of Iowa State University. We are also grateful to O. Bailey, state soil scientist of Hawaii, who selected soil series for study and helped with the field work, and to W. Mairaing and M. Lustig for their contribution to the laboratory study.

REFERENCES

1. S. Aomine. Potassium Economy of Japanese Volcanic Ash Soils. Japanese Potassium Symposium, International Potash Institute, Berne, Switzerland, 1957, pp. 88-102.
2. W. L. Badger and J. T. Banchero. Introduction to Chemical Engineering. McGraw-Hill, New York, 1955, 753 pp.
3. K. S. Birrell. Some Physical Properties of New Zealand Volcanic Ash Soils. 1st Annual New Zealand Conference on Soil Mechanics, Melbourne, 1952, pp. 30-34.
4. K. S. Birrell. Some Properties of Volcanic Ash Soils. Meeting on the Classification and Correlation of Soils From Volcanic Ash, Tokyo, U.N. Food and Agriculture Organization and Economic, Scientific, and Cultural Organization, 1964, pp. 74-81.
5. O. Erol, R. A. Lohnes, and T. Demirel. Preparation of Clay-Type Moisture Containing Samples for Scanning Electron Microscopy. Scanning Electron Microscopy, Vol. 1, 1976, pp. 769-776.
6. M. Gradwell and K. S. Birrell. Physical Properties of Certain Volcanic Clays. New Zealand Journal of Science and Technology, Vol. 36B, No. 2, 1954, pp. 108-122.
7. K. B. Hirashima. Highway Experience With Thixotropic Volcanic Clay. Proc., HRB, Vol. 28, 1948, pp. 481-494.
8. K. B. Hirashima. Highway Construction Problems Involving Plastic Volcanic Ash. HRB, Bulletin 44, 1951, pp. 1-9.
9. R. A. Lohnes, E. R. Tuncer, and T. Demirel. Pore Structure of Selected Hawaiian Soils. TRB, Transportation Research Record 612, 1976, pp. 76-79.
10. H. H. Sato, W. Ikeda, R. Paeth, R. Smythe, and M. Takehiro. Soil Survey of the Island of Hawaii, State of Hawaii. U.S. Government Printing Office, 1973.
11. L. D. Swindale and G. D. Sherman. Hawaiian Soils From Volcanic Ash. Meeting on the Classification and Correlation of Soils From Volcanic Ash, Tokyo, U.N. Food and Agriculture Organization and Economic, Scientific, and Cultural Organization, 1964, pp. 36-49.
12. E. R. Tuncer. Engineering Behavior and Classification of Lateritic Soils in Relation to Soil Genesis. PhD thesis, Iowa State Univ., Ames, 1976.
13. E. R. Tuncer, S. Paulson, T. Demirel, and R. A. Lohnes. A Method for Quantitative Determination of Soil Minerals by X-Ray Diffraction. TRB, Transportation Research Record 560, 1976, pp. 75-80.
14. W. W. Wendlandt. Thermal Methods of Analysis. Wiley, New York, 1974.
15. A. C. S. Wright. The Andosols or Humic Allophane Soils of South America. Meeting on the Classification and Correlation of Soils From Volcanic Ash, Tokyo, U.N. Food and Agriculture Organization and Economic, Scientific, and Cultural Organization, 1964, pp. 9-22.
16. T. Yamanouchi. Stabilizing Effects of Additives to Volcanic Ash Soils. Proc., 2nd Asian Regional Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 1, 1963, pp. 359-363.

Clay Structure and Rate Process Theory

O. Erol, T. Demirel, and R. A. Lohnes, Civil Engineering Department and Engineering Research Institute, Iowa State University

An effort was made to incorporate structural variations into rate-process theory by defining a primitive or ideal clay that has a linear strain-time relation implicit in rate-process theory. The slope of the creep curve of the primitive clay is equal to the slope of the tertiary curve at its inflection point. The deviations of the tertiary curve from the straight line are a measure of the behavior of the real clay caused by the structural changes. The ratio of the strain of the real clay to the strain of the primitive clay can be calculated at any point in time. This ratio, defined as the mobilization ratio, is a measure of real-clay structure relative to primitive-clay structure, which remains constant during progressive creep deformations. The method of analysis proposed here reveals that equivalent mobilization ratios correspond to points of identical structure on creep curves. This hypothesis is supported by the experimental results. The analysis of undrained simple shear creep tests conducted on grundite-illite clay at various temperatures and shear stresses demonstrates that the rate process parameters—flow volume, activation enthalpy, and proportionality constant—are structure dependent. The structural changes that accompany creep deformations can be expressed in terms of variations in these parameters.

Although structure is accepted as one of the most important properties influencing the mechanistic behavior of soil, it is at best a descriptive concept manifest in terms such as single grained, massive, aggregated, dispersed, flocculent, and edge-to-face and face-to-face association. The following definitions taken from engineering and pedologic references briefly illustrate the current concepts of structure.

The U.S. Department of Agriculture (USDA) (10) defines soil structure as follows:

Soil structure refers to the aggregation of primary soil particles into compound particles or clusters of primary particles, which are separated from adjoining aggregates by surfaces of weakness.

Although some researchers use the terms structure and fabric interchangeably, Mitchell (7) defines fabric as the arrangement of particles, particle groups, and pore spaces in a soil and states that "... structure is

taken to have the broader meaning of the combined effects of fabric, composition, and interparticle forces." Implicit in the USDA definition (10) is that the aggregates are visible to the unaided eye. Mitchell (7) on the other hand clearly states that his emphasis in soil mechanics is at the microfabric level, which requires at least an optical microscope for study, but that macrofabric is also of great importance.

According to Yong and Warkentin (12),

We define soil structure as that property of soil which provides the integrity of the system and which is responsible for response to externally applied and internally induced sets of forces and fluxes. Soil structure, as a property, includes the gradation and arrangement of soil particles, porosity and pore-size distribution, bonding agents and the specific interactions developed between particles through associated electrical forces.

Hillel (5) states:

Soil structure is generally defined as the mutual arrangement, orientation, and organization of the particles in the soil. The term is also used sometimes with reference to the geometry of the pore spaces. Since the arrangement of soil particles is generally too complex to permit any simple geometric characterization, there is no practical way to measure soil structure directly. Therefore, the concept of soil structure is used in a qualitative sense.

From these references it can be seen that Mitchell (7) and Yong and Warkentin (13) consider that fabric is one aspect of structure whereas Hillel (5) and USDA (10) make no such distinction. Hillel (5) points out the complexity involved in measuring soil structure and that methods for quantitative characterization of structure, such as pore-size distribution, are indirect methods. He further points out that structure is dynamic and changes in response to changes in environment. The very complex geometry of the individual primary particles of clays, the complex geometry of the aggregates, and the changes in structure resulting from changes in stress make the problem of relating the structure of clay-water systems to mechanistic behavior even more formidable.

APPROACH TO THE PROBLEM

Soil structure may be defined, for application in soil mechanics, as the size, the shape, and the arrangement of primary soil particles to form aggregates. The size of the primary particles is referred to as texture; the arrangement of the primary particles may be referred to in granular soils as packing and in clay soils as fabric. In most soils, at stresses commonly encountered in engineering problems, the structure changes that occur in response to changes in stress are changes in fabric. At extremely high stresses, in soils of very low density or high moisture content, or in soils containing easily deformed primary particles, all three aspects of structure may be influenced by changes in stress.

A major difficulty in the quantitative characterization of structure is the lack of a reference state. In this study, therefore, a unique point in the stress-strain history of a soil was selected that could be used to compare other points, each of which represents a different degree of deformation or structure.

Rate-process theory, which is the fundamental theory adopted for the study of the relation of clay-water structure to its mechanistic behavior, has been tested in soil studies for nearly two decades (1, 6, 8, 9). The theory, as adopted by Noble and Demirel (9), states that

$$\dot{\gamma} = Ae^{-\Delta H^*/kT}e^{\beta\tau} \tag{1}$$

where

$\dot{\gamma}$ = shear strain rate,
A = a constant that includes the activation entropy,
ΔH^* = activation enthalpy or bond energy,
β = flow volume that characterizes the mechnical motion of the building blocks of the clay-water system, and
τ = applied shear stress.

This equation does not explicitly include a term that reflects the structure of the material; however, during deformation a rearrangement of either the primary particles or the aggregates is expected to occur. This rearrangement reaction should result in an increase in strength of the bonds (essentially Van der Waal's bonds), the primary particle size, and the order of the system. An increase in flow volume and activation enthalpy and a decrease in activation entropy may therefore be expected. It is obvious that, as a particulate system such as clay-water deforms, there is the potential for change in any or all of these parameters.

Equation 1 implies (a) a linear relation between shear strain (γ) and time (t) and (b) that at constant shear stress (τ) and temperature (T) the flow volume, activation entropy, and activation enthalpy of the system are constant. Many experiments indicate that the shear strain-time curves of soils and clay-water systems are not linear but that strain rate decreases with time (primary or terminal creep) or decreases to an inflection point and then begins to increase (tertiary creep). This behavior suggests that, at constant shear stress, the activation entropy, the activation enthalpy, or the flow volume or any combination of the three may vary.

Equation 1 states that the shear strain rate is proportional to the number of bonding units that have energies equal to or greater than the activation energy. Previous studies have focused on obtaining a single set of thermodynamic parameters characterizing the activated state of clay-water or soil systems; however, in order to determine flow volumes and activation enthalpies, shear strain rates must be compared at points of identical structure. Because the creep curves are not linear, the problem of locating points of identical structure has been approached in two different ways. Some researchers (6, 8) have said that identical soil structure is attained at equivalent time of shear. Noble and Demirel (9) and others maintain that the structure is equivalent at inflection points on the tertiary creep curves.

To incorporate structural variations into rate-process theory, a primitive or ideal clay-water system may be defined by a linear strain-time relation implicit in Equation 1. Figure 1 shows that the slope for the primitive clay is equal to the slope of the tertiary creep curve at its inflection point. Thus, at the point where the second derivative with respect to time is zero, the first derivative is obtained and a straight line at that slope is drawn through the inflection point. This straight line then defines the strain-time behavior of the primitive clay according to the definition. The deviations of the real-clay strain-time curve from that of the straight line (Figure 1) are a measure of the structural changes that accompany the deformation of the real clay. The structure of the real clay at the inflection point is thus the reference structure for the real clay and is equivalent to the structure of the primitive clay, which is constant throughout its deformation history.

The ratio of the strain of the real clay (γ_r) to the strain of the primitive clay (γ_p) can be calculated at any point in time on the strain-time curve of the real clay. This ratio, defined as the mobilization ratio (M), is a structure parameter that measures the departure in the behavior of the real clay from that of the primitive clay.

To compare the strain-time curves of clay samples at various shear stresses, one must be able to assume that the structures of the samples are the same at equivalent mobilization ratios. It is possible, then, to use the mobilization ratio to study the influence of shear stress on the strain-time behavior of clay-water systems at various temperatures; it is also possible to study the variations in flow volumes, activation enthalpies, and activation entropies as shear stress varies.

At a constant shear stress, the mobilization ratio will be fractional at the beginning of a test and increase to a value of one at the inflection point. In the accelerating portion of the tertiary strain-time curve, the mobilization ratio will continue to increase until the soil fails (i.e., the shear stress is relieved).

EXPERIMENTS AND RESULTS

Grundite collected from Morris, Illinois, was used for the study. Raw clay was first sieved through a 74-μm (No. 200) sieve, and the fraction passing was used in the experiments. This clay, which is mainly illite with minor amounts of kaolinite, has the following properties: liquid limit, 58.6 percent; plastic limit, 28.4 percent; and percent finer than 2 μm, 72.2 percent.

The creep experiments were conducted by using a simple shear apparatus developed by Erol and Hartwell (2, 4). Mechanical details of the apparatus are shown

in Figure 2. The simple shear box was designed to test 12.7 × 63.5 × 63.5-mm^3 (0.5 × 2.5 × 2.5-in^3) specimens enclosed in teflon-coated metal walls. Teflon coating was found to be satisfactory in providing lubrication for the specimen-shear box boundaries to minimize load loss in friction. Because there is no change in the area of the specimens as they strain, simple shear testing appears to be most appropriate for correct and simultaneous measurements of stresses and strains. A double-layered, insulated, asbestos housing unit; heating elements; and a solid-state temperature controller [accurate within ±0.5°C (±33°F)] were used to control the temperature of the air surrounding the shear box.

Clay pastes were molded into the shear box by hand and seated by static pressure of 104 kPa (15 lbf/in^2). Normal pressure was kept constant at 41.2 kPa (6 lbf/in^2) throughout the experiments. Moisture content and temperature of the specimens were varied between 33 and 52 percent and 10 and 65°C (50 and 150°F) respectively. Undrained creep experiments were conducted at different levels of shear stresses. The shear deformations and vertical deflections were continuously recorded on strip-chart recorders having a chart speed range of 610 to 152 mm/min (24 to 0.06 in/min). Typical time deformation curves are shown in Figure 3.

Shear strain-time curves were plotted, and the strain

Figure 1. Strain versus time behavior of real clay-water system at constant shear stress and temperature and behavior of ideal or primitive clay.
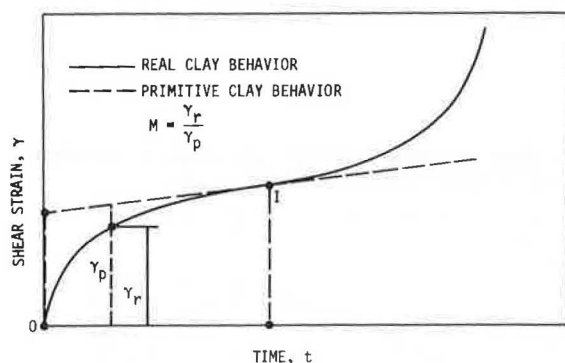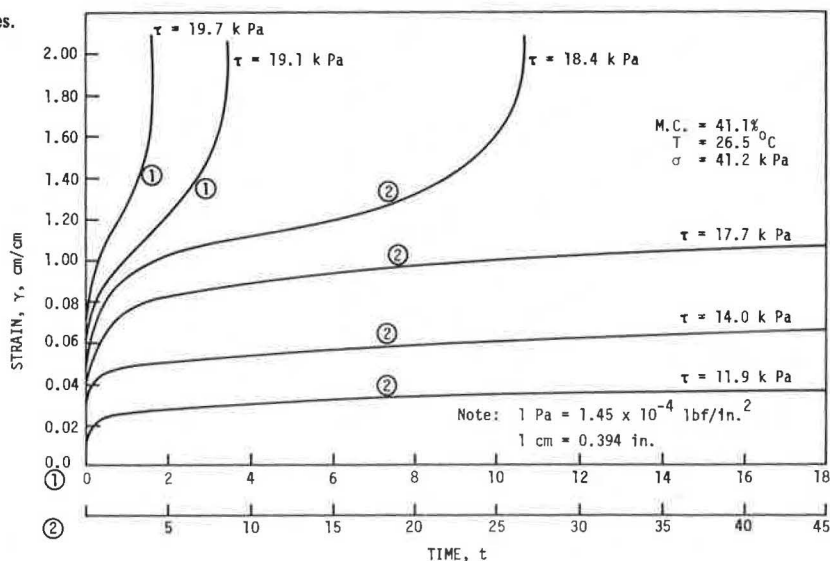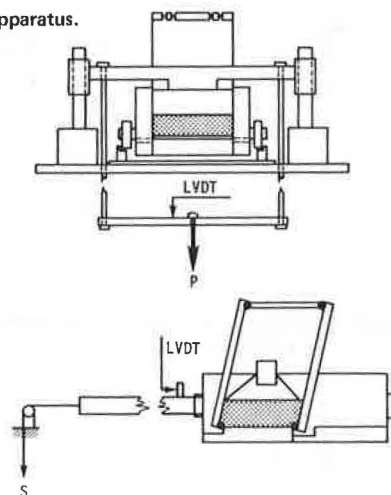

Figure 2. Simple shear apparatus.

Figure 3. Typical strain versus time curves.

rates ($\dot{\gamma}$) at equal mobilization ratios (M) were determined. The log of strain rates was plotted versus shear stress and the reciprocal of temperature, as shown in Figures 4 and 5 respectively. [The slopes of the lines in Figure 4 give flow volumes (β). In Figure 5, the slopes of the lines give activation energies (ΔH^*) and the numerical values correspond to $°K^{-1}$.] The linearity of the plots in these figures supports the validity of the concept of points of equal mobilization ratio being points of equivalent structure.
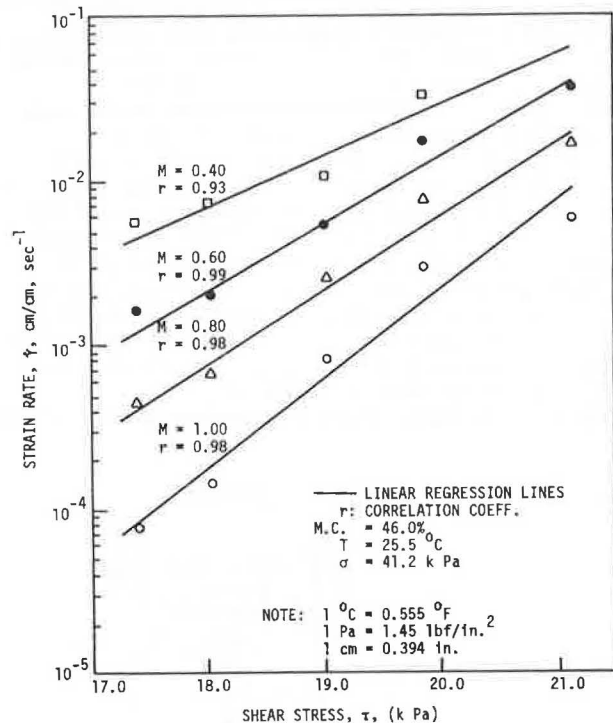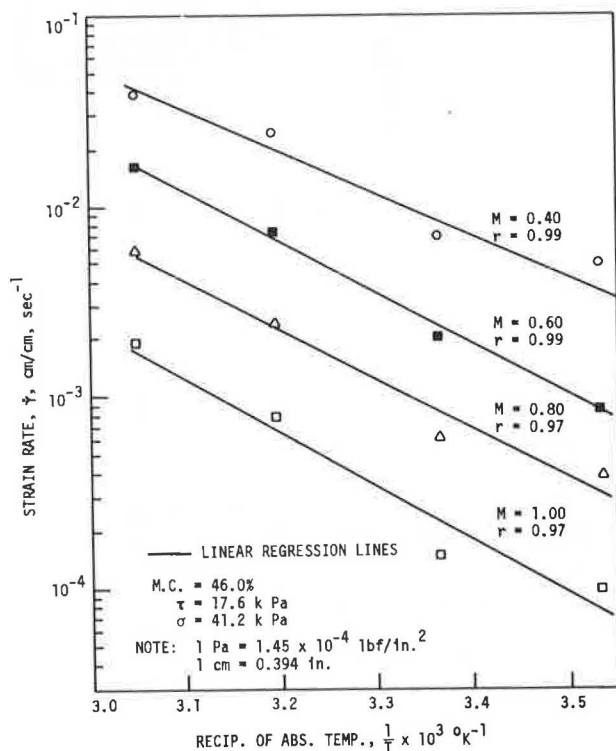
Flow volumes, activation enthalpies, and the proportionality constants that reflect the activation entropy were computed from the slopes and intercepts of Figures 4 and 5 at various mobilization ratios. Figure 6 shows the resulting variation of each of these parameters. The trends in Figure 6 show an increase in flow volume and activation enthalpy and a decrease in the activation entropy up to the mobilization ratio of one (inflection point of the creep curve). The variations in rate-process parameters follow reverse trends in the tertiary region of the creep curves. The parameters thus reach their maximum and minimum values at the inflection point. An increase in flow volume (β) and activation enthalpy (ΔH^*) implies that larger structural units and stronger bonds are formed during the primary (strain-hardening) portion of the creep curves. In the same region, the orderliness of the system increases as indicated by the decrease in the proportionality constant (A). This agrees with direct observations of shear-induced fabrics (3, 11), which showed increasing particle orientations in the shear direction at different stages of shear. The occurrence at the inflection point of the highest values of β and ΔH^* and the lowest value of log A can be interpreted as indicating that the structure achieves its coarsest texture and most strongly bonded and most orderly particle arrangement at the inflection point. The opposite behavior in the tertiary creep zone indicates disintegration of the structural units into smaller units, loosened particle association, and decrease in degree of orderliness of the structure, as implied by the decrease in β and ΔH^* and the increase in log A respectively.

Figure 4. Shear stress versus strain rate at various mobilization ratios.

Figure 5. Reciprocal of absolute temperature versus strain rate at various mobilization ratios.

Figure 6. Mobilization ratio and rupture ratio versus (a) flow volume, (b) activation enthalpy, and (c) log of coefficient A.
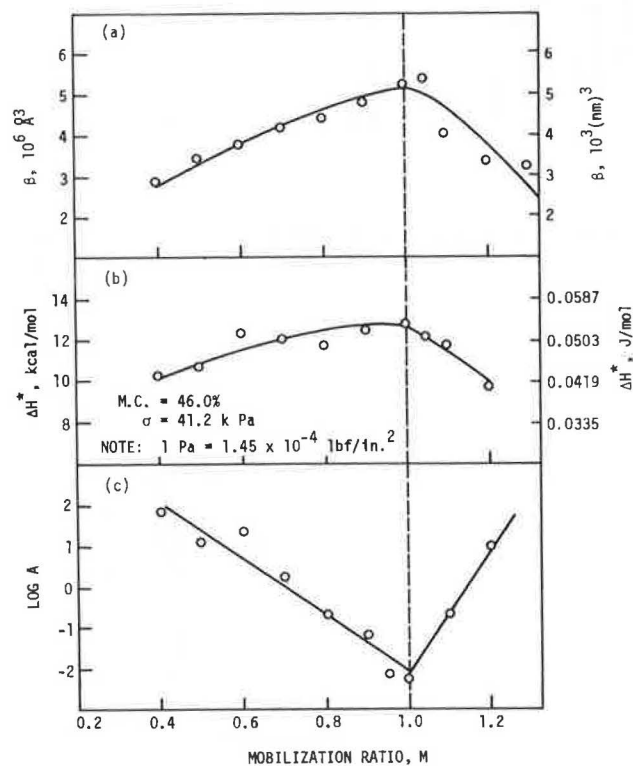
CONCLUSIONS

To incorporate structural variations into rate-process theory, a primitive or ideal clay-water system is defined by a linear strain-time relation implicit in the theory. The deviations of the real-clay strain-time curve from ideal behavior are attributed to the structural changes that accompany the deformation of real clay. To quantify the structural variations a strain ratio, called the mobilization ratio, is defined. When creep curves of clay specimens at various shear stresses or temperatures are compared, the structures of the specimens are identical at equivalent mobilization ratios.

The method of analysis reveals that the rate-process parameters (β), (ΔH^*), and (A) are structure dependent. Because these parameters vary with creep deformations they can be treated as structural parameters to characterize the structural variations. The test results indicate that strain hardening is associated with a rearrangement of particles that tends toward a more ordered system and with a tendency in the primary particles to be welded into larger particles with stronger bonds. The structure attains its coarsest texture and most strongly bonded and most orderly particle association at the inflection point of the creep curves, as indicated by maximum values of β and ΔH^* and minimum value of A.

ACKNOWLEDGMENTS

REFERENCES

1. O. B. Andersland and A. G. Douglas. Soil Deformation Rates and Activation Energies. Geotechnique, Vol. 20, 1970, pp. 1-16.
2. O. Erol. Clay Structure and Creep Behavior of Clays as a Rate Process. PhD thesis, Iowa State Univ., Ames, 1977, 118 pp.
3. R. H. Foster and P. K. De. Optical and Electron Microscopic Investigation of Shear Induced Structures in Lightly and Heavily Consolidated Kaolinite. Clays and Clay Minerals, Vol. 19, No. 1, 1971, pp. 31-48.
4. J. F. Hartwell. Development of a Simple Shear Apparatus for Rate Process Studies. MS thesis, Iowa State Univ., Ames, 1974, 92 pp.
5. D. Hillel. Soil and Water. Academic Press, New York, 1971, 288 pp.
6. J. K. Mitchell. Shearing Resistance of Soils as a Rate Process. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 90, 1964, pp. 29-61.
7. J. K. Mitchell. Fundamentals of Soil Behavior. Wiley, New York, 1976, 442 pp.
8. J. K. Mitchell, R. G. Campanella, and A. Singh. Soil Creep as a Rate Process. Proc., ASCE, Vol. 94, No. SM1, 1968, pp. 231-253.
9. C. N. Noble and T. Demirel. Effect of Temperature on Strength Behavior of Cohesive Soil. HRB, Special Rept. 103, 1969, pp. 204-220.
10. Soil Survey Manual. U.S. Department of Agriculture, Handbook 18, U.S. Government Printing Office, 1951, 501 pp.
11. S. S. Vyalov, N. K. Pekarskaya, and R. V. Maksimyak. Physical Essence of Processes of Deformation and Failure of Clayey Soils. Soil Mechanics and Foundation Engineering, Vol. 1, 1970, pp. 12-16.
12. R. N. Yong and B. P. Warkentin. Soil Properties and Behavior. Elsevier-North Holland, Amsterdam, Netherlands, 1975, 449 pp.

Sampling a Glacial Silty Clay

J. H. Lamb, Department of Civil Engineering, Wayne State University
J. M. Ritchie, Michigan Department of State Highways and Transportation

Over a period of 10 years a total of 42 borings were made in a glaciolacustrine deposit on the western boundary of Detroit, Michigan. The soil profile consisted of yellow-brown mottled silty clay underlain by gray silty clay. The undrained shear strength, moisture content, and dry density of 329 soil specimens of the gray silty clay are statistically analyzed. The variables are randomly distributed in both the vertical and the lateral directions. The lognormal distribution is the most likely fit for the data but, on engineering grounds, the normal distribution is preferred. Statistical estimation theory indicates that as few as 5 borings arranged in an X pattern and containing at least 30 specimens could adequately estimate the values for design at this site.

Any program to determine the properties of a natural soil deposit requires answering the following questions:

1. How many borings should be made?
2. Where should the borings be located?
3. How many field tests or laboratory tests or both should be performed?

Once these questions are answered, usually somewhat arbitrarily, it is still necessary to adopt a method for calculating the value of the design parameter. For example, a procedure that has been recommended for analysis of bearing capacity in clays is to take several borings in the area of the footings, average the values in each boring within the significant depth, and take the minimum boring average divided by the factor of safety for design (4). Although this procedure is generally effective, the actual factor of safety is not known. Because the various properties of natural soil deposits behave like random variables, their variations can be analyzed by statistical methods. Such methods make it possible to answer the above questions systematically.