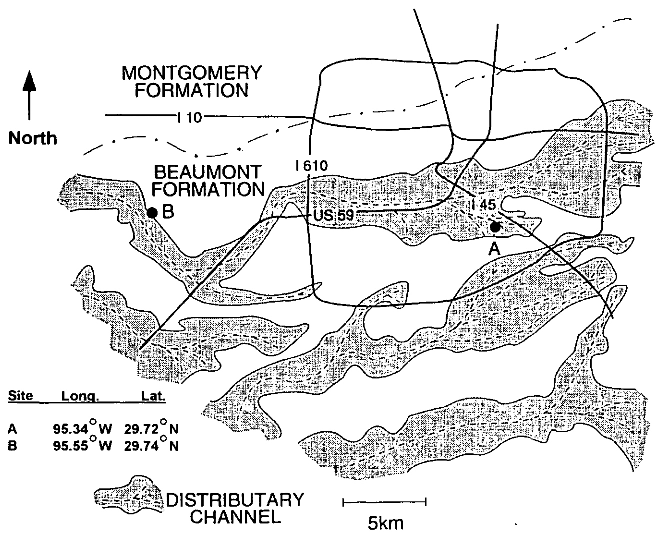

FIGURE 1 Location of distributary channels within Beaumont formation in the Houston, Texas, area (*3*).

150 m, allowing relatively large and heavy structures to be constructed in the Houston area by taking advantage of the deep preconsolidation zone through the use of partially compensated rafts.

GENERAL ENGINEERING PROPERTIES

Two sites (Sites A and B, Figure 1) are examined that have been profiled geotechnically by various means, with emphasis on the National Geotechnical Experimentation Site (NGES) at the University of Houston, Site A. Site A is a microdelta depositional site within the Beaumont formation, whereas Site B is a backswamp site within the Beaumont formation near a natural levee. These sites represent the lower and upper limits, respectively, for theoretical preconsolidation in the region, and possibly in the Beaumont formation. A general profile for Site A is shown in Figure 2. The Beaumont-Montgomery contact is at a depth of about 8 m. At Site B, the contact appears to be at a depth of about 11 m. At both sites the piezometric surface is at a depth of about 2 m.

The mineralogy of the soils at the two sites is somewhat different, as characterized by the average index properties (I_p and w_L) in Table 1.

A profile of the OCR (OCR = maximum past vertical effective stress/present vertical effective stress) is presented in Figure 3 for both sites, as determined by relatively sparse data from Shelby tube samples using the indicated laboratory test methods. Individual values are shown only for Site A. The trend lines are strictly visual fits. More details on the interpretation of the laboratory test methods are presented by Mahar and O'Neill (*5*) and O'Neill et al. (*6*).

The resolution of test data is insufficient to delineate any difference in OCR at either site as one passes from the Beaumont into the Montgomery (Figure 3), despite the geological history and the differences in index properties.

UNDRAINED SHEAR STRENGTH

Typical stress difference/pore-water-pressure relations are shown in Figure 4 for anisotropically consolidated, saturated, undrained tri-

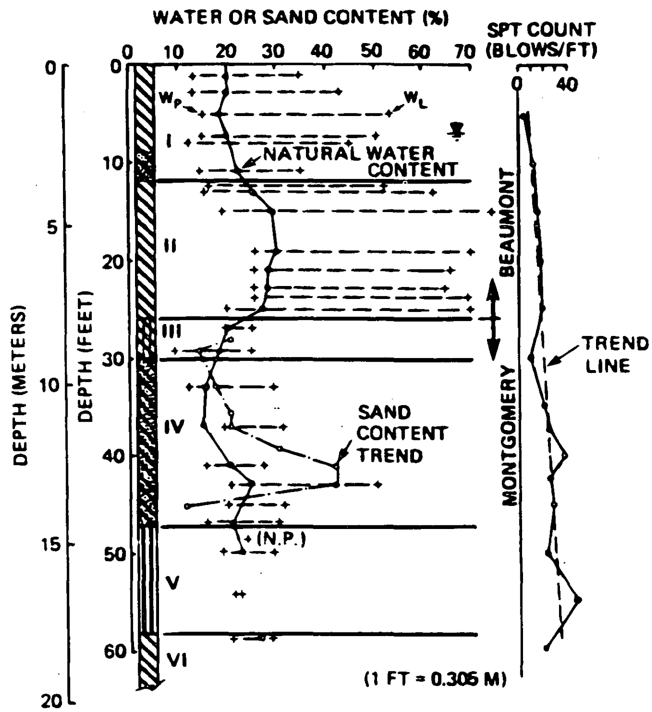

FIGURE 2 General profile of Beaumont-Montgomery sequence at Site A (*5*). Stratigraphy: I, very stiff gray and tan clay (CL-CH); II, stiff to very stiff red and light gray clay (CH); III, medium stiff light gray very silty clay (CL); IV, stiff to very stiff light gray and tan sandy clay with sand pockets (CL); V, dense red and light gray silt with clayey silt and sand layers (ML); VI, very stiff red and light gray clay (CL).

axial test specimens (CAU tests) from the Beaumont formation at Site A. Samples were trimmed horizontally and tested vertically in triaxial cells. σ_a represents axial stress (horizontal direction in ground) or stress in the direction of compressive loading. At the end of the consolidation stage, σ_a was equal to the estimated K_0, discussed later, times the present vertical effective stress, σ'_{vo}. σ_l is lateral stress in the triaxial stress system, which was set equal to the average of σ'_2 and σ'_3 for the horizontal specimen, or $0.5(1 + K_0)\ \sigma'_{vo}$. The test therefore models horizontal loading.

Stratum II soils (lower portion of the Beaumont, Figure 2) are more blocky than those in Stratum I (upper portion of the Beaumont) and so exhibit a more decided "knee" at lower axial strains (ϵ_a) than in Stratum I. Both soils are dilative beyond a major principal strain of about 1.5 percent.

TABLE 1 Index Properties at Sites A and B

	Beaumont			Montgomery		
		Avg Index Properties			Avg Index Properties	
Site	Depth Range (m)	I_p	w_L	Depth Range (m)	I_p	w_L
A	0–8	42	61	8–20	15	29
B	0–11	35	55	11–35	25	37

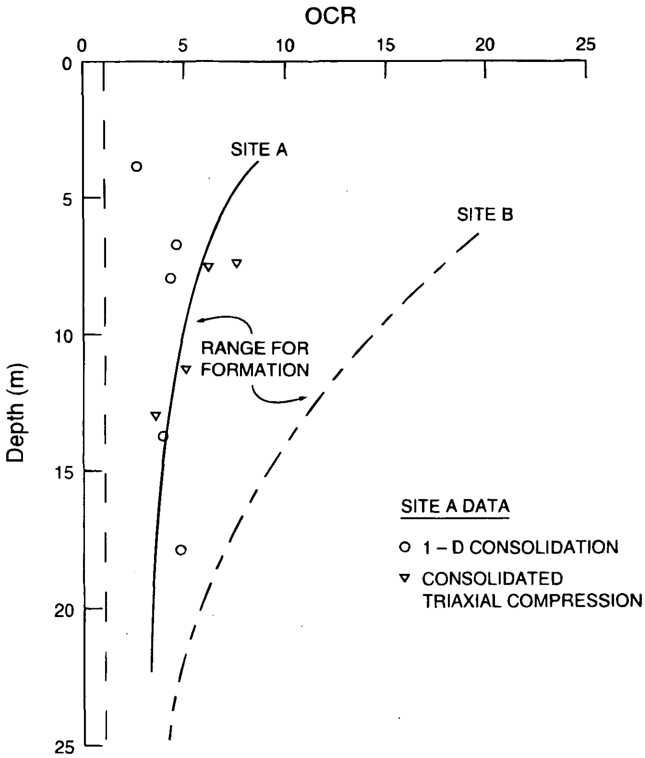

FIGURE 3 OCR-depth profiles at Sites A and B (*3,5*).

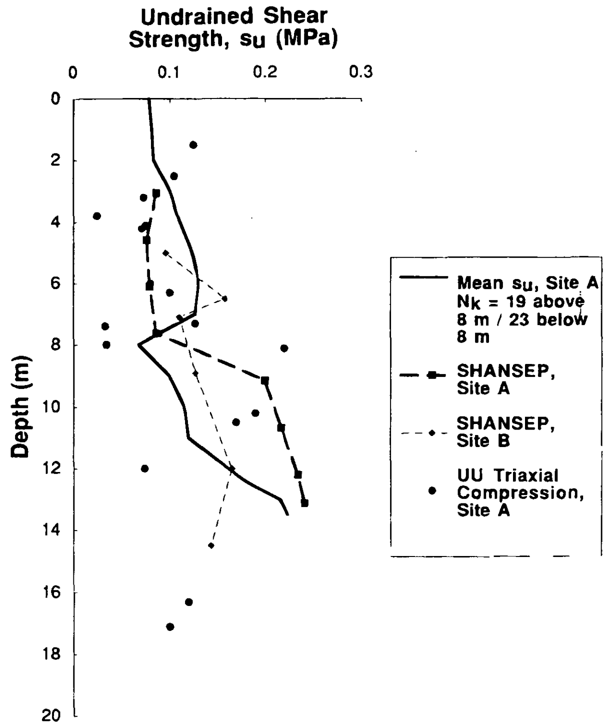

FIGURE 5 Relationship of undrained shear strength to depth at Sites A and B (*3,5*).

Undrained shear strength (s_u) is profiled by several methods at Site A in Figure 5. Also shown is a profile of s_u at Site B by the stress history and normalized soil engineering properties (SHANSEP) method (*3*). UU triaxial compression tests, in which the total, isotropic confining stress is equal to the total overburden stress, exhibit a wide variation in shear strength, reflecting the variable local joint structure and the presence of sand or silt seams. The SHANSEP method provides a marked improvement in consistency of s_u values at Site A, with the only major difference being that of the values below 8 m (within the Montgomery) from those above (within the Beaumont). At Site B, SHANSEP tests indicated higher s_us within the Beaumont than within the Beaumont at Site A, which is expected because of the differences in preconsolidation (capillary) pressures at the two sites, owing to their different micro-depositional environments. In the upper Montgomery, the opposite effect is evident, possibly because the clay in the Montgomery has a higher I_p (is less weathered) at Site B.

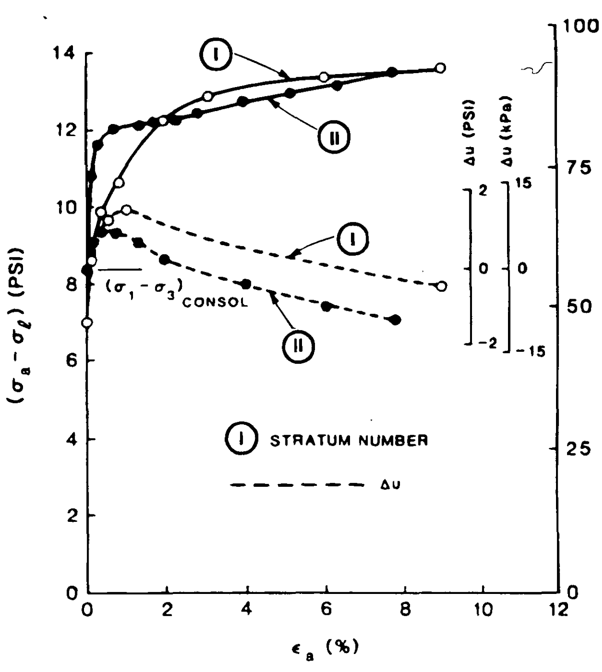

FIGURE 4 Typical stress-strain-pore-water pressure relations for Beaumont formation soils.

The most consistent routine procedure for profiling s_u at Site A appears to be the cone penetration test (CPT). The curve labeled "from mean q_c" shown in Figure 5 is an average s_u relation from 16 electronic CPT tests, all made within a zone 30 m square, using s_u =, where q_c is the cone tip resistance. Statistical properties of the variation among individual CPT soundings are discussed by O'Neill (*7*). A sense of that variability is shown later in this paper in the section on CPTU profiling. Note that the Beaumont-Montgomery contact is evident by a reduction in s_u at a depth of 8 m, where a thin zone of waterbearing silt exists atop the Montgomery.

A relationship between s_u and OCR at Site A for both the Beaumont and Montgomery formations is suggested in Figure 6, where w_L is liquid limit and σ'_{vo} is the present vertical effective stress, using total unit weights of 19.9 kN/m^3 in the Beaumont and 20.7 kN/m^3 in the Montgomery to compute σ'_{vo}. The data were developed from SHANSEP triaxial compression tests on K_0-consolidated, undrained (CU) vertically trimmed triaxial test specimens. w_L is used as a surrogate for ϕ', the effective angle of internal friction.

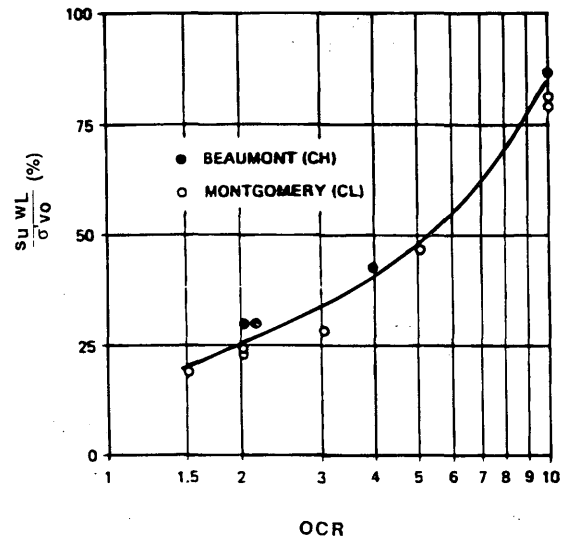

FIGURE 6 Nondimensional relationship between undrained shear strength and OCR for Beaumont and Montgomery formations (5).

Alternatively, Williams (1), using data from both Sites A and B, as well as from three other well-documented sites in the Houston area, determined a direct empirical relationship between s_u and OCR for the Beaumont formation:

$$s_u/\sigma'_{vo} = 0.24\text{OCR}^{0.6} \quad (1)$$

The range of validity of Equation 1 is $1 < \text{OCR} < 20$.

CPTU PROFILING

The CPTU test holds promise in profiling the Beaumont and Montgomery soils. Figure 7 shows a typical result for a CPTU test at Site A, in which pore-water pressures were measured on the sleeve just behind the cone tip on a 14-mm diameter, Fugro-type electronic cone. There is a clear change from positive to negative pore-water pressure (u) between Strata I (sandy, silty clay) and II (plastic clay), with relatively high values of $+u$ being observed in Stratum IA, which contained thin seams of waterbearing sand and silt. Again, the value of u changes from negative to positive when Stratum IV (very sandy clay) is reached, although the very top of the Montgomery (Stratum III) was not distinguished by a change of sign in u. Once interbedded silt layers were encountered below about 12 m, the sign of u became erratic.

Figure 8 shows results of a CPTU test made about 50 m from that shown in Figure 7 but in which the piezometric element was located on the cone tip. There are no negative pore-water pressures in this case, and it is difficult to distinguish strata, except that the Beaumont-Montgomery contact is clearly indicated by a sudden drop in u to near zero. The extreme variation in u may not reflect the variability of the soil but may instead be characteristic of continual plugging and unplugging of the piezo element by blocks of clay that attach and detach from the tip of the cone.

The cone with the piezo-sensing element on the tip appears to be the more appropriate for sensing pore-water pressures at Site A, as indicated by the results of dissipation tests shown in Figure 9. The sleeve element exhibits essentially instantaneous dissipation, possibly because of incomplete contact between the element and the stiff soil, whereas the tip element exhibits a dissipation pattern more representative of normal consolidation processes. If this hypothesis is correct, the magnitudes of the values of u presented in Figure 7 are probably incorrect, although the signs may be correct. Friction ratio (FR) values in Figures 7 and 8 are generally representative of

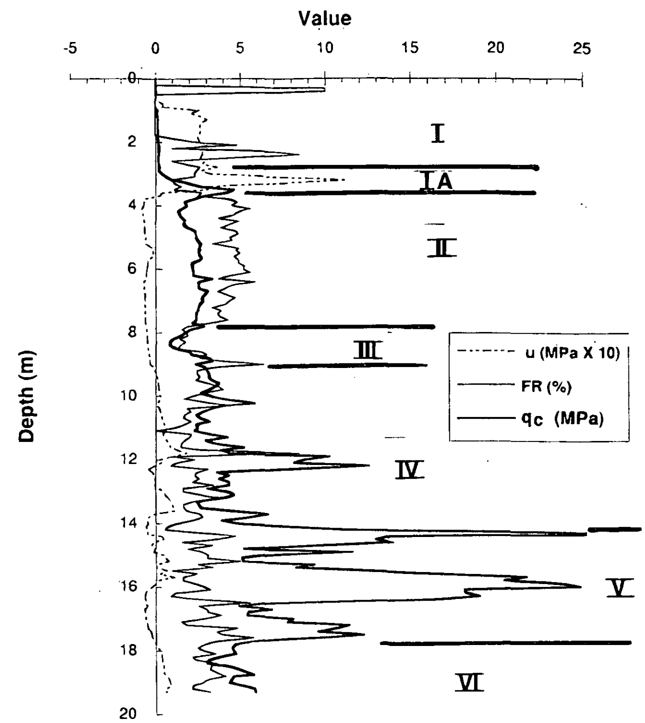

FIGURE 7 CPTU record No. 1 (Piezo element on sleeve), Site A.

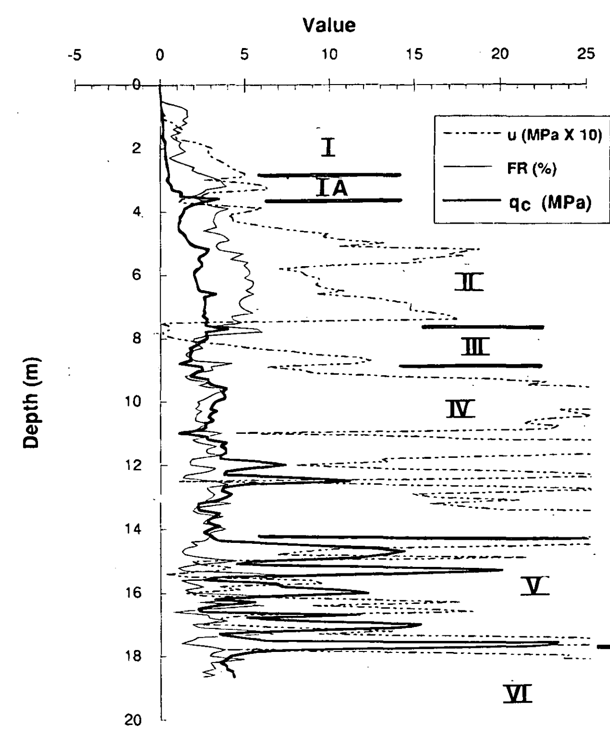

FIGURE 8 CPTU record No. 2 (Piezo element on tip), Site A.

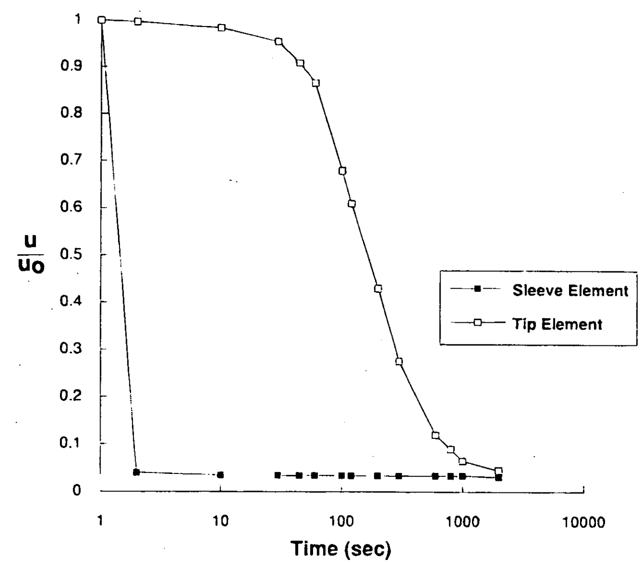

FIGURE 9 Pore-water pressure dissipation patterns: sleeve piezo element versus tip piezo element at depth of 8.6 m, Site A.

the soils described in Figure 2. q_c values at the two test locations shown in Figures 7 and 8 (50 m apart) are superimposed in Figure 10 to provide some indication of consistency. Coherence is high above a depth of about 12 m. The same general pattern can be observed below 12 m, but spike peaks vary slightly in elevation and more greatly in amplitude, indicating density variations in the silt and sand seams.

Mayne (*8*) proposed a practical method of determining a semicontinuous OCR profile from q_c readings, which has obvious potential advantages over profiling OCR using a few expensive laboratory tests, as follows:

$$\text{OCR} = [K\,(q_c - \gamma z)]/\sigma'_{vo} \tag{2}$$

where

K = site- or formation-specific constant of proportionality,

γz = total vertical stress at depth z, and

σ'_{vo} = present vertical effective stress in depth z.

In Figure 11 OCR was computed from Equation 2 at 0.1-m depth intervals from the data in Figure 8, using $K = 0.2$ (the optimum value for Site A) and then fitting the pointwise variable results with a second-order least squares regression line. All (four) computed OCR values above 20 were discarded.

Mayne and Bachus (*9*) have also suggested that OCR can be predicted from the CPTU u values. The u data from Figure 8 were plotted at 0.1-m depth intervals in Figure 12 and were then fit with a second-order least squares regression line. Mayne and Bachus, considering pore pressure generation from expanding cavity theory, propose that

$$\text{OCR} = a\,[(u/\sigma'_{vo}) - 1]^b \tag{3}$$

Factors a and b were shown to be 0.38 and 1.33 respectively from theory, and, using these values, Mayne and Bachus were able to profile OCR in the Yorktown formation in Virginia. At Site A the optimum values of a and b are 0.31 and 1.20, respectively, using the u values from the continuous, fitted relationship in Figure 12. Results of both the Mayne and Mayne-Bachus methods for Site A are shown in Figure 13. Both give generally similar results, and both

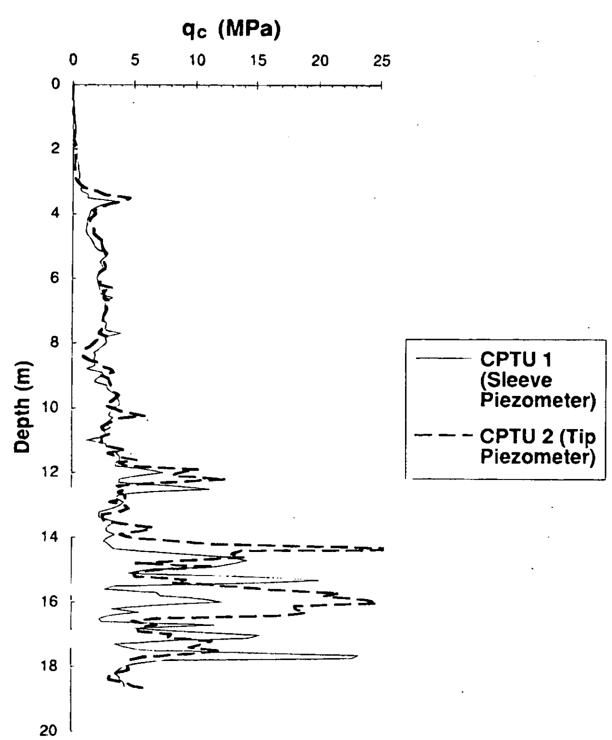

FIGURE 10 Comparison of q_c from CPTU records at Site A.

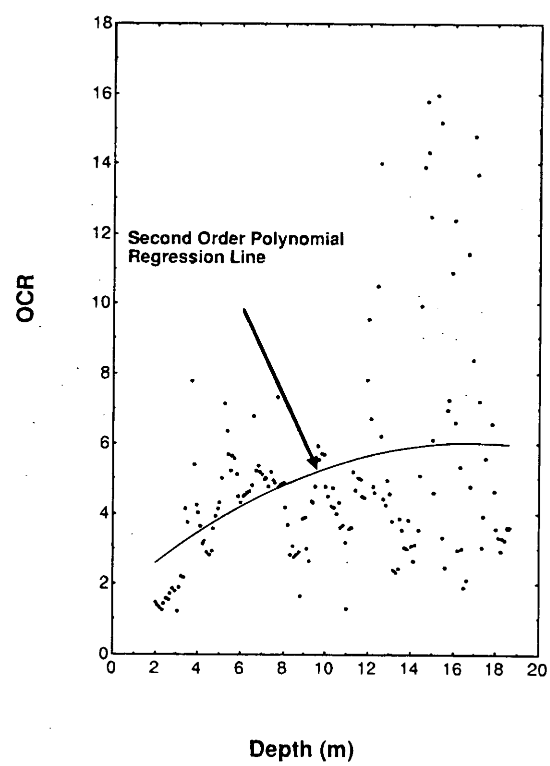

FIGURE 11 OCR versus depth from Mayne method using q_c from Figure 8.

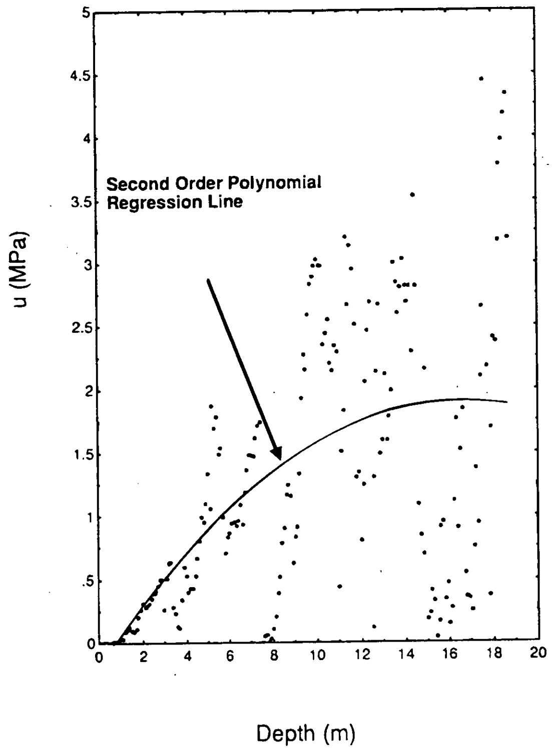

FIGURE 12 Fitted pore-water pressure for record in Figure 8.

provide a reasonable fit to the triaxial and one-dimensional consolidation test data, as fitted visually in Figure 3, but it is unclear at present which method is the more appropriate one.

COEFFICIENT OF EARTH PRESSURE AT REST (K_0)

K_0 has been estimated at Site A by numerous methods, as indicated in the legend to Figure 14. Results are from random positions around the site. The test locations for the two dilatometer soundings were at the extremities of the site, approximately 120 m apart, whereas most of the other tests were nearest DMT1. Interpretations of the dilatometer test (DMT) were in accordance with work by Marchetti (*10*). Good correspondence between DMT1 and DMT2 is observed except at the depth of 8 m, at the surface of the Montgomery formation, which indicates some degree of inconsistency at the unconformable contact. There is also general agreement in the patterns of K_0 between the DMT and the FHWA stepped blade (*11*), although the stepped blade is somewhat more variable. The self-boring pressuremeter test (SBPMT) (*6*) yielded two very high values (at 6 and 18 m), but otherwise was consistent with the other in situ tools.

The familiar correlation of Brooker and Ireland (*12*), based on OCR measured in the laboratory and or index properties, tends to provide a good fit of the directly measured values of K_0. The trend line for the computations of K_0 from the Brooker and Ireland method is shown in Figure 14.

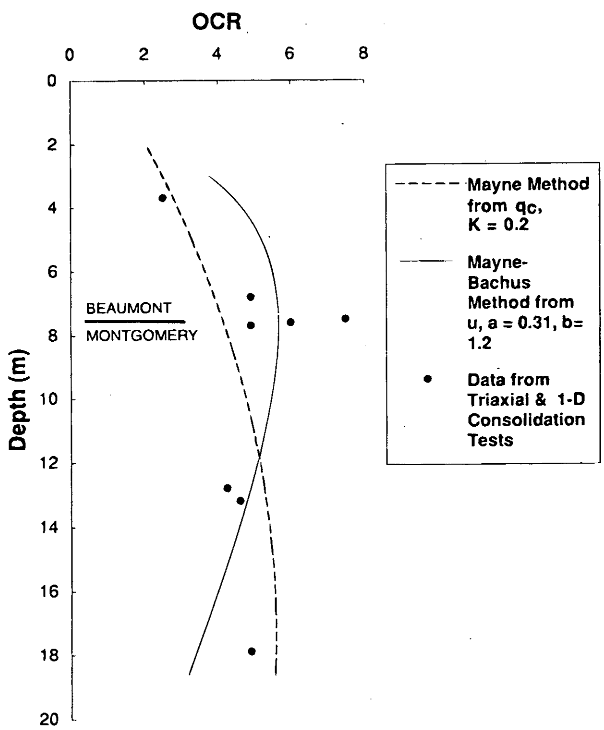

FIGURE 13 OCR versus depth at Site A: Mayne method, Mayne-Bachus method, and laboratory measurements.

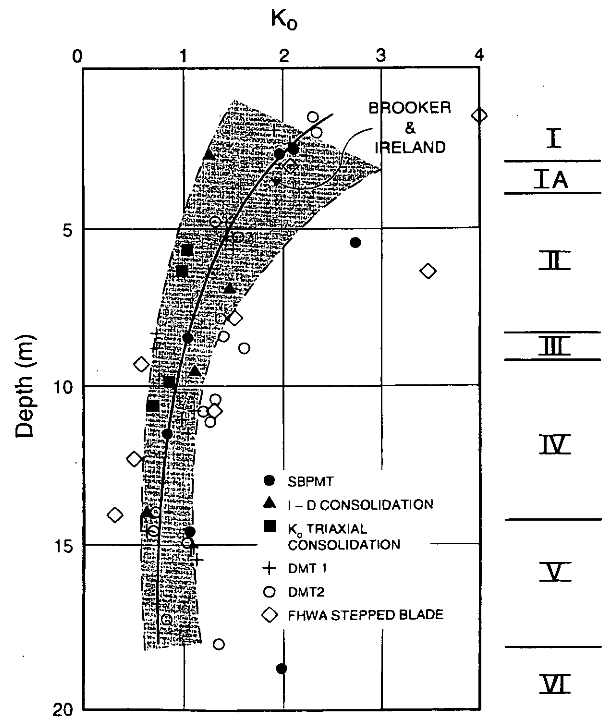

FIGURE 14 K_0 versus depth for Site A.

YOUNG'S MODULUS

Young's modulus (E) for undrained loading has been determined as a function of depth at Site A by several methods, as shown in Figure 15. For the DMT tests, E was taken to be equal to the dilatometer modulus E_d, assuming Poisson's ratio of the soil (ν) to be 0.33 (*10*). For the crosshole tests, the velocities of vertically polarized s-waves were measured to obtain shear modulus, which was converted to E by assuming ν to be 0.45. A similar value was used in converting pressuremeter modulus to E. In the triaxial tests, E was evaluated as the secant modulus at a principal stress difference equal to one-fifth of the peak principal stress difference. The various methods are generally consistent, except for the crosshole method, which gives values that are almost an order of magnitude higher than the other methods, as expected because of the small strains associated with crosshole testing.

E values determined from UU triaxial compression tests were highly erratic and are therefore not shown in Figure 15. By comparing a linear least squares fit of the values from Figure 15 (except crosshole) with the cone-generated s_u relation in Figure 5, one obtains

$$E/s_u = 206 + 1.4\,z\text{ (m)} \qquad z < 20\text{ m} \tag{4}$$

for Site A, where z is depth below the ground surface. Williams and Focht (*13*) have back-calculated from short-term settlement observations on 15 large raft-supported structures within the Houston area, in which the raft depth was at or below the Beaumont-Montgomery contact. Where the soil profile was predominantly clay, they found that

$$100 + 32\,z'\text{ (m)} < E/s_u < 200 + 40\,z'\text{ (m)} \tag{5}$$

where z' is depth below the raft (mean raft depth of about 10 m below the surface). The lower z-intercept values in Equation 5 may be due to effects of excavation, and the higher gradient with depth may be due to decreasing strain levels below the raft foundations, whereas in the in situ tests (Equation 4), relevant strain values remain approximately constant with depth.

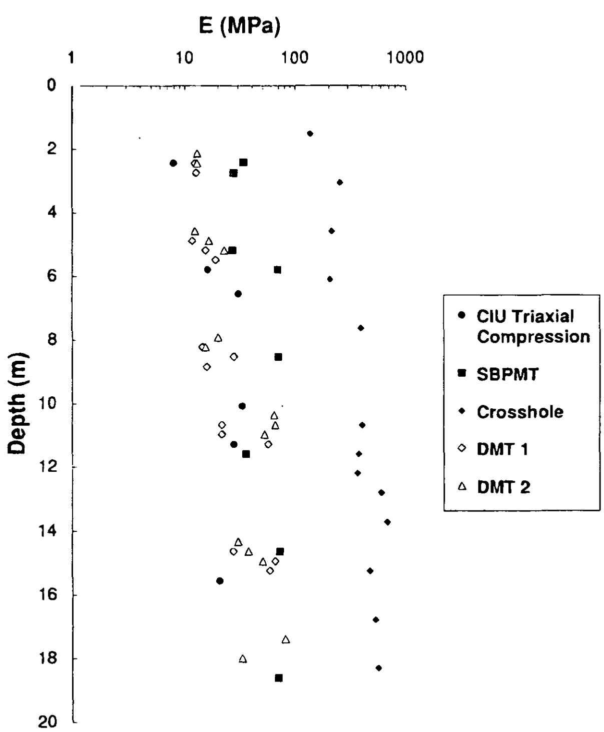

FIGURE 15 First-cycle Young's modulus versus depth, Site A.

CONCLUSIONS

The Beaumont-Montgomery sequence was deposited in a deltaic environment and was preconsolidated by desiccation. As such, the properties are complex and variable. Nonetheless, relatively clear estimations of mean s_u, OCR, K_0, and E are possible with sufficient investigation at a given site. However, mean properties change from site to site depending on the location of the site relative to ancient distributary channels. The most comprehensively studied property, OCR, is in the range of 3 to 7 below the piezometric surface at Site A, a pro-delta site within the upper (Beaumont) formation. Site B, a backswamp site in the upper formation several kilometers from Site A, indicated that values of OCR are two to three times as high. Corresponding average values of s_u from CPT records (Figure 5) are about 0.09 MPa in the upper formation at Site A and 0.11 MPa in the upper formation at Site B. Use of s_u to characterize shear strength in the lower (Montgomery) formation may not be entirely appropriate for most applications because of its sandy nature; however, s_u values from both q_c correlations and SHANSEP tests trend higher than those in the upper formation, approximately 0.1 to 0.2 MPa, although the OCR tends to be less in the lower formation.

Preliminary evidence indicates that CPTU profiling can be used to establish the OCR profile in the formations considered here, either with q_c readings or u readings for a cone with the piezo element on the tip. Piezocones can also possibly be used to identify stratum changes, but it is not clear whether the tip or sleeve piezo element is more appropriate for that task.

ACKNOWLEDGMENTS

Numerous agencies provided assistance to produce and provide the data reported here. They include Fugro Geosciences, Inc., McBride-Ratcliff and Associates, Southwestern Laboratories, Inc., University of Houston, University of Texas, Iowa State University, and Louisiana State University. Most of the data at the NGES (Site A) were acquired under several grants and contracts from the National Science Foundation and FHWA from 1979 to 1993.

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Characterization of Preconsolidated Soils in Richmond, Virginia

RAY E. MARTIN, EDWARD G. DRAHOS, AND JOHN L. PAPPAS

The Miocene-age Calvert formation underlying Richmond is highly preconsolidated and very sensitive and requires careful evaluation for foundation design. This soil is of marine origin, and preconsolidation results from desiccation associated with several identifiable drying surfaces and overburden erosion. Major structures are typically supported within this formation by spread footings and belled caissons. The high undrained shear strength and preconsolidation pressure of the formation allow the design of high-capacity foundations. The standard penetration test N-values for the soil are typically low and not indicative of its quality. Conventional triaxial compression and consolidation tests are often utilized to obtain parameters for design of foundations. More recently pressuremeter test results have been used for foundation design. The purpose of this paper is to present a summary of available laboratory and pressuremeter test data and to characterize the engineering properties of this soil.

The stratigraphy of the Richmond area is of interest because one of the units underlying the city, the Miocene-age Calvert formation, is very sensitive and highly preconsolidated. About 30 years ago, Arthur and Leo Casagrande identified the characteristics of this unit while providing consulting foundation engineering services for several major structures in Richmond, including City Hall. Leo Casagrande (*2*) compiled their findings in a paper that became the basis for future research concerning the soil. The techniques used by the Casagrandes to identify the characteristics of the soil were classification, consolidation, and unconsolidated undrained triaxial compression tests. However, engineers at that time were relying on standard penetration test (SPT) *N*-values to determine soil properties for foundation design. The *N*-values are typically very low, ranging from about 4 to 20 for this highly preconsolidated formation where overconsolidation ratios can approach 4 because of the sensitivity of the soil, which varies from about 10 to 22. Based on *N*-values, the soil was considered to be too soft and compressible for support of major structures. Prior to this time most major structures in Richmond were founded on piles driven through this stratum or straight shaft caissons supported on rock as much as 150 to 175 ft (46 to 53 m) below the ground surface. Most recent structures have been supported on single and double under-reamed caissons founded in the clay or spread footings supported on the surface of the clay. These designs have resulted in significant foundation cost savings (*7*).

GEOLOGY

Richmond is located on the James River at the fall line, about 100 mile from the Atlantic Ocean. In the downtown area, ground surface varies from about sea level at the James River to about El 170 (52 m) to 180 (55 m) in the highest areas along Broad and Marshall streets. This upper portion of downtown Richmond is the area considered in this study (Figure 1).

Precambrian bedrock consisting of Petersburg granite underlies the downtown area and is typically at about El 30 (9 m) to −10 (−3 m). Overlying bedrock is a sequence of weathered residual soil and coastal plain sediments. The residual soil is very compact and is described as disintegrated rock. The overlying soils were deposited during various transgressions and regressions of the sea and include the very compact Eocene sand and Cretaceous sand and gravel. The Calvert formation overlies the Eocene soils and typically extends from El 60 (18 m) to about El 140 (43 m) in the upper portion of the downtown area and thus is about 80 ft (24 m) thick. Overlying the Calvert is a stratified Pleistocene terrace deposit consisting of sand, gravel, and clay layers. The clays in this formation are somewhat preconsolidated due to desiccation. The sand and gravel layers are usually compact. A typical boring log and geologic profile along Marshall Street illustrating this geologic sequence are included as Figures 2 and 3.

The preconsolidated nature of the Calvert formation likely resulted from (*a*) desiccation during periods of emergence as the Miocene sea level fluctuated and (*b*) erosion of overlying sediments during post Miocene time. The Miocene sea is estimated to have risen to maximum El 240 (73 m) (*5*). Approximately 2 mi (3.2 km) to the north of the downtown area and in a position parallel to the old Miocene shoreline, the Calvert formation has been observed at El 180 (55 m) or about 40 ft (12 m) above the top of the formation in the downtown area. Terrace deposits in the area rise to about El 230 (70 m) to 240 (73 m). Based upon this geologic evidence, it is possible to estimate that the maximum previous ground surface in the downtown area may have been as high as about El 230 (70 m).

Groundwater is located in the Cretaceous deposit at about El 40 (12 m) and is hydraulically connected to the James River. Thus, the full column of soil above this level is effective. A perched water condition is often present above the Miocene formation, and its elevation is dependent upon the amount of precipitation and level of the top of the formation.

RECENT INVESTIGATIONS

Over the intervening years since the Casagrandes' work, additional laboratory and in situ testing has been performed on this soil during investigations for numerous structures (*1,3,4,13*). In addition, the pressuremeter has been used to further characterize the soil. In 1986 Martin and Drahos (*8*) published a paper describing more recent laboratory testing data and included correlations between Menard pressuremeter (MPM) and triaxial and consolidation test results. Specifically, the paper established undrained shear strength

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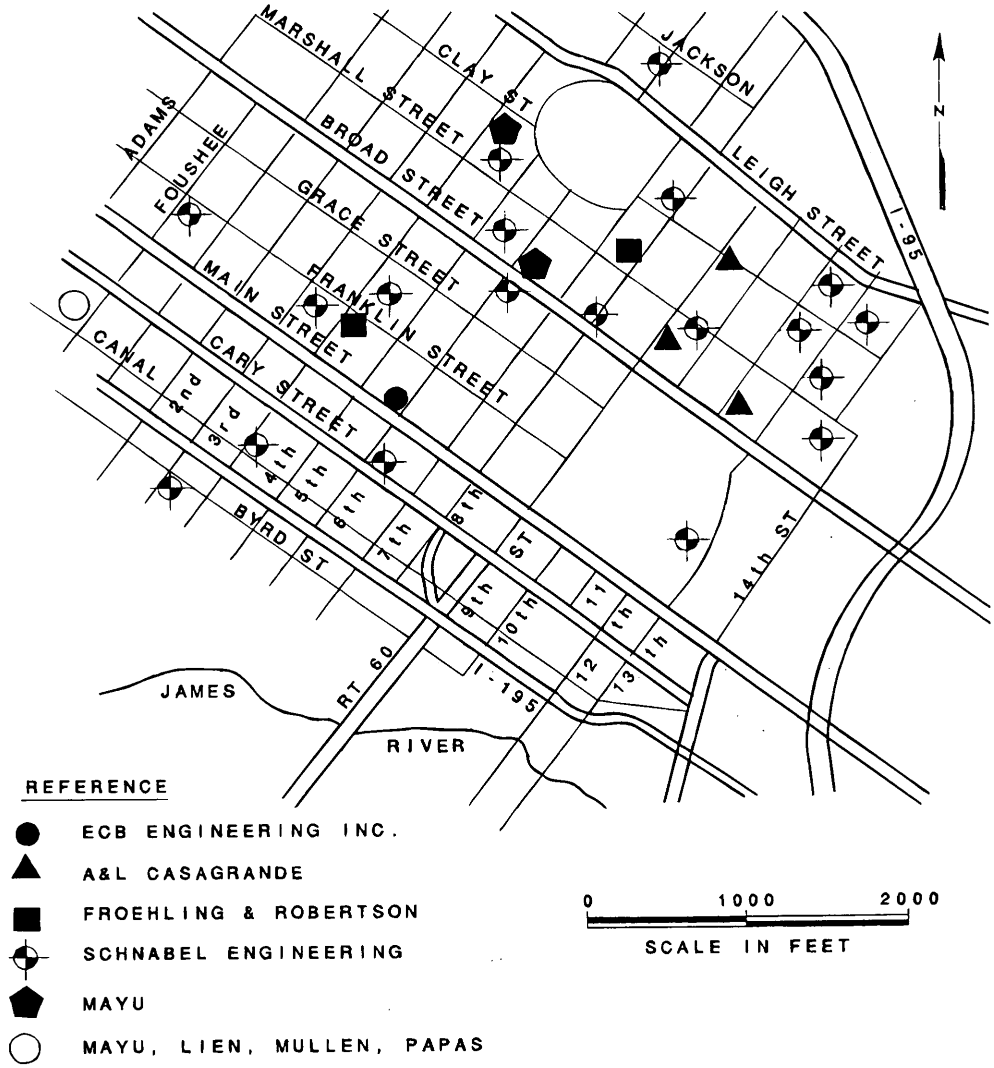

FIGURE 1 Map of upper portion of downtown Richmond (1 ft = 0.305 m).

from the pressuremeter limit pressure based on correlations with unconsolidated-undrained triaxial compression tests.

Mayu (*10*), Mullen (*11*), Lien (*6*), and Pappas (*12*) have also performed additional pressuremeter tests in the soil. They each used the self-boring pressuremeter (SBPM) and the MPM to evaluate soil modulus, undrained shear strength, and the earth pressure coefficient at rest, K_0. Mullen also performed tests with the dilatometer and cone penetrometer. These data from in situ tests suggest the undrained shear strength and soil modulus to be higher than that obtained by conventional laboratory testing of undisturbed tube samples.

The clays in the formation tend to be highly plastic. Mayu (*10*) evaluated the clay mineralogy of one sample of the clay and found the following constituents:

Constituent	*Percent*
Mica	25
Kaolinite	10
Smectite	55
Phyllosicates	9
Quartz	1

The high percentage of smectite, which includes montmorillonite, provides the characteristic high plasticity of these soils.

SAMPLE DATA BASE

This study summarizes the results from over 200 samples tested in the laboratory from undisturbed tube and block samples and over 70 pressuremeter tests. These data represent 24 sites in downtown Richmond as shown in Figure 1. The data were developed from the references listed in the previous section. Most of the laboratory test results were obtained from 3-in. diameter tube samples, which compose about 80 percent of the samples. Both shear strength and consolidation test results are included for these samples. About 20 percent of the tube samples were 2-in. in diameter and were used to perform shear strength testing. Shear strength and consolidation results from four 5-in. diameter tube samples and three block samples are also included.

SAMPLE QUALITY

One issue of concern when estimating shear strength and compression properties of stiff and hard consistency clays from laboratory tests is the disturbance that occurs during both sampling and sample

ELEV (FT)	STRATA DESCRIPTION	SPT	REMARKS
	GROUND SURFACE EL 150 FT		
141	SAND FILL, SOME BRICK & CINDERS - BROWN & BLACK	7 7	FILL
132	WELL GRADED SAND WITH CLAY - BROWN (SC)	54 18 12	PLEISTOCENE ▼ (PERCHED)
121	SILT - BROWN (ML)	6 6 9	UPPER MIOCENE (CALVERT FORMATION)
60	FAT CLAY WITH SAND - GRAY (CH)	12 7 14 12 16 16 20 11 17 12 13 14 13 13 89	LOWER MIOCENE (CALVERT FORMATION)
38	SILTY SAND WITH CEMENTED SAND LAYERS - BROWN & GRAY (SM)	21 100/0.7 100/0.8 38	EOCENE ▼
12	SILTY SAND WITH GRAVEL - GREEN (SM)	100/0.3 100/0.6 91 93 100/0.3	CRETACEOUS
-10	DISINTEGRATED ROCK - GRAY	100/0.8 100/0.7 100/0.3 100/0	PRECAMBRIAN
-15	HARD SLIGHTLY FRACTURED GRANITE ROCK	REC 100%	

▼ = WATER TABLR
SPT = STANDARD PENETRATION TEST
REC = RECOVERY
100/0.3 = SPT BLOWS/FT

FIGURE 2 Typical test boring (1 ft = 0.305 m).

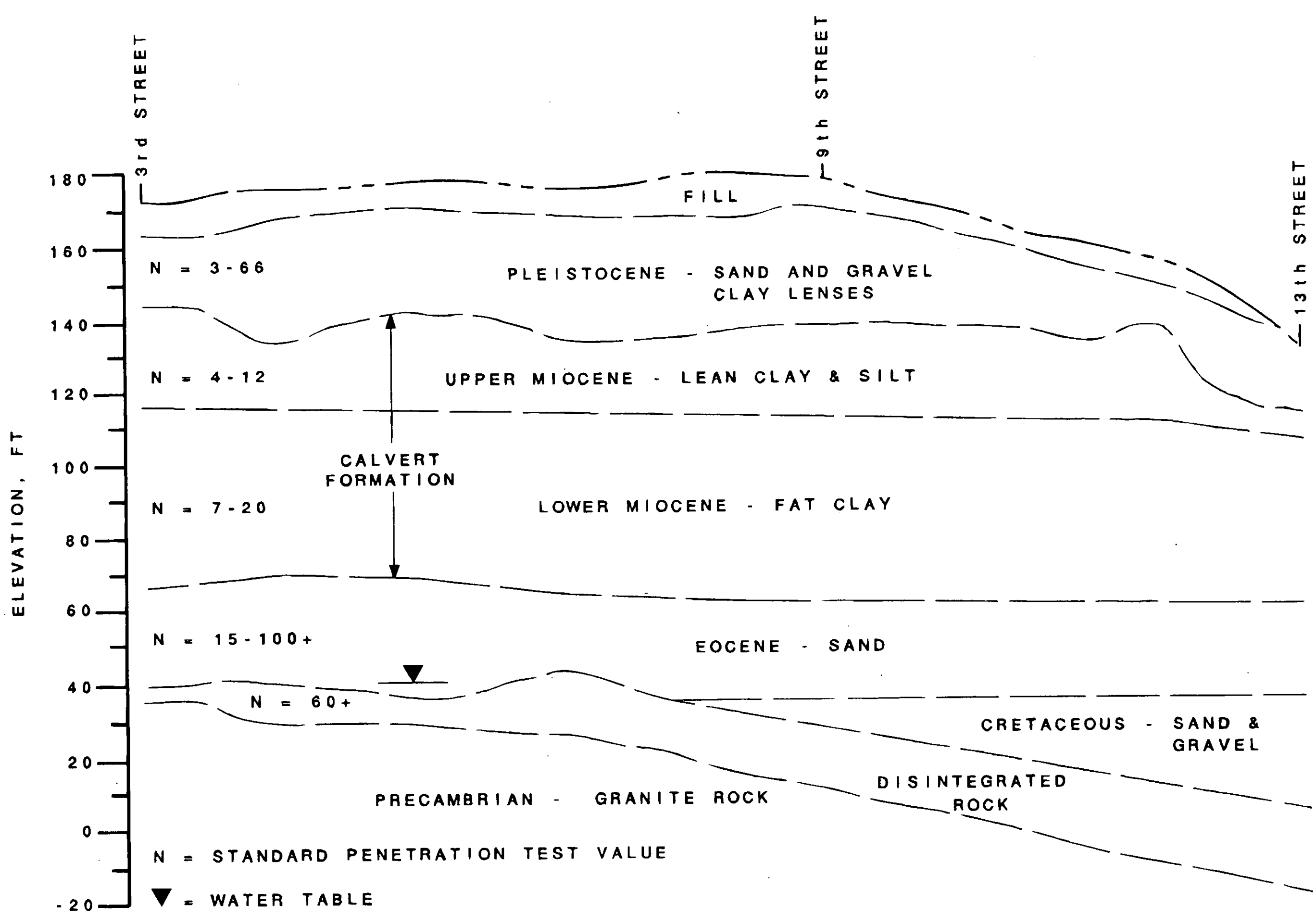

FIGURE 3 Profile along Marshall Street (1 ft = 0.305 m).

extrusion in the laboratory. Block samples would be expected to experience less disturbance than 5-in. diameter tube samples and the 5-in. and 3-in. diameter tube samples likewise should experience less disturbance than the smaller 2-in. diameter samples. The area ratio A_r may be used to estimate sample disturbance and is defined as follows:

$$A_r(\%) = [(D_o^2 - D_i^2)/D_i^2] \times 100$$

where D_o is the outside diameter of the sample and D_i is the inside diameter of the sample.

The term "undisturbed" is generally used for a sample obtained with a sampling tube for which the area ratio is equal to or less than 10 percent. The area ratios for the types of samples tested are as follows:

Sample Type	*Area Ratio (%)*
2-in. tube	13.7
3-in. tube	8.9
5-in. tube	5.2
Block	~0

The 2-in. sample exceeds the 10 percent limit. The block samples are designated as approximately zero since they are obtained by cutting from the ground. From the area ratios it is obvious that the block sample should be far superior to the 3-in. or 5-in. tube samples. Less-disturbed samples should produce higher-quality results when tested for shear strength and consolidation properties. The preconsolidation pressure and compression index would be expected to be higher for comparable samples that have experienced less disturbance. Likewise the shear strength would be expected to be higher and strain at failure lower under less-disturbed conditions. Only three block samples and four 5-in. diameter tube samples are included in the data base. These samples are identified in the test results presented below for comparison with results for 2-in. and 3-in. diameter tube samples.

The pressuremeter test has the advantage of being performed in situ, and the soils surrounding the test should be less affected by disturbance. The SBPM should also subject the soils to less disturbance than the MPM, since the two-step process of excavating the hole and replacement with the MPM is not required. This is also true for the dilatometer and cone penetrometer tests.

LABORATORY TESTING

The data base includes index property, consolidation, undrained shear strength, and soil modulus test data. The data are discussed under these topic headings below for clarity.

Index Properties

Index property tests for purposes of this paper include moisture content, fines content (percentage passing the No. 200 sieve), plasticity index, and density. Wet density is used in overburden calculations since the full column of soil is effective. Dry density is more meaningful with respect to strength and compressibility properties.

The Miocene-age soils were previously divided into two strata (*2*). The upper Miocene extends from about El 140 (43 m) to 120 (37 m). The lower Miocene extends below this level to about El 60 (18 m). All boundaries vary up to about ±10 ft (3 m) across the area included in the study. The results of these tests are plotted in Figures 4–6.

The moisture content of the upper Miocene typically ranges from about 30 to 40 percent as shown in Figure 4. The range for the lower Miocene is about 40 to 60 percent between about El 120 (37 m) and 100 (30 m). Below this level the moisture content ranges much more widely from about 40 to 90 percent. Thin beds of sand occur occasionally in the lower Miocene. Moisture contents are typically lower in these thin layers, approximating the values of the upper Miocene. The average values for each of these layers are 34.9 percent for the upper Miocene, 48.6 percent for the upper portion of the lower Miocene above about El 100, and 61.4 percent for the remainder of the lower Miocene as shown by the vertical lines on Figure 4.

The upper Miocene has a much lower fines content, typically ranging from about 30 to 60 percent. The fines content in the lower Miocene ranges from about 70 to 100 percent, but the majority of samples have more than 85 percent fines with no change indicated at El 100 (30 m). The average for the upper Miocene is 45.1 percent and for the lower Miocene, 91.2 percent.

Dry and wet densities are also distinctly different as would be expected based on the variation in moisture and sand content. Dry density values range from about 80 to 100 pcf (1281 to 1602 kg/m^3) for the upper Miocene and average 85.5 pcf (1370 kg/m^3). The lower Miocene ranges from 50 to 80 pcf (801 to 1281 kg/m^3) with a distinct change in density at about El 100 (30 m). Above this level the values range from 60 to 80 pcf (961 to 1281 kg/m^3) and average 73.2 pcf (1173 kg/m^3). Below this level the dry density drops to a range of about 50 to 70 pcf (801 to 1121 kg/m^3) with an average of 61.4 pcf (984 kg/m^3). Wet density values illustrate a similar variation.

The liquid limit and plasticity index are also distinctly different in the two strata. The liquid limit and plasticity index are much higher in the lower Miocene. Typical liquid limits range from about 30 to 60 in the more sandy upper Miocene and 50 to 110 for the lower Miocene. Plasticity index values range from about 10 to 40 for the upper Miocene and 30 to 70 for the lower Miocene.

The samples in the upper Miocene typically classify clayey sand (SC) to sandy lean clay (CL) as shown in Figure 5. The lower Miocene typically classifies as fat clay (CH). Casagrande (*2*) noted that these soils contain colloidal organic matter and that samples should be air-dried prior to Atterberg limit testing. Oven drying of samples results in a reduction in liquid limit and thus the plasticity index. The samples that plot below the A-line in Figure 5 are probably not representative of the strata properties. These results are likely due to improper testing procedures.

These index properties confirm that the Miocene formation should be separated into two strata, described above as upper and lower, with the boundary at about EL 120 (37 m). In addition, these data suggest that consideration should be given to further dividing the lower Miocene at about El 100 (30 m). This concept is supported by the data discussed below.

Consolidation Test Data

Consolidation tests are widely used to evaluate the deformation characteristics of these soils. The soils generally exhibit preconsolidation pressures (P_c) well in excess of the existing overburden pressures (P_o) as noted in Figure 6.

The ground surface grades at the locations of the borings where samples were obtained vary from about El 140 (43 m) to 180 (55 m). The overburden pressures for all samples were normalized for evaluation purposes using a ground surface grade of El 180 (55 m), the typical ground surface grade in the upper portion of downtown Rich-

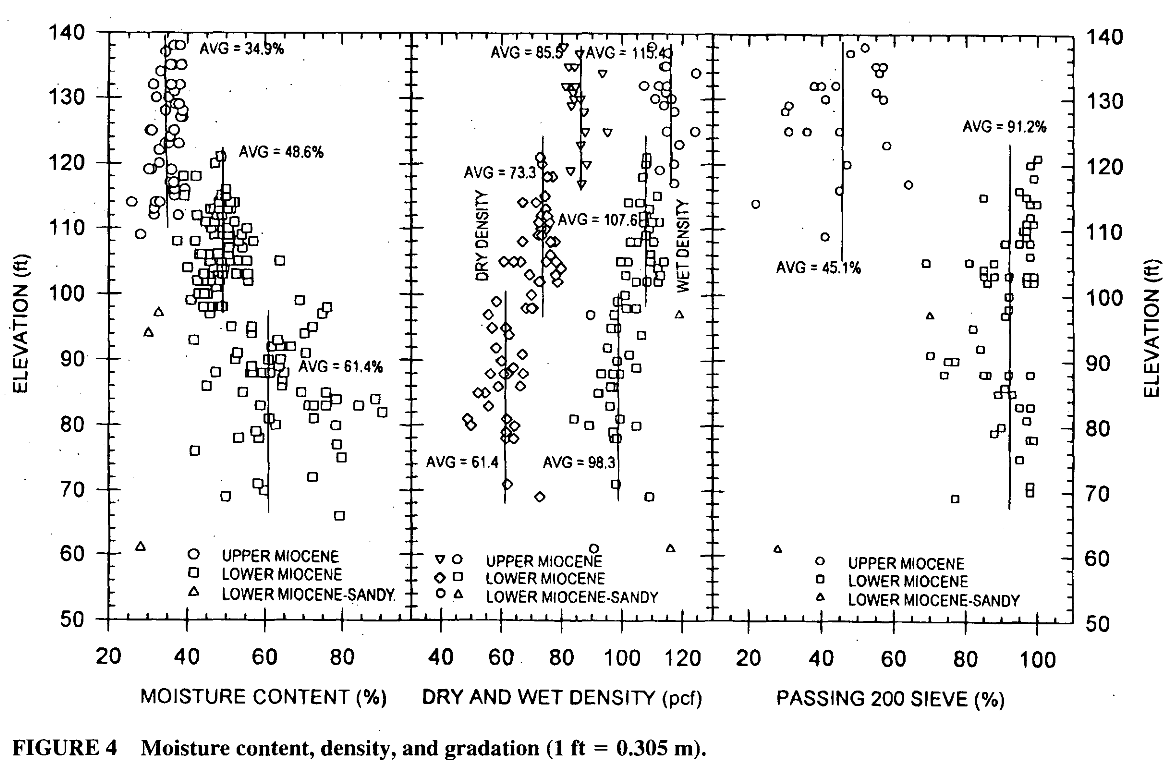

FIGURE 4 Moisture content, density, and gradation (1 ft = 0.305 m).

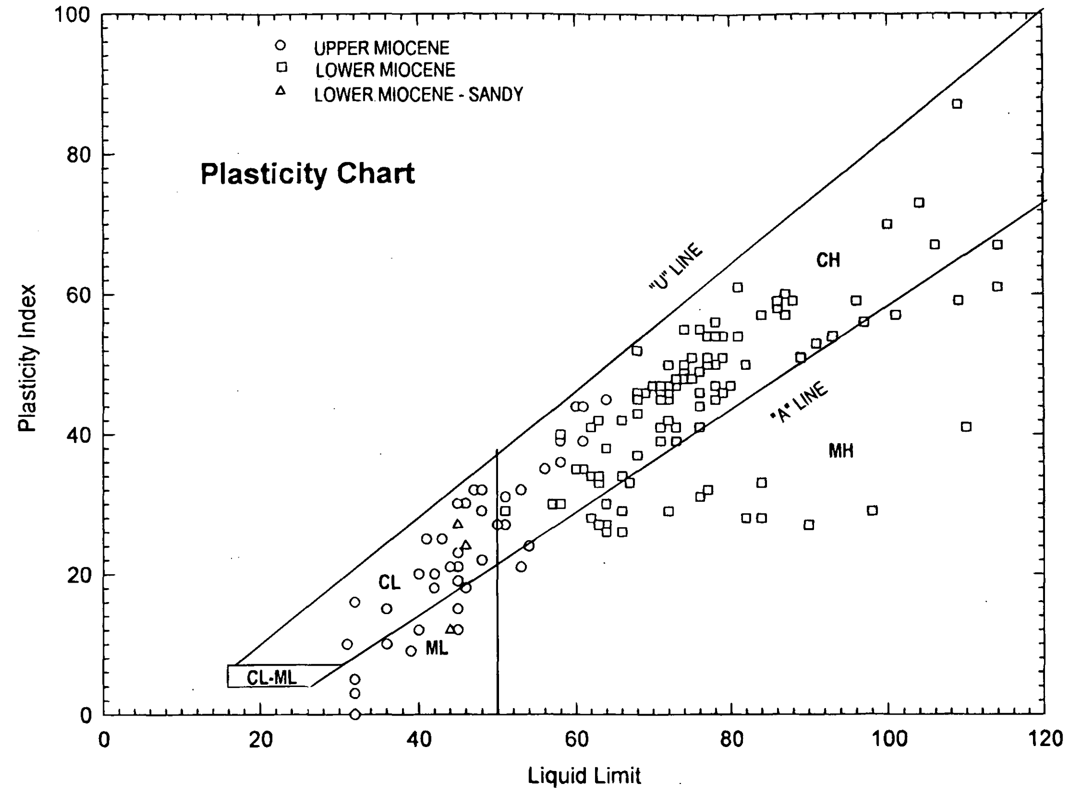

FIGURE 5 Atterberg limits (1 ft = 0.305 m).

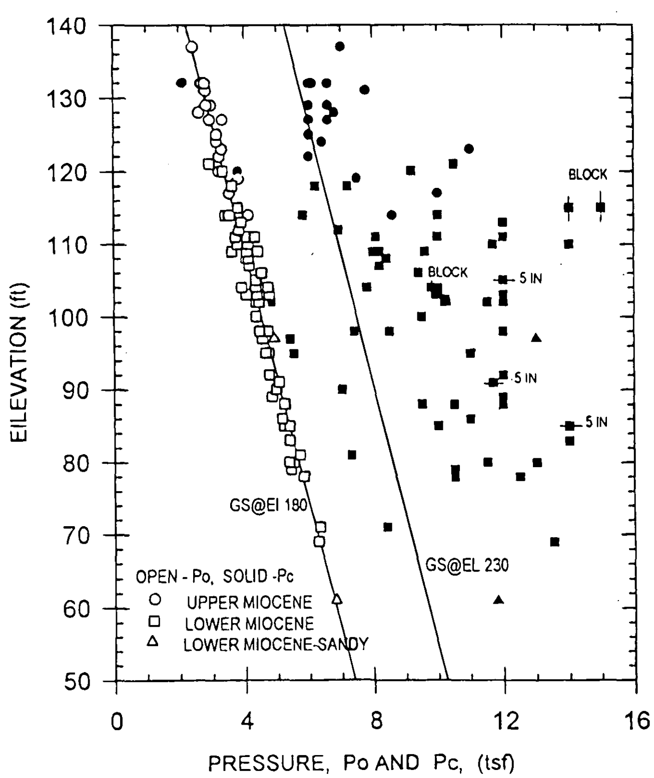

FIGURE 6 Consolidation test data (1 ft = 0.305 m, 1 tsf = 0.1 MN/m²).

mond. This normalized overburden pressure was also used to calculate the normalized overconsolidation ratio (OCR) for all the sites in the downtown area as shown in Figure 6. This approach allows the comparison of other data from all sites with the normalized OCR.

The maximum ground surface elevation in this part of Richmond in past geologic history was likely about El 230 (70 m) as discussed above. Note that the maximum past pressures for the vast majority of samples exceed the El 230 (70 m) line thus indicating the appropriateness of this assumption. Two possible explanations for why the preconsolidation pressures plot to the left of the El 230 line are (*a*) that the samples tested were disturbed or (*b*) that the ground surface may not have been as high as assumed.

The wide variation of the maximum past pressures is believed to be due to old surfaces of drying that occurred during the regressions of the Miocene sea. This effect is most noticeable in the more plastic soils with higher fines content of the lower Miocene, as would be expected, since they are more susceptible to the effects of surface tension.

The normalized OCR data are illustrated in Figure 7. These data indicate that the strata between about El 100 (30 m) and 120 (37 m) in the lower Miocene have higher normalized OCR values than the strata below El 100 (30 m). This suggests that the strata should be suitable for support of somewhat higher foundation bearing pressures than the remaining portion of the lower Miocene below El 100 (30 m). This also supports the idea that the lower Miocene should be separated into two distinct strata.

The initial void ratios are also presented in Figure 7. The initial void ratios are somewhat higher for the lower Miocene than the

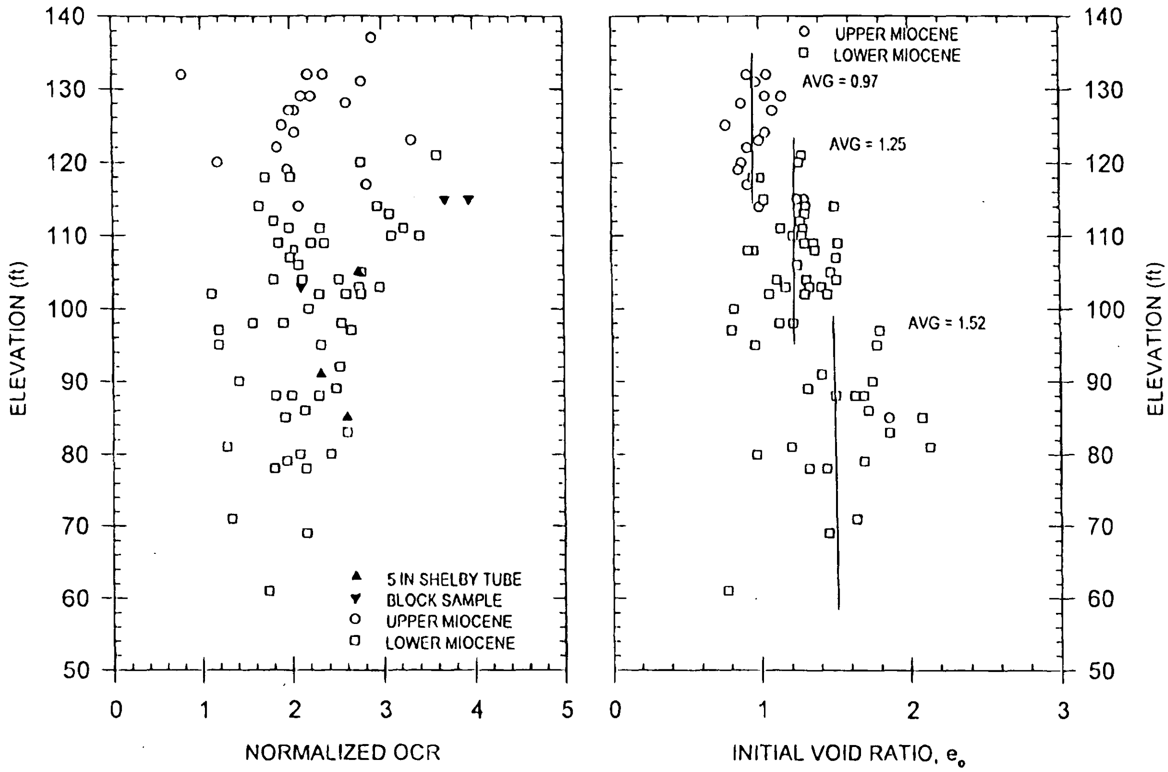

FIGURE 7 Normalized OCR and initial void ratio (1 ft = 0.305 m).

upper Miocene. The portion of the lower Miocene between about El 100 (30 m) and 120 (37 m) exhibits somewhat lower initial void ratios as would be expected based on the data presented above including the higher density and normalized OCR values associated with this stratum. The average e_0 value is 0.97 for the upper Miocene, 1.25 for the lower Miocene above about El 100, and 1.52 for the remainder of the lower Miocene. The average values for the three zones are shown by the vertical lines on Figure 7.

The variation of the compression (C_c) and recompression (C_r) indices with liquid limit and void ratio are illustrated in Figure 8. The equations for the linear regression trend lines are included in the figures and are as follows.

Compression Index

$C_c = 0.0326(LL - 43.4) \qquad C_c = 1.79(e_0 - 0.808)$

Recompression Index

$C_r = 0.00045(LL + 11.9) \qquad C_r = 0.05(e_0 - 0.444)$

These equations may be used for preliminary estimates of C_c and C_r in lieu of actual test data. The ratio C_r/C_c based on these equations is about 1/25 or very high. Typical values range from about 1/5 to 1/10.

Results for the 5-in. tube and block samples are noted in Figures 6–8. The estimate of the preconsolidation pressure would be expected to improve with reduction in sample disturbance. Direct comparisons cannot be made, since samples were not obtained side by side but, rather continuously, down the hole. However, five of the six block and 5-in. tube samples tested are in the highest range of preconsolidation pressures and normalized OCR values, suggesting that these samples are likely to be less disturbed. All 5-in. diameter tube and block samples were obtained in the lower Miocene formation.

Figure 8 illustrates the position of the six 5-in. and bulk samples tested versus all samples. The C_c values would be expected to be higher for less disturbed samples with similar properties, but the values for these samples do not illustrate this trend. The recompression index should be little affected by sample disturbance since these values were calculated from unload-reload cycles.

Soil Modulus

Soil modulus data were developed from both triaxial and pressuremeter tests. The triaxial test data represent tangent modulus values measured at 50 percent of peak stress and are shown in Figure 9. Because the triaxial test samples from undisturbed tube samples undergo more disturbance and test smaller amounts of soil, the values are typically lower than pressuremeter values, ranging from 100 to 400 tsf (10 to 40 MN/m^2). The 5-in. diameter tube sample tests are generally at the upper edge of the unconsolidated, undrained (UU) and consolidated, undrained (CU) data from undisturbed tube samples. Triaxial testing done on the block samples that have experienced less disturbance recorded higher results, which ranged from 600 to 1000 tsf (60 to 100 MN/m^2).

The MPM is lowered into a preformed hole where the soil has slight to moderate disturbance caused by the drilling process and insertion techniques. These tests were slightly higher than the triaxial tests and ranged from 100 tsf (10 MN/m^2) to approximately 1300 tsf (130 MN/m^2). The highest modulus values were calculated from the

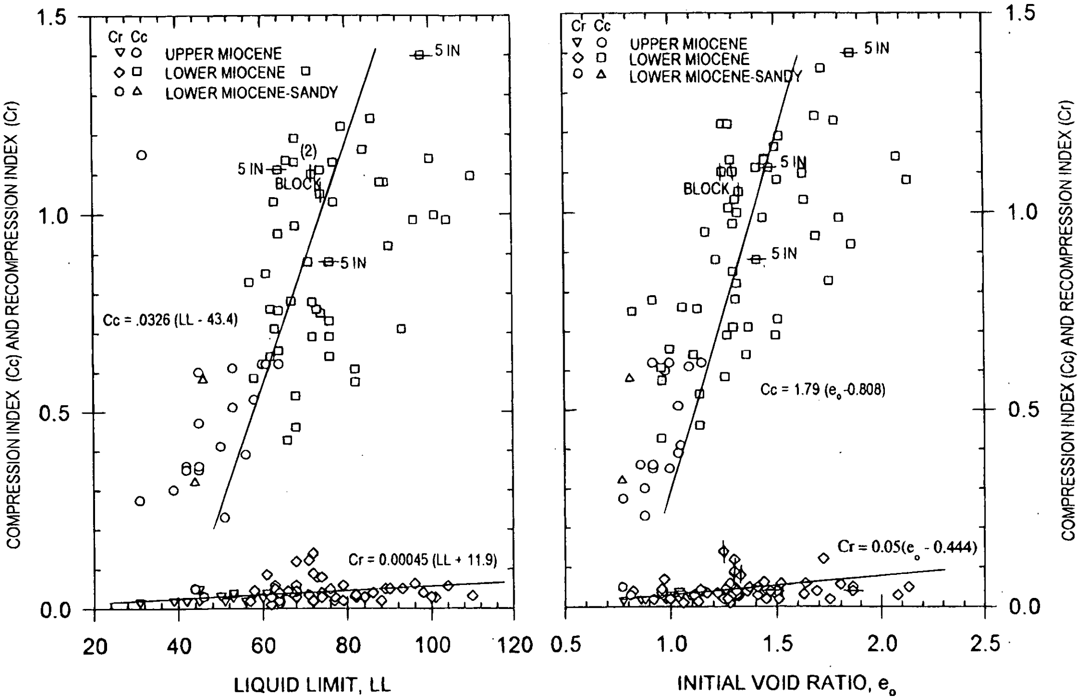

FIGURE 8 Variation of C_c and C_r with liquid limit and initial void ratio.

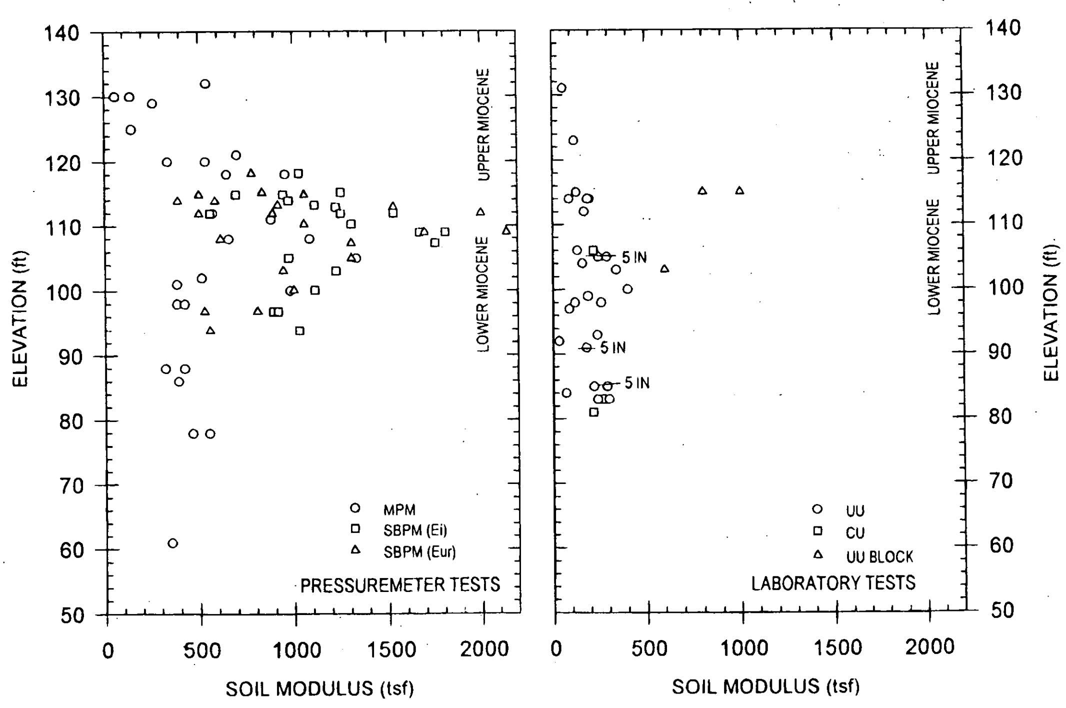

FIGURE 9 Soil modulus versus elevation (1 ft = 0.305 m, 1 tsf = 0.1 MN/m²).

SBPM, which causes the least disturbance to the soil as it drills its own hole to the level being tested. The probe membrane expands pneumatically and uses three strain arms 120 degrees apart in the middle of the probe to measure the membrane displacement during expansion. The SBPM tests provide an initial modulus value (E_i) from a tangent to the steepest portion of the initial loading curve and a value from subsequent unload reload curves (E_{ur}). The SBPM tests produced the highest initial modulus values, ranging from approximately 500 tsf (50 MN/m^2) to peak values near 1800 tsf (180 MN/m^2).

It should be noted that the highest soil modulus values were found above El 100, again confirming the significance of this upper portion of the lower Miocene.

Shear Strength

The majority of the laboratory undrained shear strength, s_u, test data represent UU triaxial compression and unconfined compression (Q) tests as indicated in Figure 10. The UU tests were performed at or near a confining stress equal to the overburden pressure under the assumption that K_0 equals about one. K_0 values have been shown to range from 1 to 2 for the upper Miocene and 2 to 8 for the lower Miocene on the basis of SBPM tests (*6*). These are very high values. Equations for estimating K_0 from OCR data, such as those developed by Mayne and Kulhany (*9*), suggest that K_0 would be in the range of about 0.75 to 1.25. The use of $K_0 = 1$ for the UU test appears conservative based on the SBPM tests, and the resulting s_u values should be lower than those from the in situ tests.

The typical values for s_u in the upper Miocene range between about 0.5 and 1.0 tsf (0.05 and 0.1 MN/m^2) with the triaxial and in situ test results in the same range. The values for the lower Miocene range from about 1.0 to 7.0 tsf (0.1 to 0.7 MN/m^2). The in situ tests are typically higher than the triaxial results. Once again the results are higher in the lower Miocene above El 100 (30 m). The very high s_u values would not be expected based on SPT N values. The usual s_u correlations with N-values would suggest s_u values of between 0.5 and 1.0 tsf and (0.05 and 0.1 MN/m^2) as opposed to 1.0 to 7.0 tsf (0.1 to 0.7 MN/m^2).

The block and 5-in. diameter tube samples are in the upper range of laboratory s_u values and these are indicative of less disturbance. This is not true for the MPM because of the built-in bias by using correlations with triaxial test results to estimate S_u as previously described (*8*).

The s_u values also increase with reduced strain at failure but with much scatter as illustrated in Figure 11. Typically failure strain values are less than 6 percent for the lower Miocene formation. Block samples produced the highest s_u values with s_u = 5 tsf (0.5 MN/m^2) at 1 percent strain at failure.

SUMMARY AND CONCLUSIONS

The Miocene-age formation in downtown Richmond is unique in that it is both highly preconsolidated and sensitive but not fissured. Standard penetration test N values typically underestimate the soil quality, and laboratory or in situ testing are required for accurate assessment of the soil properties. The data base presented herein includes results from a variety of sources including tests from over 200 undisturbed samples and over 70 pressuremeter tests. The reader is referred to the various figures for specific ranges of data for various soil properties. The conclusions may be summarized as follows.

1. Previous studies identified upper and lower Miocene layers based on gradational properties and this is confirmed. The upper layer, which typically extends from about El 120 to 140 (37 to 43 m), is more sandy and varies from clayey sand (SC) to sandy lean clay (CL). The lower layer from about El 60 to 120 (20 to 37 m) has a high fines content and usually classifies as fat clay (CH).
2. The test data suggest that a third layer consistently exists at the top of the lower Miocene layer between about El 100 and 120 (30 and 37 m). This layer was previously identified as an old drying surface at specific sites. The layer typically has higher normalized OCR values, lower initial void ratios, and higher shear strength and soil modulus values than the soils below or above.
3. In situ testing with the MPM and the SBPM generally provide higher soil modulus and undrained shear strength values than conventional triaxial testing of undisturbed samples for both the upper and lower Miocene formations. Only the block samples produced results similar to the in situ test results.

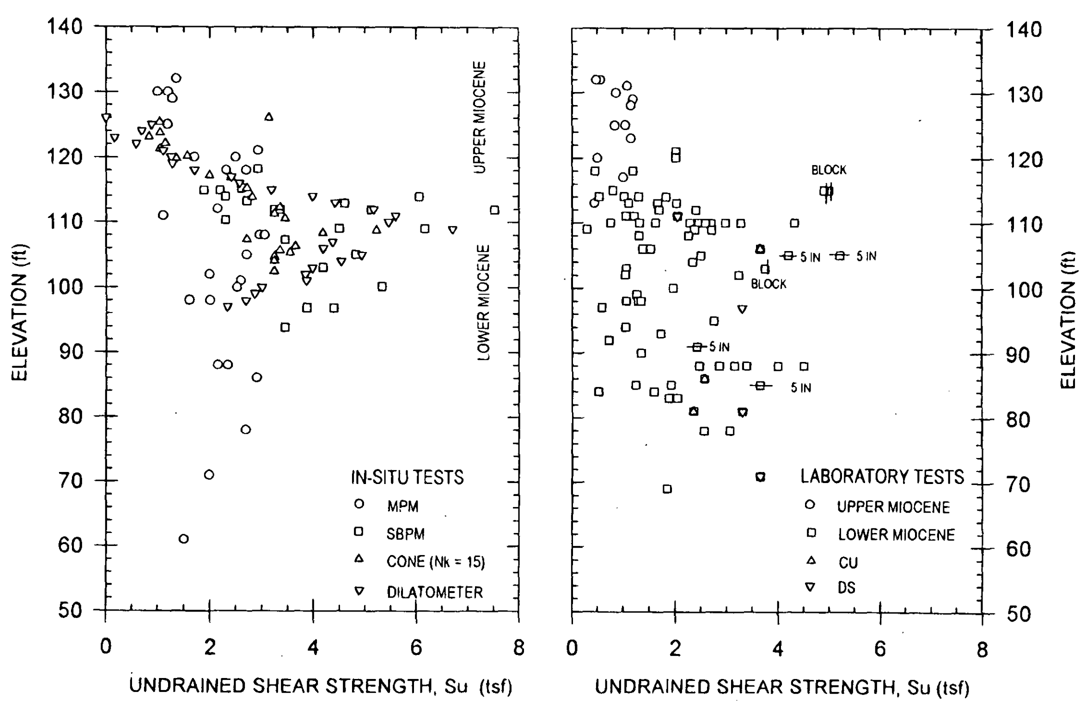

FIGURE 10 Undrained shear strength versus elevation (1 ft = 0.305 m, 1 tsf = 95.8 kN/m^2).

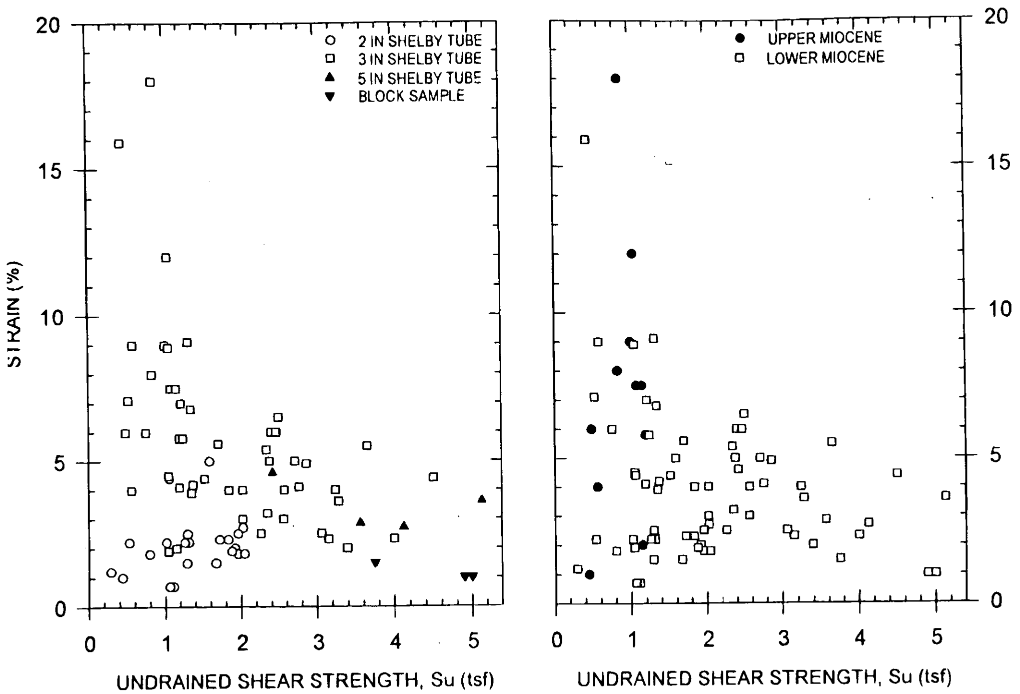

FIGURE 11 Undrained shear strength versus strain at failure (1 tsf = 95.8 kN/m²).

4. Sample disturbance does have a major impact on laboratory test results for both undrained shear strength and compression properties. Based on the area ratio the block samples and 5-in. diameter tube samples should be the least disturbed and in fact generally do produce higher-quality results than smaller-diameter tube samples. Only the block samples produced results similar to the in situ test results for undrained shear strength and soil modulus. The block samples also produced the highest estimates of preconsolidation pressure.

5. Equations for C_c and C_r in terms of liquid limit and initial void ratio are presented. Due to the scatter in the data, these equations should only be used for preliminary estimates of these indices.

ACKNOWLEDGMENTS

The authors gratefully acknowledge Schnabel Engineering Associates, Inc., for sponsoring the project through their in-house research and development program and Mary Ann Millikin for her tireless work during preparation of the manuscript.

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Overconsolidated Glacial Tills in Eastern Wisconsin

TUNCER B. EDIL AND DAVID M. MICKELSON

A geological and geotechnical analysis of overconsolidated till units in eastern Wisconsin forms the basis of this paper. The glacial stratigraphy and geotechnical properties (grain size, clay mineralogy, Atterberg limits, hydraulic conductivity, strength, and compressibility) of the till units are presented based on a large number of tests on samples from a wide geographical area. The clay tills exhibit varying degrees of overconsolidation depending on their vertical location, but there is no discernible difference between the overconsolidation ratio of different till units and the same effective overburden stress. The preconsolidation stresses are much lower than the total ice pressure, indicating limited drainage during ice loading, possibly because of permafrost conditions that prevailed during the deposition of these tills. The higher overconsolidation ratios and preponderance of jointing encountered in the upper 10 m of these tills could be attributed to groundwater lowering resulting from a drier climate that prevailed subsequent to their deposition. The theories of transport and deposition of glacial till are reviewed and interpreted for the tills of eastern Wisconsin.

Although glaciers entered eastern Wisconsin numerous times in the past, only deposits of the last (late Wisconsin) glaciation are present. They overlie dolomite throughout the area. The path of glacier ice was controlled by the regional topography, and lobes of ice went south into Illinois in the Lake Michigan basin (Lake Michigan Lobe) and south to Madison in the Green Bay–Lake Winnebago lowland (Green Bay Lobe). The lobes advanced into Wisconsin about 23,000 B.P. (before present) and fluctuated numerous times until about 11,000 B.P, when ice finally left the area. The glacier fluctuations left till sheets of different composition, and texture is controlled mostly by the absence or presence and extent of ice-marginal lakes that formed in front of the ice margin. Some readvances of ice incorporated lake sediment, producing a fine-grained till, and other readvances incorporated sand and gravel, producing sandy till.

Significant differences in engineering characteristics may also result from the nature of glacial transport and deposition of the till. It has been suggested that most till in areas away from Lake Michigan was deposited by meltout from debris-rich ice after retreat of ice that was frozen to its bed (*1*). Others (*2*) have suggested that transport of sediment takes place beneath the ice (subglacial deforming bed) as a wet, unfrozen sediment. Subsequent deposition would take place by a decrease in glacier driving stress and dewatering. These two modes of deposition may have produced differences in the internal structure of the till and its strength properties.

In this paper, the glacial stratigraphy of eastern Wisconsin and a compilation of till properties are presented, and possible effects of genesis on overconsolidation of these till units are discussed.

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STRATIGRAPHY OF GLACIAL DEPOSITS

The glacial deposits in eastern Wisconsin have been classified into formations and members (3), and their distribution is shown in Figure 1. Each unit contains till and associated sand and gravel. Generally, major distinctions among units are based on till properties. Sand and gravel are then classified in one formation or another based on correlation with a till unit. The lowermost unit in the Lake Michigan basin, the Tiskilwa Member of the Zenda Formation (Figure 1), contains light reddish-brown silty till. In many areas it rests on bedrock, although in shore bluffs in southern Wisconsin it overlies deformed sand and gravel. Because it is generally thin or absent, few engineering properties have been developed for those materials.

Much of southeastern Wisconsin is covered by sandy, stony till of the New Berlin Formation (Figure 1). The till of this formation generally contains about 65 percent sand in the less than 2-mm fraction and is very rich in dolomite (Table 1). It was evidently deposited when glacier ice was excavating bedrock and sand and gravel and therefore depositing coarse till. In many places it rests on dolomite. Along the Lake Michigan shoreline where it outcrops above beach level, it forms a resistant layer and also an accumulation of boulders on the beach, which slows the rate of erosion of the shoreline. Extensive sand and gravel at the surface and in the subsurface are also considered part of the New Berlin Formation.

By 14,000 B.P. ice retreat had extended far enough north to allow a large lake basin to form in what is now southern Lake Michigan (*4*). Subsequent advances then incorporated lake sediment, therefore depositing a clayey till. Within 10 km of the Lake Michigan shoreline and in the Lake Winnebago–Green Bay lowland, till sheets are commonly separated by lake sediment units that are silty clay or interbedded fine sand and silt. The Oak Creek Formation (Wadsworth Formation of Illinois) contains an extensive gray, clayey till that extends from north of Milwaukee around the south end of the lake basin and northward along the shore of the lake in the state of Michigan. The till is thick (greater than 30 m) in moraines but thin (often less than 3 m) between moraines. Very little sand and gravel is associated with this formation, presumably because little coarse material was available to the streams of meltwater that flowed from the ice. The till is fractured to depths of at least 10 m, and this provides for passage of water and contaminants through the upper part of the unit. Below the 10-m depth, where fractures often appear to be closed or nonexistent, hydraulic conductivity is very low. Presence of the fractures in surface material allows rapid recharge of the groundwater system in locations where Oak Creek till is less than about 10 m thick over sand or sand and gravel. Although moraines in the Oak Creek Formation indicate several readvances, there is no significant difference in engineering properties of Oak Creek till from one moraine to another. In its outer regions (behind what is known as the Valparaiso Moraine) Oak Creek till is thin and sometimes absent.

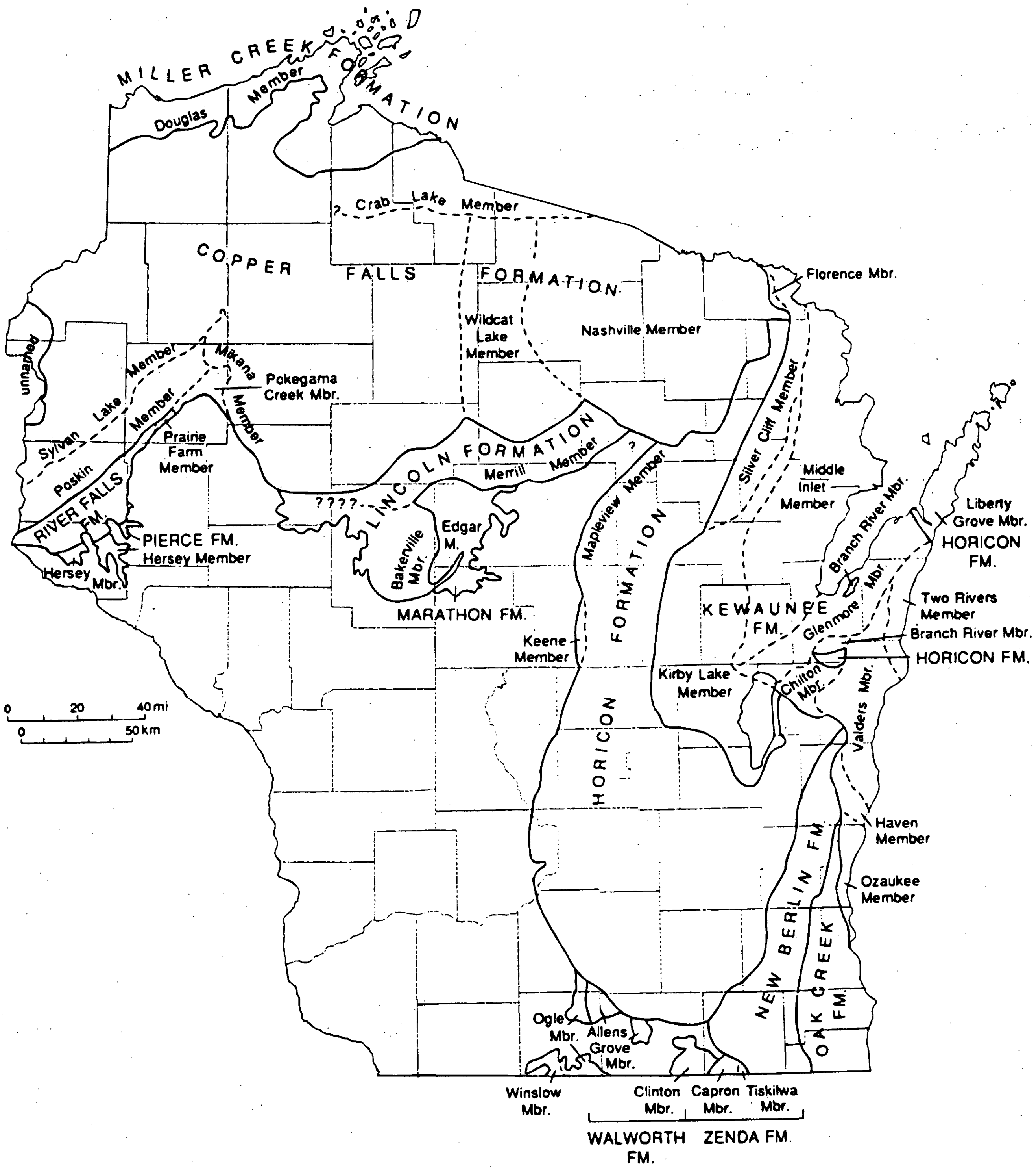

FIGURE 1 **Extent of till units in Wisconsin.**

At about 13,000 B.P. a series of ice advances began in both the Green Bay and the Lake Michigan lobes that deposited so-called "red till." Just previous to 13,000 B.P. drainage from Lake Superior carried red clay from that basin into the Green Bay and Lake Michigan basins. Subsequent glacial advances deposited reddish-brown silty clay till throughout eastern and northeastern Wisconsin. All of these units are included in the Kewaunee Formation, and numerous members have been described and defined (Figure 1), (*5,6*).

Clayey red-brown till extends southward to Milwaukee in the Lake Michigan Lobe and just south of Lake Winnebago in the Green Bay Lobe. There is little documentation of the engineering properties of these units in the Green Bay Lobe, but those along the Lake Michigan shoreline were analyzed in a 1977 shoreline erosion study (*7*). Grain size and other characteristics of the units are given in Tables 1 and 2. Most of the members cannot be distinguished in the field unless the stratigraphic position is known. A combination

TABLE 1 Mean Grain Size and Atterberg Limits of Tills in Eastern Wisconsin

Till Unit	Percent* Sand	Percent* Silt	Percent* Clay	Number of Samples	Liquid Limit (%)	Plasticity Index	Number of Samples	Activity
Two Rivers M.&	31	50	19	11	26	12	3	0.63
Valders M.&	30	52	18	33	29	15	54	0.83
Haven M.&	16	56	28	27	28	15	14	0.53
Ozaukee M.&	13	47	40	19	28	14	25	0.35
Oak Creek Fm.	12	53	35	566	31	15	627	0.42
New Berlin Fm.	58	29	13	15	17	4	14	0.31
Tiskilwa M.	42	35	23	8	22	9	4	0.39

* Percent of < 2mm fraction. Upper size boundaries used are 2, 0.0625, and 0.002 mm for sand, silt and clay, respectively. Standard deviations were 4 to 8 for grain size and 2 to 6 for Atterberg limits.
& Members of the Kewaunee Formation. Tiskilwa is a member of the Zenda Formation.

of clay mineral composition, texture, and magnetic susceptibility can be used to distinguish the units in the laboratory, and these are described in more detail by Acomb (*8*). Throughout the extent of the Kewaunee Formation, lake sediments are interbedded with the till units. Many of the lake sediments along the Lake Michigan shoreline are interbedded fine sand and silt. In the central part of the Green Bay–Lake Winnebago basin, laminated silt and clay and, in places, massive lake sediment can be mistaken for till.

GEOTECHNICAL PROPERTIES

The soils of eastern Wisconsin vary considerably and, as discussed, can be broadly grouped as glacial till, lacustrine silt and clay, and sandy outwash based on their genesis and geotechnical behavior. The first two are generally cohesive, and the last is cohesionless. The properties and behavior of only the tills are considered here. The geotechnical properties of each unit and variation of these properties between units are presented.

Index Properties

The mean index properties of the till units are summarized in Table 1. Size of the sample population varies from one till unit to another, from as few as 3 to as many as 627 samples. The liquid limits and plasticity indices for all tills, except New Berlin and Tiskilwa tills, vary in a relatively narrow range. Practically all of the means for liquid limit vary between 22 and 31, with corresponding means for plasticity index between 9 and 15 percent. These tills can be classified broadly as low-plasticity silts and clays (CL or CL-ML according to the Unified Soil Classification System). New Berlin till has distinctly different composition and index properties compared with the other tills. However, differences in these properties for the remaining till units are not significant.

The mean textural composition given in Table 1 and the mean clay mineral percentages given in Table 2 indicate marked differences in clay content and amount of expandable minerals among these till units that are not reflected in the Atterberg limits. For example, Ozaukee till has a clay content 12 to 22 percent higher than Haven and Valders tills, respectively (Table 1), yet its liquid limit falls within the same range as those of the Haven and Valders tills. Mean activity numbers, obtained by dividing the mean plasticity index by the mean percent clay fraction, vary somewhat from one till unit to another, often balancing the influence of texture on Atterberg limits, resulting in materials hard to differentiate on the basis of Atterberg limits alone. These groups are usually fairly easily distinguishable by color, and the so-called red tills (Two Rivers, Valders, Haven, and Ozaukee) have long been recognized as distinct from the "gray tills" (Oak Creek) (*9*).

TABLE 2 Mean Relative Clay Mineral Percentages of Tills in Eastern Wisconsin

Till Unit	Percent Expandable Clays*	Percent Illite*	Percent Kaolinite and Chlorite*	Number of Samples Tested
Two Rivers Member&	35	52	13	42
Valders Member&	46	42	12	53
Haven Member&	25	56	19	58
Ozaukee Member&	20	60	20	20
Oak Creek Formation	14	70	16	81
New Berlin Formation	17	66	17	26
Tiskilwa Member	18	67	15	24

*Percentages are relative amounts of clay minerals analyzed (total always adds to 100%). Expandables include smectites and vermiculite. Standard deviations are typically less than 5.
&Members of the Kewaunee Formation. Tiskilwa is a member of the Zenda Formation.

TABLE 3 Mean Hydraulic Conductivity of Tills in Eastern Wisconsin

Till Unit	Laboratory Hydraulic Conductivity (cm/s)	Field Hydraulic Conductivity (cm/s)
Two Rivers Member*	4.0×10^{-8} (11)	- - -
Valders Member*	4.0×10^{-7} (19)	3.2×10^{-5} (12)
Haven Member*	5.0×10^{-8} (27)	5.0×10^{-6} (9)
Ozaukee Member*	- - -	1.6×10^{-7} (1)
Oak Creek Formation	1.8×10^{-8} (102)	1.4×10^{-8} (153)
New Berlin Formation	- - -	2.0×10^{-5} (20)

Note: Number of tests is given in parentheses.

* Members of the Kewaunee Formation.

Hydraulic Conductivity

Hydraulic conductivity data from solid waste, hazardous waste, and sewer pipeline investigations submitted to the Wisconsin Department of Natural Resources by engineering firms were grouped according to till stratigraphic units (*10*) and were supplemented by data from new field sites (*11*). Differences in hydraulic conductivities of the till units are controlled by the grain size distribution of the till. Table 3 presents the mean hydraulic conductivities of various till units as measured in the field as well as on laboratory samples. The influence of the coarser texture of New Berlin till (Table 1) is reflected in its higher hydraulic conductivity. Field-measured hydraulic conductivity of the fine-grained units (all but New Berlin) is higher than laboratory-measured values. This difference is probably due to significant fracture porosity observed in the upper several meters of the fine-grained tills. Fractures and sedimentary heterogeneity in the upper 10 m cause hydraulic conductivity to be much higher than below the 10-m depth (*12*).

Shear Strength

The mean natural water content and density values of the tills are given along with the effective strength parameters in Table 4. The lack of differentiation observed in the Atterberg limits of the Kewaunee and Oak Creek formations is also apparent for the natural water content. Mean unit dry weights also vary in a relatively small range (17.7 to 19.9 kN/m^3), and all of these tills are very dense. The standard penetration number of the tills varies with water content, from a high of about 50 down to 20 blows /0.3 m. The logarithm of unconfined strength of the tills follows a typical linear relationship with water content. There is an observable differentiation in this relationship between the Kewaunee and Oak Creek formations. The unconfined compressive strength of these till units varies between 200 and 500 kPa (*13*).

The effective strength parameters, ϕ' and c' (Table 4), are parameters normalized with respect to the influence of consolidation pressure and corresponding equilibrium void ratio. Thus, they are fundamental parameters and basically are functions of the composition, texture, fabric, and stress history of soils. They are determined from consolidated, undrained, triaxial compression tests with measured pore pressures. The sample population is much smaller for these tests, varying from one to five samples per till unit; however, often the samples of the same unit were obtained over great distances from each other. Based on the *SD* of the effective friction angle and cohesion, it becomes apparent that the shear strength parameters vary within very narrow limits for a given till unit despite the geographic distances involved. The effective angle of internal friction,

TABLE 4 Mean Natural Density, Water Content and Effective Strength Parameters of Tills in Eastern Wisconsin

Till Unit	Dry Unit Weight (kN/m^3)	Water Content (%)	Friction Angle* (degrees)	Cohesion* (kPa)
Two Rivers Member&	19.0	16	30	11
Valders Member&	17.7	17	29	28
Haven Member&	18.6	17	31	24
Ozaukee Member&	17.9	18	30	7
Oak Creek Formation	17.7	18	31	6
New Berlin Formation	19.9	8	35	0
Tiskilwa Member	19.3	14	27	17

* Number of samples tested varied from 1 to 11. Standard deviations were 0.5 to 3 degrees for friction angle and 5 to 13 kPa for cohesion.

& Members of the Kewaunee Formation. Tiskilwa is a member of the Zenda Formation.

ϕ', had an *SD* of less than 0.5 to 3 degrees in each till unit. Most units exhibited no or low effective cohesion intercept, whereas Haven and Valders tills had effective cohesion intercepts varying between 20 and 30 kPa. A generalization cannot be drawn relating the Atterberg limits of a particular sample and its effective strength parameters. Presence of higher cohesion intercept in the Haven and Valders Members indicates higher overconsolidation of these tills in the range of test consolidation pressures (100 to 600 kPa). This overconsolidation can possibly be traced back to the processes that took place during deposition, postdepositionally, or both, and this is discussed in the next section.

Compressibility and Preconsolidation

The conventional consolidation tests performed on selected samples from the Kewaunee Formation tills provide information regarding the compressibility and stress history (Table 5). The preconsolidation stress (the maximum vertical stress under which the soil is consolidated) can be estimated from a laboratory compression curve by observing the stress at which a change in the slope of the compression curve occurs from recompression to virgin compression. This transition is gradual, so it may not be very easy to identify the preconsolidation stress. Four methods of determining the preconsolidation stress, including the most widely used procedure suggested by Casagrande (*14*), were used (*15*). The most probable values of the preconsolidation stress based on these methods are summarized in Table 5 along with the other compression parameters.

These tills, in general, are relatively stiff, with compression indices between 0.10 and 0.20 with a mean value of 0.16. The compression index values obtained from the consolidation tests compare well with the values predicted by the empirical equation based on liquid limit (*16*). The overconsolidation ratio (OCR) varies in general with depth of the sample or, more specifically, with the effective overburden stress (σ_0) as shown in Figure 2. For $\sigma_0 < 100$ kPa, OCR is quite high (9 to 31); for $\sigma_0 = 100$ to 200 kPa, OCR = 4, and for larger σ_0, OCR decreases to 2 (at 330 kPa). Presence of fractures in the upper 6 to 9 m in these tills supports the high overconsolidation values observed in the laboratory for $\sigma_0 < 100$ kPa.

OVERCONSOLIDATION OF TILLS

The characteristics of tills deposited by various glacial advances are relatively consistent over extended distances (Figure 1), roughly along the direction of transport. Natural water content, liquid limit, and plasticity index (Tables 1 and 3) show minor differences among the various tills, with the exception of New Berlin till, even though they exhibit discernible compositional differences. The effective friction angle also varies over a relatively narrow range within each till unit as well as between till units, except New Berlin till. Effective cohesion intercept, although varying relatively little within each till unit over large geographic distances, is markedly higher for Valders and Haven tills than the others. The consolidation stress history, in addition to the compositional factors, is the most important factor in defining the mechanical properties of soils. Therefore, a careful consideration of stress history is warranted for a clearer understanding of the mechanical behavior of the tills.

Consolidation Stress History

Traditionally, the transport of sediment by glaciers has been observed and interpreted to be either supraglacial (on top of the ice), englacial (within the ice), or basal (as a debris-rich ice layer at the base of the glacier). Release of sediment from the base of the ice to produce till takes place by lodgment (plastering on) from beneath an active glacier sole or by melt-out of the debris-rich layer after it has stopped being transported by the glacier. Sediment is also released at the ice surface because of melting or sublimation. This sediment is slowly let down onto the ground surface as the ice below it melts. Both of these processes have been observed on modern glaciers in Alaska (*17–19*). Till deposited from basal debris-rich ice is called basal till and is characterized by having fairly uniform properties over broad areas, a wide range in grain size, poor sorting, and little stratification, and it is being compact and sometimes overconsolidated. Supraglacial sediment is typically loose and normally consolidated, more variable in texture over small distances both vertically and horizontally, and typically more rich in boulders.

In an earlier investigation of the preconsolidation characteristics of eastern Wisconsin tills by the authors, it was believed that there

TABLE 5 Compressibility and Preconsolidation of Kewaunee Formation Tills in Eastern Wisconsin

Till Unit	Depth of Sample (m)	Initial Void Ratio	Compression Index	Effective Overburden Stress (kPa)	Preconsolidation Stress (kPa)	Over-consolidation Ratio, OCR
Two Rivers	1.5	0.50	0.20	30	931	31
Member	4.8	0.49	0.15	109	518	5
Valders	2.4	0.38	0.10	58	518	9
Member	6.9	0.54	0.20	156	614	4
Haven	6.5	0.48	0.17	87	835	10
Member	6.6	0.44	0.12	132	518	4
	8.4	0.49	0.14	182	672	4
	9.2	0.48	0.11	118	413	4
Ozaukee	6.0	0.54	0.20	130	422	4
Member	12.3	0.54	0.13	156	413	3
	15.3	0.51	0.20	330	634	2
	15.3	0.48	0.17	330	672	2

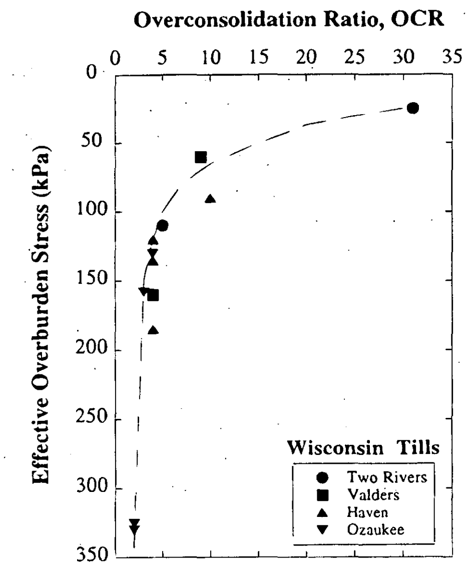

FIGURE 2 Over consolidation ratio versus effective overburden stress for Kewaunee Formation.

was a difference in the overconsolidation of these tills and thus in their mode of deposition (*20*). Several workers, especially Boulton (*21*), have also tried to explain the presence or absence of overconsolidation in tills. Harrison (*22*) suggested that the weight of glacial ice was responsible for creating the preconsolidation often found in tills. Others have suggested that desiccation, fluctuation of the water table, erosion of overlying materials, and ability of pore water to drain under loading are also ways of explaining preconsolidation (*23*). Boulton (*21*) concluded that the state of consolidation "depends almost entirely on depositional and postdepositional changes." He also concluded that lodgment tills are generally overconsolidated, that flow tills often have low preconsolidation pressures, and that melt-out (ablation) tills seem likely to be normally consolidated. In the cases investigated here it seems likely that all of the tills are basal in origin. Washed or flow tills have been noted in local areas but were not included in the samples reported here. In some localities, the lower parts of the till units are probably waterlaid, but these were also avoided for the geotechnical sampling. Thus, the apparent difference in the overconsolidation between the two younger units (Valders and Haven) and the two older units (Ozaukee and Oak Creek) as evidenced by the difference in effective cohesion and preconsolidation stress had to be explained (*20*).

All of these units occur at different heights above Lake Michigan in different places along the bluff, and these samples of the tills were taken at various depths below the bluff top. It seems unlikely that the mode of deposition or general source of materials was different. All of the units are fine-grained compared with the older New Berlin till. All seem to have been derived from fine-grained lake sediments deposited in the Lake Michigan basin. The lake level during the time of deposition of the tills was probably the same (Glenwood, or about 18 m above present level) during the deposition of all of these tills. There is no consistent relationship between the presence or lack of overconsolidation and the presence of permeable sandy units above or below the till. There is no stratigraphic reason why Haven and Valders tills should have drained more easily than the others.

The consolidation tests (Table 5) indicate that, in all cases, the preconsolidation stress is less than the total effective stress that the glacier ice would be expected to exert (as much as 3000 kPa) if tills were deposited under fairly thick ice or during ice advance and if pore pressures were fully dissipated. Another possible factor causing different overconsolidation is a difference in load due to ice thickness. This explanation would require thicker ice during or after the deposition of Haven and Valders tills than the older tills. There seems to be no evidence of this (*5*). In addition, there are situations in which overconsolidated till lies stratigraphically above, in the same section, as a till showing less overconsolidation. Clearly, factors other than ice thickness are more important in determining the values measured today.

The degree of overconsolidation exhibited by a soil is related not only to ice thickness but also to the duration of loading and the ability to drain. As discussed here, the duration of loading (hundreds of years) is sufficient for 100 percent consolidation if adequate drainage is provided, and therefore the ability to drain appears to be the remaining factor controlling the amount of overconsolidation. It is expected that near the margin of a stagnant (not advancing) glacier, complete drainage could occur through the underlying layers; however, farther under the ice mass, where flow paths are longer, incomplete drainage would be the rule rather than the exception even though downward groundwater gradients might be quite significant. Field investigations (*24*) have indicated that the pore-water pressure at the bed of modern temperate glaciers can have a head on the order of two-thirds of the ice thickness. In this situation, the soil would be consolidating under an effective stress equal to only a portion of the weight of the ice mass. This may partially explain the relatively low overconsolidation values of these tills but not the differences among their OCRs.

In the late 1970s it was thought that a likely reason for differences in OCRs was the temperature regime and resulting distribution of frozen or melted bed and subbed (*20*). Attig et al. (*25*) have shown that in southern Wisconsin, tundra conditions were present until about 14,000 B.P., when the ice melted and a spruce-dominated woodland developed. They suggest that permafrost lasted until at least 13,000 B.P. in northeast Wisconsin.

The best minimum date available on the advance of the Ozaukee till is about 13,500 B.P. in the northern part of the lower peninsula of Michigan (*26*). Minor retreat after deposition of this till was followed by advance and deposition of the Haven and Valders tills about 13,000 B.P. All but Two Rivers till, deposited about 11,800 B.P., were deposited under permafrost conditions. Therefore, it was concluded (*20*) that the difference in preconsolidation of the tills might be due to the presence or absence of ice in pore spaces of the till while it was under an ice load. Additional consolidation data obtained since then, as presented in Table 5 and Figure 2, now indicate that the differences in overconsolidation are more a function of sample depth than till unit, with all tills having approximately the same OCR at the same range of effective overburden stress, i.e., OCR of 4 for σ_0 of a range of 100 to 200 kPa.

Recent Theories of Transport and Deposition of Tills

During the last 10 years a new hypothesis of subglacial transport and deposition has developed. Evidence of deformation, such as folding and faulting, has commonly been observed in and beneath till sheets. Most interpretations before 1980 stated that the till was carried as debris-rich ice and that the deformation of stratified sediments below took place in either a frozen or unfrozen state because of shear stress applied from above. In most deformed units below till, strain is relatively small relative to the movement of the glacier above. Now a number of researchers argue that in some situations, much of the flux of glacier ice is the result of transport on a deforming bed beneath. It is suggested that this unfrozen deforming layer can be several meters thick and can result in till evidently similar in most respects to that derived from basal melting. It is further suggested that this type of deformation takes place beneath the Antarctic ice sheet (*27,28*) as well as the Pleistocene ice sheets (*2,29*). Theoretical considerations are given by Boulton and Hindmarsh (*2*) and Boulton (*30*). Although there is no field evidence for extensive subglacial deformation in the unfrozen state in eastern Wisconsin, it may be that within the Lake Michigan basin subglacial deformation was a significant contributor to southward transport of sediment and ice.

Whether or not sediment is mobilized and transported beneath the ice is controlled by the relationship between shear strength, as determined by the Coulomb relationship, and the driving stress or basal shear stress. The driving stress is controlled by the following relationship:

$$\tau = \rho g h \sin \alpha$$

where

τ = shear stress,
ρ = density of ice above,
g = acceleration of gravity,
h = thickness, and
α = surface slope of the ice.

Sin α and h are self-adjusting to produce the flow of ice required by continuity arguments. It is argued that where bed strength is low because of low effective pressure (high pore pressure) or where the sediment has low friction angles, ice surface slope is low, and thickness is not as great for any given distance back of the margin. In areas where bed materials are stronger, the ice surface slope is greater, and the ice is thicker at any given distance behind the ice margin.

It is argued that there are several degrees of deformation possible in deformable beds. The thickness of the continuously deforming layer is a function of the balance between strength and driving stress. Hart et al. (*31*) predict that this reaches a maximum perhaps 50 km behind the ice margin, and the base of deformation decreases under thicker ice as well as toward the ice margin. Under the right conditions up to 10 m of sediment could be continuously deforming and producing the layers that are now interpreted as till.

Alley and others (*27,28*) have argued that this continuously deforming layer is responsible for the rapid flow velocities of ice streams in the Antarctic. Although this has not been observed first hand, geophysical evidence suggests a wet, very weak layer just beneath the base of the glacier. Because of the shape of the Lake Michigan basin it is possible that the ice sheet here behaved in a similar way. Water would have been confined because of the shape of the basin and the fine-grained nature of rocks and lake sediment beneath. This would lead to low effective stress and relatively low resistance to deformation. Ronnert (*32*) examined sediment at three localities along the Lake Michigan shoreline and concluded that sediments were deposited from debris-rich ice and were not continuously deforming unfrozen sediment. Others, however (*33*), have argued that there was likely a deforming bed in the Lake Michigan basin at the time the tills under discussion were deposited.

It is not clear how the process of deposition affected the OCR, except that if the zone of deforming bed thinned as ice thinned, this hypothesis would predict larger OCRs with depth. The overconsolidation profile evident in Figure 2 shows the highest OCRs near the ground surface, the opposite of the above hypothesis. On the other hand, a drier climate prevailed in this region between 9,000 and 5,000 B.P., which would be expected to have lowered the groundwater table to levels below the current levels. Soderman and Kim (*34*) suggested that a period of lower water table was responsible for overconsolidation of near-surface samples of St. Claire till in Ontario. This argument is further supported by the evidence of oxidation of the tills sampled below the current groundwater table. The authors noted in field investigations that such samples had a reddish-gray matrix with a thin reduction zone adjacent to the joints in the till. This long-term lower groundwater episode is likely to explain the overconsolidation of the tills and the preponderance of jointing observed in the upper 6 to 9 m, which coincides with the most heavily overconsolidated till samples.

CONCLUSIONS

The glacial stratigraphy and geotechnical properties (grain size, clay mineralogy, Atterberg limits, hydraulic conductivity, strength, and compressibility) of eastern Wisconsin tills are presented based on a large number of tests conducted on these materials under the authors' supervision over many years. These till units show discernible differences in their grain size distribution and mineralogical composition. However, these differences are not reflected in their index properties (Atterberg limits) and are not traceable in their strength and compression parameters. In other words, index properties do not lead to identifying different till units. The clay tills exhibit varying degrees of overconsolidation depending on their depth, but there is no discernible difference between the OCR of different till units with the same effective overburden stress. The preconsolidation stresses are much lower than the total ice pressure, indicating limited drainage during ice loading possibly due to permafrost conditions that prevailed during the deposition of these tills. Decreasing OCRs with depth are not consistent with the view of a thinning deforming layer during deglaciation. The higher OCRs and the preponderance of jointing encountered in the upper 10 m of these tills could be attributed to groundwater lowering resulting from the drier climate that prevailed subsequent to the formation of these tills.

The ability to identify different till units is important because it leads to better correlation among the properties of samples retrieved from a given unit. The ranges of values reported for engineering properties allow better prediction of the distribution of measured properties based on a smaller number of samples. Furthermore, a clear knowledge of the cause of overconsolidation is important in assisting us to know where to expect it because it is an important factor in differentiating the engineering properties of the tills.

ACKNOWLEDGMENTS

The data presented were generated in the course of numerous projects and theses that the authors supervised over many years. The following former students were responsible for performing many of the tests or gathering of the data: B. J. Haas, L. J. Acomb, S. R. Pulley, T. W. Wolf, W. W. Simpkins, and S. A. Rodenbeck.

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Geotechnical Engineering Practice in Overconsolidated Clays, San Diego, California

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A significant portion of the San Diego metropolitan area is underlain by overconsolidated clays. The erosion of overburden soils has been the primary mechanism for overconsolidation. A decrease in regional water levels has also contributed to the relatively high levels of overconsolidation observed in several soil formations. Overconsolidated clays within the San Diego area are generally Cretaceous to Pleistocene in age and are characterized by a high expansive potential and low residual shear strength. The current status of geotechnical engineering practice in overconsolidated clays within the San Diego metropolitan area is discussed, particularly as it relates to foundation engineering and slope stability analysis.

The population of San Diego has increased tremendously since the late 1940s. A major portion of this population growth has occurred in areas underlain by overconsolidated (OC) clays. Except for the immediate coastal area, the topography of the San Diego metropolitan area generally consists of mesas and incised canyons. This topography requires extensive hillside construction techniques to provide building pads for residential and commercial developments. Construction earthwork (grading) operations involving excavation or filling, or both, in excess of 33 m (100 ft) in elevation are not uncommon. Hence, significant quantities of OC clays can be generated or exposed during the grading operations for a large residential or commercial project. The geotechnical characteristics of these OC clays has often resulted in damage to building foundations and ancillary site improvements. Deep-seated landslides and surficial slope failures, in both natural soil formations and man-made fills, have also occurred because of the presence of OC clays.

GEOLOGICAL AND GEOTECHNICAL CHARACTERISTICS

The metropolitan San Diego area is located in the Coastal Plain province encompassing the strip of land from the Pacific Ocean to approximately 8 to 16 km (5 to 10 mi) inland and parallels the coastline from the Mexican border to the greater Los Angeles area (Figure 1). Granitic rocks of the southern California batholith outcrop east of the Coastal Plain province and typically possess a shallow (less than 1.5 m) soil mantle. OC clays occur primarily within the Coastal Plain province where they are present in marine and nonmarine sedimentary deposits. Except for local faulting or folding, the inclinations of OC clay layers are nearly horizontal with regional dips on the order of 5 to 10 degrees.

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Two previous regional studies that included evaluations of the geotechnical engineering characteristics of OC clays throughout San Diego County were performed by Kennedy (*1*) and by Pinckney et al. (*2*). The study by Kennedy (*1*) consisted of detailed geologic mapping and analysis of the San Diego metropolitan area. As part of his work, numerous samples of OC clays from different soil formations were obtained and were tested for Atterberg limits, grain size, and mineralogy. Studies by Pinckney et al. (*2*) evaluated the geotechnical engineering characteristics of OC clays in areas of known or suspected landslides.

The results of laboratory tests performed by Kennedy (*1*) on OC clays over a wide area of San Diego County are summarized in Table 1. A review of Table 1 indicates that, with the exception of the bentonite layers of the Otay Formation, the Atterberg limits of the OC clays from the various soil formations are comparable, as is the clay fraction of the soil. Individual test results were not presented by Pinckney et al. (*2*); however, they reported that measured geotechnical engineering properties of OC clays from the Friars Formation exhibited similar values to those reported by Kennedy (*1*). Atterberg limit test results from the Mission Valley, Ardath shale, and Del Mar Formations were higher than those presented by Kennedy (*1*). In general, their test results indicated a liquid limit between 75 and 85, with plasticity indices ranging from 40 to 60. Residual shear strengths measured in consolidated-drained (CD) direct shear tests consisted of effective friction angles of 6 to 12 degrees, with effective cohesion values less than 9.5 kPa (200 psf). All of the laboratory test results reported by Pinckney et al. (*2*) were performed on OC clays located within known or suspected landslide areas. Additionally, the majority of their tests were performed on clays obtained within shear zones believed to represent the basal sliding surfaces of the landslides.

The Friars Formation has been the most extensively studied of the various soil formations containing OC clays because of its significant areal extent and, hence, impact on residential and commercial developments. The mineralogical composition of the Friars Formation is primarily montmorillonite, with traces of kaolinite and quartz, whereas the Atterberg limits generally range from a liquid limit of 45 to 80 with a plastic limit of 20 to 30 (*1*). In situ water contents are typically 15 to 20 percent, resulting in a liquidity index of approximately 0 to 0.10. Neither of the two referenced regional studies included consolidation tests or maximum shear strength tests on any of the OC clays. Consolidation tests performed by the author for numerous development projects in the San Diego area indicate that the overconsolidation ratio (OCR) of the Friars Formation ranges from approximately 10 to 20. It has been estimated that 60 to 120 m (200 to 400 ft) of overburden soil has been eroded above the top of the Friars Formation. Direct shear tests performed

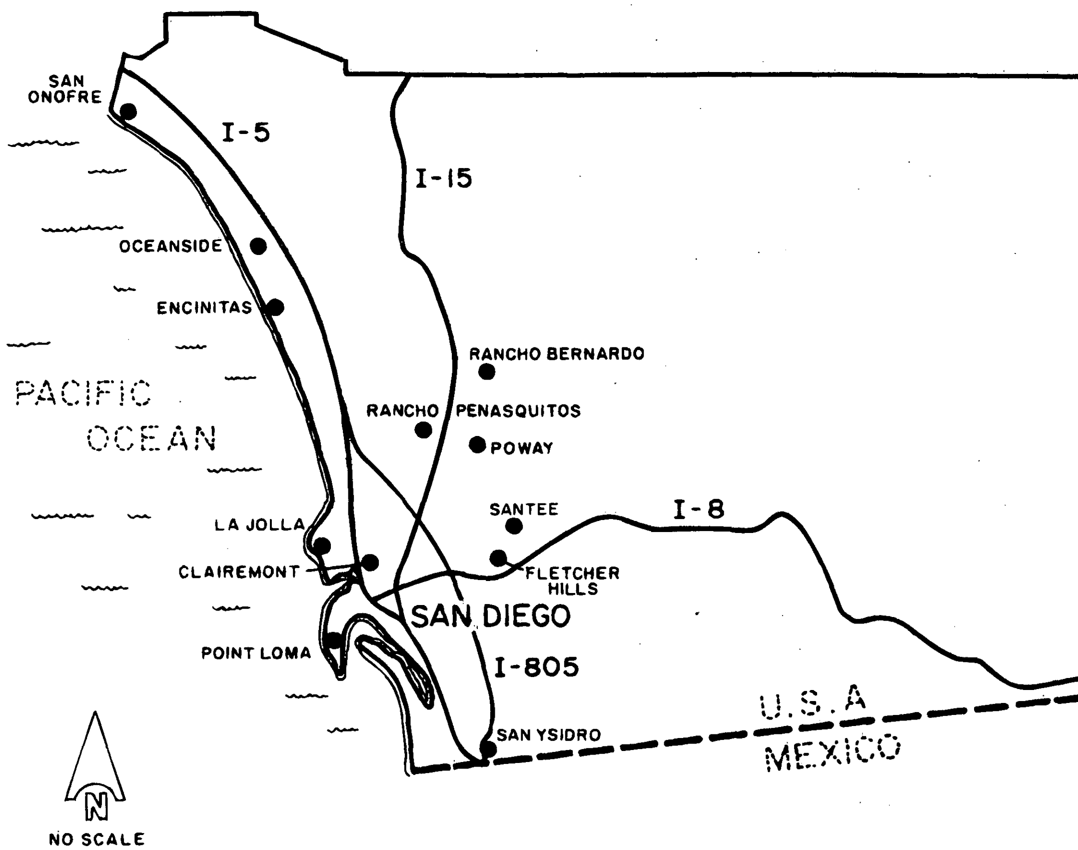

FIGURE 1 Map of San Diego County, California.

on 6-cm-diameter by 2.5-cm-thick ring samples obtained with a modified penetration sampler typically indicate an effective peak friction angle of 28 to 35 degrees and a cohesion of 14 to 35 kPa (300 to 700 psf) for "undisturbed" specimens of the Friars Formation when a single "best fit" line is applied to the direct shear test data. Triaxial testing of OC clays is rarely performed because of the difficulty in obtaining an undisturbed sample of the soil with conventional tube sampling equipment. Normalized undrained shear strength ratios for triaxial, anisotropic compression testing can be estimated from the following equation (*3*):

$$S_u/\sigma'_{vo} = 0.30OCR^{\lambda} \quad (1)$$

TABLE 1 Overconsolidated Clay Atterberg Limits and Clay Fraction

Soil Formation	Liquid Limit	Plastic Limit	Plasticity Index	Clay Fraction %<0.002mm
Del Mar	40-60	15-25	15-40	20-50
Friars	45-80	20-30	20-50	15-60
Mission Valley	40-55	10-25	15-25	20-40
Ardath Shale	40-60	20-35	20-30	10-35
Otay (Bentonite)	90-130	30-45	60-100	35-60

where

S_u = undrained shear strength,
σ_{vo} = effective overburden pressure,
OCR = overconsolidation ratio, and
λ = strength rebound parameter.

Using an average strength rebound parameter of 0.8 (*3*) and OCR values from 10 to 20 results in normalized shear strength ratios of approximately 1.8 to 3.2. Hence, for depths of interest in typical geotechnical engineering analyses (3 to 20 m), the maximum undrained shear strength would vary from approximately 190 kPa (4000 psf) at 3 m to 625 kPa (13,000 psf) at 20 m.

Residual shear strength testing is commonly performed on OC clays in areas where ancient landslides are suspected or in areas where new construction will result in excavation slopes that will expose OC clays. Current practice is to perform CD residual shear tests in a direct shear box apparatus. Ring shear testing is also becoming more common. Residual shear strength test results for the majority of OC clays typically ranges from 6 to 10 degrees (effective friction angle), with nominal values of cohesion (2.5 to 5 kPa). As discussed by Skempton (*4*), the residual shear strength friction angle is nonlinear, particularly at low levels of effective normal stress. However, the current standard of practice typically consists of using a single value of effective friction angle and cohesion for geotechnical engineering analysis.

The expansion potential of an OC clay is typically measured in a one-dimensional, free swell test known as the Expansion Index (EI) test. Uniform Building Code Standard 29-2 presents recommended test procedures (*5*). The soil specimen is remolded to a water content corresponding to approximately 50 percent saturation. It should be noted that a standard soil dry density is not specified and that a specific gravity of 2.7 is assumed for the soil. A surcharge pressure of 7.2 kPa (150 psf) is applied to the top of the specimen, and the specimen is then inundated with distilled water. The vertical strain is measured, and at equilibrium (typically 6 to 24 hr), the EI is calculated as $\Delta H/H * 1000$, where ΔH is the vertical movement (swell), and H is the original sample thickness. Expansive soil classifications based on EI test results have been developed by the Uniform Building Code as follows:

Expansion Index (EI)	*Expansion Potential*
0–20	Very low
21–50	Low
51–90	Medium
91–130	High
Above 130	Very high

EI values for OC clays are commonly in excess of 90, with several bentonite-rich OC clays exhibiting EI values above 150. The EI test is performed on soil that is finer than the No. 4 (6.35-mm) sieve. Many expansive OC clay soils in the San Diego area consist of gravel to cobble materials within a clay matrix. Houston and Vann (*6*) presented data that indicated that the expansion characteristics of a clay soil are significantly influenced by the percentage of material larger than the No. 4 sieve. Therefore, the main purpose of the EI test is to provide a qualitative assessment of the swelling potential of a soil as opposed to a parameter that can be used in the direct design of a foundation to mitigate the potential for swell of an expansive clay.

SLOPE STABILITY

Slope stability problems associated with OC clays can be divided into two categories. The first category consists of large slope movements or landslides that occur in natural soil formations along OC clay seams or lenses. These landslides may result from natural processes, such as erosion or hillside creep, or as a consequence of construction activities. Typically the sliding surface is located along a previously sheared, slickensided clay seam or lens that is at or slightly above its residual shear strength. The second category includes surficial slope movements associated with the use of OC clays for constructing fill slopes.

There are many ancient landslides within OC clays in San Diego County. A significant number of ancient landslides have been identified by Hart (*7*), Kennedy (*1*), and Pinckney et al. (*2*) in regional geologic studies. Major landslide areas include the Poway, Rancho Bernardo, Fletcher Hills, and San Ysidro areas (Figure 1). The predominant soil formations in these areas are the Friars and Otay formations. These landslides commonly have a basal surface along a low residual shear strength OC clay seam; the thickness of a clay seam can be as thin as 6 mm (0.25 in.). Ancient landslides are believed to have occurred when the climatic conditions in southern California were much wetter than today. The combination of lower effective stresses, higher soil unit weights, and active stream erosion strained the OC clays to residual shear strength values, resulting in downhill movements. Landslides then resulted because of the low residual shear strength of the OC clays. Deep-seated slope movements along OC clay seams or lenses have also occurred as a result of construction processes. The excavation at the base of a natural hillside containing an OC clay seam dipping out of slope (toward the excavation) has resulted in slope movements of varying magnitudes.

Identification and analysis of OC clay seams or lenses is a crucial part of any geotechnical investigation for a site that has, or will have, any significant topographic relief. Additionally, the presence of ancient landslides, which may have resulted in the straining of OC clay seams or lenses to residual shear strengths, is also important to identify. Because of erosion, the topographic expression of an ancient landslide is usually subdued and is not always easily identifiable from the ground surface. Stereoscopic aerial photographs can provide assistance in evaluating topographic remnants of ancient landslides, such as hummocky terrain, scarps, or altered drainages. Areas of suspected ancient landsliding, as well as areas where cut slope excavations are proposed, are typically investigated with down-hole geologic mapping techniques. A bucket-auger drill rig is used to advance a 60- to 120-cm (24- to 48-in.) diameter boring into the ground. Minimum depth of excavation is typically 3 m (10 ft) below the base of a proposed cut slope or below the basal surface of a suspected landslide. The boring is then inspected in situ by a geologist or engineer with expertise in identifying landslide materials. Geologic features used to identify landslides include the continuity and integrity of the soil materials, joints or other discontinuities, and the presence of sheared clay seams or lenses. If present, the depth and inclination of any clay seams or landslide basal surfaces are also determined for subsequent geotechnical engineering analyses. Samples of undisturbed and sheared clay materials are obtained for laboratory testing. For large projects a minimum of three bucket-auger borings are advanced to determine the continuity and three-dimensional characteristics (strike and dip) of the clay seams or lenses.

If an ancient landslide is identified within the area proposed for development or if an OC clay layer is present that may potentially affect the stability of proposed excavation slopes, laboratory testing and slope stability analyses are performed to evaluate the proposed site-grading configuration. A cross section showing a typical grading configuration and existing site condition is presented in Figure 2. Direct shear or ring shear tests are performed to determine the maximum and residual shear strengths of the OC clay material. Slope stability analyses are typically performed using commercially available computer programs. Local building code ordinances and the standard of practice result in the requirement of slope mitigation measures if the minimum factor of safety determined from the analysis is less than 1.5.

Numerous slope stabilization techniques have been used over the years to increase the factor of safety against slope movements. The most common and widely used procedure has been the construction of buttresses at the toe of the landslide or proposed slope (Figure 2). The buttress is constructed of compacted fill soil and is intended to provide an area (per unit width) of higher shear strength material, which will result in an increased resistance to slope movement. Buttress thicknesses (into the slope) can range from approximately 3 m (10 ft) (minimum width of construction equipment) to 33 m (100 ft) or more. Buttress construction begins by excavating a temporary slope into the natural hillside, which will provide the required buttress width. Because of slope stability considerations, the buttress construction is typically performed in slots parallel to the slope, with the width of a slot usually on the order of 30 to 60 m (100 to 200 ft). Using this procedure, the grading contractor builds the buttress along the length of the slope requiring mitigation. For smaller, more local areas of potential slope instability, other techniques that have been

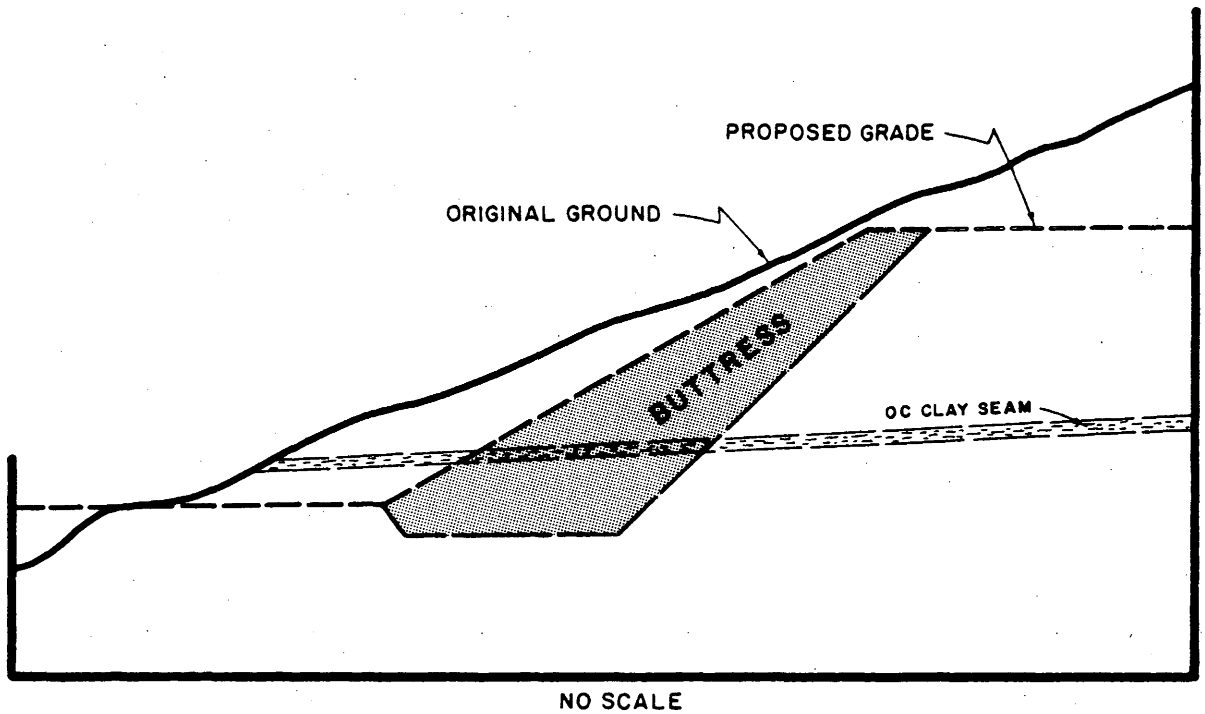

FIGURE 2 Typical cross section with buttress constructed to stabilize excavated slope.

used successfully include soil reinforced with metal or plastic strips (Reinforced Earth, geogrids, etc.) and drilled piers or caissons.

Another slope stability problem with OC clays exposed in excavated slopes is the presence of fractured or blocky clay materials. Slopes are generally excavated at inclinations of 26 to 33 degrees from horizontal and typically possess factors of safety against slope failure in excess of 1.5 (if no low-strength clay seams are present). However, the local presence of fractured or blocky clay materials in an excavation slope face can result in surficial slope instability. Where present, this potentially unstable soil is excavated and replaced with a stability fill. A stability fill consists of compacted fill soil placed within the outer 3 m (10 ft) (approximate width of earthwork construction equipment) of a slope for a height and length that encompasses the zone of fractured or blocky clay material. Other forms of mitigation, such as retention of fractured materials with wire or jute netting, can be used when aesthetics and landscaping are not of concern.

Slope stability problems can also result from the use of OC clays as fill soils when highway or railway embankments or slopes for development projects are constructed. Stability problems are typically more of a surficial nature with soil sloughage or "pop-outs" occurring in fill slopes to depths of 0.6 to 1.2 m (2 to 4 ft) below the surface. The areal extent of instability is typically on the order of 15 to 60 m (50 to 200 ft) in length (parallel to slope) and can include the entire height of the slope face. Postconstruction saturation of the outer portion of the fill slope is the primary reason for these movements. The soil to a depth of 0.6 to 1.2 m (2 to 4 ft) below the slope face can become saturated because of rainfall or irrigation waters. Saturation results in a decrease in effective stresses, which reduces the shear resistance of the soil. The soil unit weight also increases because of saturation. Additionally, insufficient compaction of the outer 1 to 1.2 m (3 to 4 ft) of a fill slope inclined at 26 to 30 degrees can result in lower shear strengths than those used for the slope stability design. Typical design strategies consist of not placing clay materials within a minimum horizontal distance of 3 m (10 ft) from the slope face to reduce the potential for surficial slope instabilities. However, the large volume of clay soil excavated from other areas of the property or a limited area available for disposing of clays generated during site excavation operations can result in clays soils being placed within the outer portions of a fill slope. In these cases, using drought-resistant vegetation with deep roots and controlling the volume of irrigation water are the typical techniques used to reduce the potential for surficial slope instability. The compactive effort applied to the outer portion of the fill slope can also be increased by overbuilding the slope (typically 1.5 to 3 m) and then trimming the soil back to the final fill slope design grade.

FOUNDATION ENGINEERING

The relatively high expansive potential of OC clays can present problems for residential and commercial buildings as well as other on-grade improvements. Expansive soils can be a problem in areas that are excavated to finish grade and that expose OC clays or in fill areas where the compacted fill consists of OC clay. Various methods of expansive soil mitigation have been used, including removal and replacement, recompaction, and the use of structurally designed posttensioned foundations and slabs-on-grade.

Geotechnical investigations for residential and commercial developments in areas of suspected expansive OC clays usually include soil borings or backhoe trenches to determine the areal extent and depth of expansive soils. Samples of the soils that may affect foundation design are obtained for laboratory testing. Based on the laboratory test results (primarily EI and Atterberg limits) and project economics, several expansive soil mitigation procedures may be viable.

Removal of expansive soils exposed at building finish pad grade and replacement with non- to low-expansive soils (EI < 50) is a common technique. This technique is often economically feasible when the export of expansive soils and replacement with non- to low-expansive soils can be performed within the boundaries of the project. Typical projects for which this would apply include large residential and commercial developments in which the expansive soils can be "wasted" in nonstructural areas, such as parks and landscape areas. Additionally, a source of non- to low-expansive soils (i.e., sand) within the materials being excavated for general site grading will enable this technique to be economically viable. Removal and replacement of expansive soil are generally not economical for small project areas or single building construction projects. The depth of removal (undercut) typically extends 1 to 1.5 m (3 to 5 ft) below finish pad grade. Chen (*8*) recommend using a minimum thickness of 1.2 m (4 ft) of non- to low-expansive soils when expansive soils are present at finish grade. The depth of expansive soil undercut is based on the expansive potential of the soil (as defined by the EI), structural footing loads and floor loads, and composition of areas immediately surrounding the building (e.g., impervious cover, such as pavement or hardscape versus permeable cover, such as landscaping). Removal of expansive soil is typically performed to a horizontal distance of 1.5 to 3 m (5 to 10 ft) beyond the perimeter of the structure or affected improvements. This technique can also be used when expansive soils are used for fill soils. The upper 1 to 1.5 m (3 to 5 ft) of a building pad can be "capped" with non- to low-expansive fill soil.

A problem that can result from the above procedure is the so-called "bathtub" effect, which occurs when water infiltrates down through the permeable non- to low-expansive soil and accumulates at the surface of the underlying expansive clay soil. Continued infiltration can result in a groundwater mound beneath the structure. Water vapor can then migrate through concrete slabs-on-grade or hardwood floors and saturate overlying carpet or other floor coverings. The potential for moisture problems can be reduced if a drain system is installed beneath the structure or if the surface of the expansive clay soil is graded so that the infiltrated water drains away from the structure.

Recompaction of expansive soils at higher water contents or lower dry densities is sometimes performed when the removal and replacement of the expansive soils is cost prohibitive. This procedure consists of excavating the soil to a depth of approximately 1.3 to 1.5 m (4 to 5 ft) below finish grade and (*a*) increasing the moisture content to approximately 4 to 6 percent above the optimum moisture content and/or (*b*) recompacting the soil at a lower relative compaction (e.g., 85 percent relative compaction in lieu of 90 percent relative compaction). As would be expected, the compaction of clayey soil at moisture contents 4 to 6 percent above optimum (approximately 90 percent saturation) can be extremely difficult. In addition, most municipal building codes within the county (including the city of San Diego) require a minimum relative compaction of 90 percent (ASTMD-1557-90) for fill soils that will support building foundations or other structural improvements. Although one- and two-story commercial and residential buildings typically have nominal foundation loads [less than 95 kPa (2000 psf)], it can be very difficult to have a city or county official agree to waive the 90 percent compaction requirements to allow a lower relative compaction to reduce the expansion potential of the soil.

A third option for designing foundations in expansive soil is to use a structurally designed footing and slab system. This will typically consist of a posttensioned slab with deepened exterior footings and possibly interior grade beams. Design of the foundation is performed by a structural engineer based on soil parameters provided by the geotechnical engineer. A majority of this design has also been codified by governing agencies. Other techniques successfully used in other parts of the country for expansive soil mitigation, such as drilled piers, have not been used in the San Diego area.

Typical minimum foundation design recommendations for residential and light industrial/commercial structures based on the EI of the soil within 1 to 1.5 m (3 to 5 ft) of finish pad grade are presented in Table 2. These recommendations are based only on the expansion characteristics of the underlying soils and do not consider the bearing capacity or settlement potential of the soil.

Bearing capacity of shallow foundations for residential and light industrial/commercial structures is rarely a problem within OC clays. Allowable foundation bearing capacities of 95 to 288 kPa (2000 to 6000 psf) with foundation dimensions on the order of 30 to 60 cm (12 to 24 in.) in depth and 45 to 122 cm (18 to 48 in.) in width are fairly typical. Deep foundations supporting heavy structures (or light structures overlying compressible soils) typically use drilled piers or driven piles embedded in the underlying OC clay. Drilled piers are commonly 61 to 122 cm (24 to 48 in.) in diameter, and driven piles commonly consist of 30- to 61-cm (12- to 24-in.), square precast, prestressed concrete piles. Steel H-piles or pipe piles are occasionally used also. The predominant deep foundation used for heavy structures is a drilled pier excavated without casing or slurry support. Steel casing and/or bentonite slurry is used in areas where there is groundwater or unstable soil conditions. The capacity of deep foundations is determined based on the side resistance and end bearing resistance of the OC clay. The current standard of practice is to design deep foundations on the basis of the undrained shear strength (S_u) of the OC clays (i.e., a total stress analysis). Side resistance of the pier or pile is calculated using the undrained shear strength of the clay and an adhesion factor (α). The adhesion factor for stiff, overconsolidated clays is typically in the range of 0.4 to 0.6 (*9*). Undrained shear strength values along the length of the pier or pile can be divided into layers or averaged together as a single value. End bearing capacity is typically determined from the total stress deep foundation bearing capacity formula $q = S_u * N_c$ where $N_c = 9.3$ and $q =$ ultimate bearing capacity. Factors of safety applied to the side resistance and end bearing resistance typically range from 1.5 to 3.0. Very few documented results of load tests performed on drilled piers or driven piles embedded in OC clays exist for the San Diego area. The results of a drilled pier load test in the Ardath Shale Formation designed to measure side resistance only were reported by Spang (*10*). The actual drilled pier capacity was much higher than that predicted, with commonly used undrained shear strength values [190 to 380 kPa (4000 to 8000 psf)]. This is believed to be the result of conservative undrained shear strength values or the use of total stress analysis in lieu of the more fundamentally correct effective stress analysis or the result of both.

CONCLUSION

Geotechnical engineering in the metropolitan San Diego area is greatly influenced by the presence of OC clays. The primary geotechnical engineering challenges associated with these clays are slope stability and foundation design. Laboratory testing procedures have been developed to assist characterizing the expansive potential and residual shear strength of OC clays. Numerous construction and design techniques have been used to reduce the potential for detri-

TABLE 2 Typical Foundation Recommendations on Expansive Soils (Residential and Light Industrial and Commercial Structures)

	Foundations		Concrete Slab-on-grade	
Expansion Index	Depth below finish pad grade (cm)	Steel Reinforcement	Thickness (cm)	Steel Reinforcement
0-50	30	2 - #4 bars, 1 top & 1 bottom	10	6x6-10/10 welded wire mesh or #3 bars @ 61 cm
51-90	46	4 - # 4 bars, 2 top & 2 bottom	10	6x6-10/10 wwm or #3 bars @ 46 cm
91-130	61	4 - #4 bars, 2 top & 2 bottom	10 to 12.5	#3 bars at 46 cm
130+	61 to 91	4 - #5 bars, 2 top & 2 bottom	10 to 15	#4 bars @ 38 cm

Note: Remedial grading operations and/or structurally designed post-tensioned foundation systems are also utilized for soils having an Expansion Index above 130.

mental movements of OC clays; however, no method can completely mitigate the movements that can be generated by expansive soils. Continuing research should provide assistance in assessing the geotechnical properties of OC clays and provide additional methodologies to mitigate the presence of OC clays on slopes and foundations.

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