

Understanding-Soil-Behavior Runs Through It

Understanding what soils are and the principles that govern their geotechnical properties are essential to sound geotechnical engineering practice.

JAMES K. MITCHELL

Geotechnical engineering is a continually evolving discipline, with shifting emphases and new challenges as needs and priorities change in the world around us. Although the focus of today's practice is on problems that are far different from those of 50 years ago, and the concerns of the 21st century can be expected to be much different from those of today, the key to their solution has depended, and will continue to depend, on the proper understanding of soil and rock properties and behavior when subjected to the many forces of nature as well as the activities of humankind.

Before examining specific considerations of some special and unique aspects of soil behavior, however, it is instructive to review very briefly how the geotechnical field has devel-

oped and how problems have changed over the past 50 years to illustrate the extensive breadth and wide diversity of problems in geotechnology and how new subdisciplines have emerged as new problems have arisen.

The Changing Focus of Geotechnical Engineering

The concerns, challenges and developments within geotechnical engineering over the past 50 years, as viewed within five- or 10-year periods, might look something like the following, which has been prepared with full realization that important topics may have been omitted and that there may be some disagreement over some of the selections and dates indicated:

1940–1950: Soil mechanics comes of age through the efforts of the early giants of the field such as Terzaghi, Casagrande, Peck, Taylor, and Skempton; each of whom built the mechanics on a carefully developed understanding of the properties and behavior of real soils.

1950–1960: Shear strength, bearing capacity, slope stability, soil structure and sensitivity, compacted clay, repeated loading, pavement design, soil stabilization, transient loading and soil dynamics.

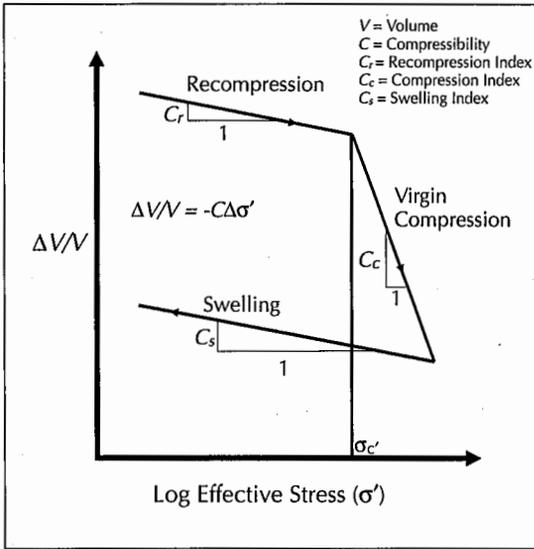


FIGURE 1. Idealized relationship for volume change (inert soil model).

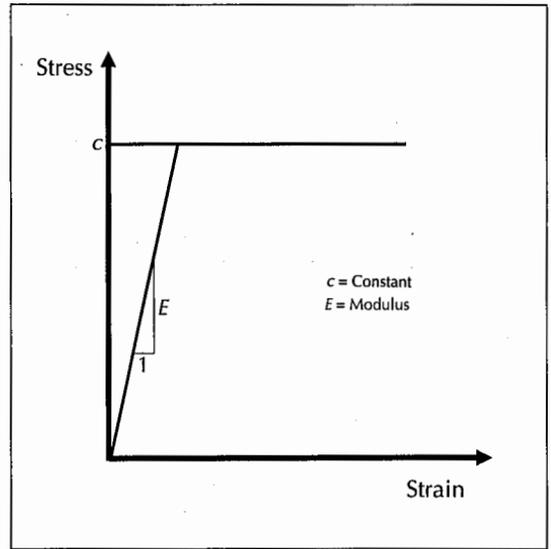


FIGURE 2. Idealized relationship for stress-strain (inert soil model).

1960–1965: Pore pressures and effective stress analysis, physico-chemical aspects of soil behavior, early development of rock mechanics, increased use of computer applications and introduction of the finite element method for the analysis of soil deformation and soil-structure interaction problems, Anchorage and Niigata earthquakes spawn earthquake geotechnical engineering.

1965–1970: Geotechnical engineering for offshore structures, nuclear power plants, lunar soil mechanics, soil-structure interaction.

1970–1975: Computer applications, soil dynamics, ground improvement.

1975–1980: In-situ measurement of soil properties, constitutive modeling, centrifuge testing, re-recognition of the importance of properties.

1980–1985: Earth reinforcement, geotechnical earthquake engineering, beginnings of geoenvironmental engineering, geosynthetics, ground improvement, risk and reliability.

1985–1990: Waste containment, landfills, site remediation, seismic risk mitigation, geosynthetics.

1990–1995: Geoenvironmental engineering, risk mitigation, infrastructure, properties for numerical analysis, land reclamation.

It seems that as new problems emerge (some of them unfortunately as a result of failures of one kind or another), as the need to treat and improve soils to make them suitable as foundation materials or for construction increases, and as we seek better and more efficient ways to predict future performance, attention keeps returning to basic questions of what soil is and why it responds the way it does.

Soils as Environmentally Sensitive Materials

If soils were inert and insensitive to their composition and the environment in which they exist, it is likely that designs, construction and expectations for future performance could be reduced to sets of rules, charts and graphs that could quickly transform what we do from a profession to a trade. Fortunately, or unfortunately, depending on one's perspective, soils are not inert materials, and their properties are very sensitive to and dependent on the environment (temperature, pressure, and chemical and biological regimes) in which they exist. Furthermore, the possible ranges and variations in particle size, composition, size distribution, void ratios, saturations, pressures, temperatures, and chemical and biological environments means that we are confronted with an almost infinite range of materials, each

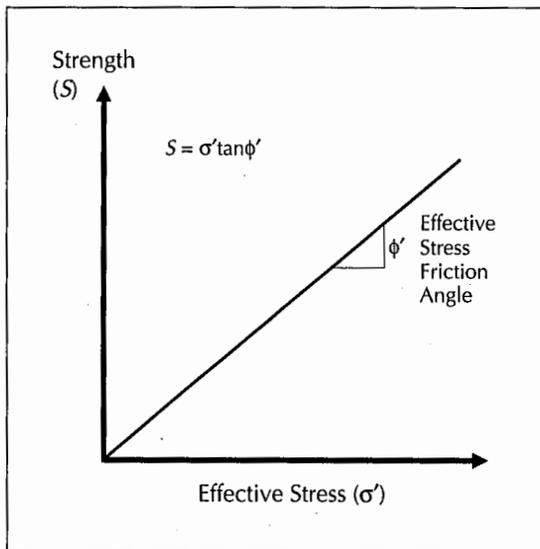


FIGURE 3. Idealized relationship for strength (inert soil model).

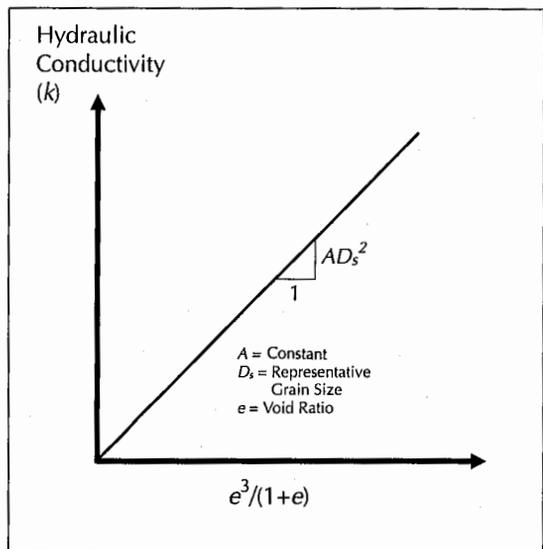


FIGURE 4. Idealized relationship for hydraulic conductivity (inert soil model).

with its own unique set of mechanical properties.

The following soil factors are of most importance in any project:

- Volume change characteristics;
- Stress-strain response;
- Strength, stress-strain-time properties;
- Conduction properties (hydraulic, thermal, chemical and electrical);
- Chemical and biological conditions; and,
- The variations of these properties with time and changes in temperature and pressure.

Any single property can usually vary over a wide range, depending on how it is measured and defined. Therefore, careful consideration of testing conditions and analytical modeling in relation to the particular problem or application of interest is also necessary.

Inert & Real Soil Models. If soil particles were inert and unaffected by physico-chemical interactions with the environment and other particles around them, then analyses and designs could be performed using rather simple time-independent relationships that describe properties over a wide range of conditions. For example, an idealized relationship between volume change and effective stress is shown in

Figure 1, between deviatoric stress and strain in Figure 2, between effective stress and strength in Figure 3, and between hydraulic conductivity and void ratio in Figure 4. In these relationships the effective stress, σ' , for saturated soils is defined by the difference between the total stress, σ , and the pore water pressure, u . To account for differences in grain size, grain shape and grain size distribution among different soils, families of curves of the type shown in Figures 1 through 4 might be required.

The closest approximation to an inert real soil that has been found to date is perhaps lunar soil.¹ This material, which has a gradation typical of terrestrial silty fine sands, has mechanical properties that are readily explainable and quantifiable in terms of void ratio and confining pressure. Differences in strength and compressibility among samples from different locations are explainable in terms of gradation, particle size, particle shape and particle composition (which controls size, shape and crushability). Since there is neither water nor significant gas in the lunar environment, there are no adjacent phases for the soil particles to interact with — hence, their inertness (except when in close contact with each other, which may result in small amounts of cohesion).

The actual relationships describing the mechanical properties of a terrestrial soil are rarely

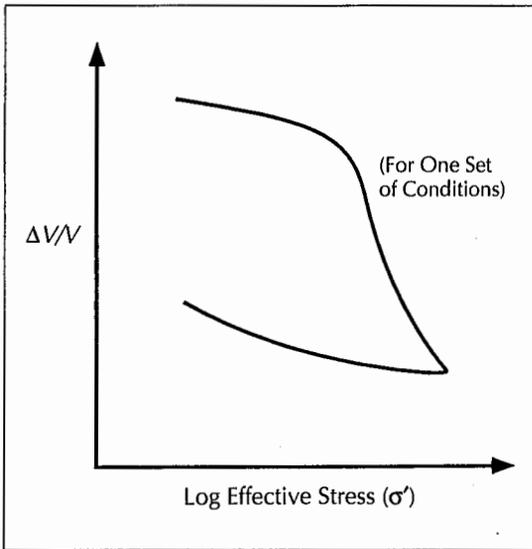


FIGURE 5. Idealized relationship for volume change (real soil).

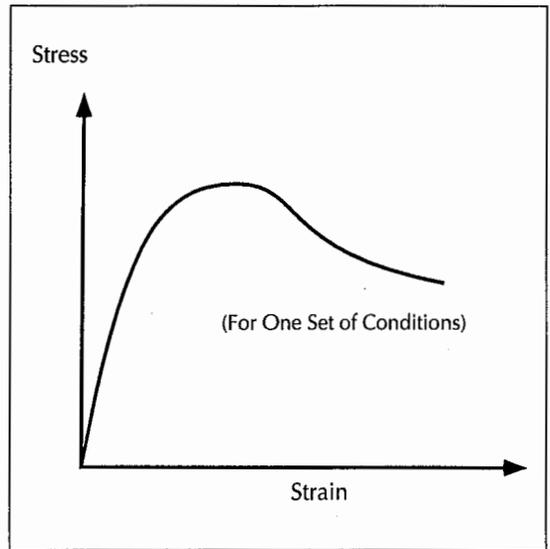


FIGURE 6. Idealized relationship for stress-strain (real soil).

as simple as those shown in Figures 1 through 4. More realistic representations for volume change, stress-strain, strength and hydraulic conductivity are shown in Figures 5 through 8. Parameters C , ϕ' , E and A are seldom constants, although for engineering purposes they are often assumed to be so. Furthermore, the volume changes and deformations are usually time dependent, as a result of pore pressure

adjustments and the viscous behavior of most soil structures. Fortunately, however, the variations are often at least understandable, if not quantitatively predictable, and the attempt to develop general constitutive models for behavior has occupied the attention of numerous researchers for many years.

Some Special Soil Behavior Phenomena. Not only does the mechanical behavior of real soils

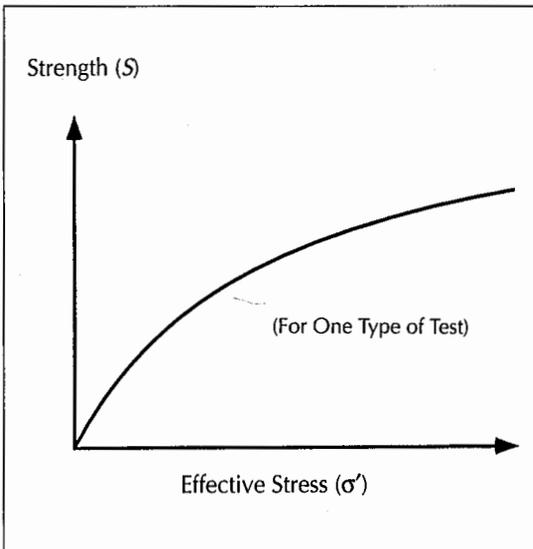


FIGURE 7. Idealized relationship for strength (real soil).

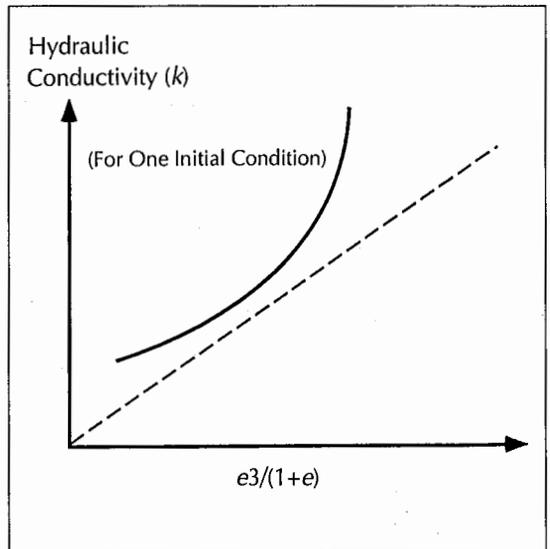


FIGURE 8. Idealized relationship for hydraulic conductivity (real soil).

deviate from that of ideal inert soils in the ways shown above, but there are numerous other phenomena that may be important, depending on the nature of a project or problem. Understanding them is the first step in developing suitable strategies for dealing with them. Some of these phenomena include:

Swelling Soils. Expansive soils are ubiquitous in many parts of the world, especially in semi-arid climates and where smectite group clay minerals are found. Figure 9 shows the swelling behavior of a clay soil compacted at the same water content and dry density by two different methods, thus producing two different initial structures. The magnitude of swell is significantly influenced by the structural difference. The figure shows also that the amount of swell decreases as the electrolyte concentration of the absorbed water increases. In addition, the data in Figure 10 indicate that the amount of swell is influenced by the stress path followed during unloading.

Collapsing Soils. Large areas of the earth's surface are covered by soils that are susceptible to large decreases in bulk volume when

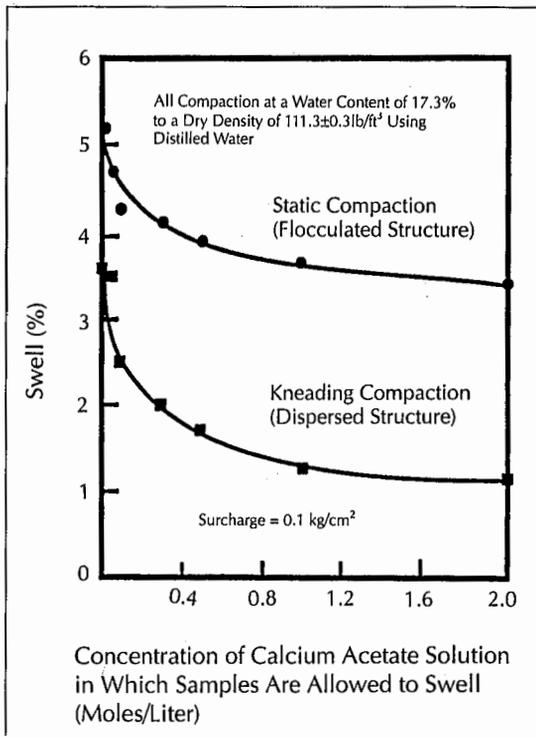


FIGURE 9. The effect of structure and electrolyte concentration of absorbed solution on the swell of sandy clay compacted by two methods.

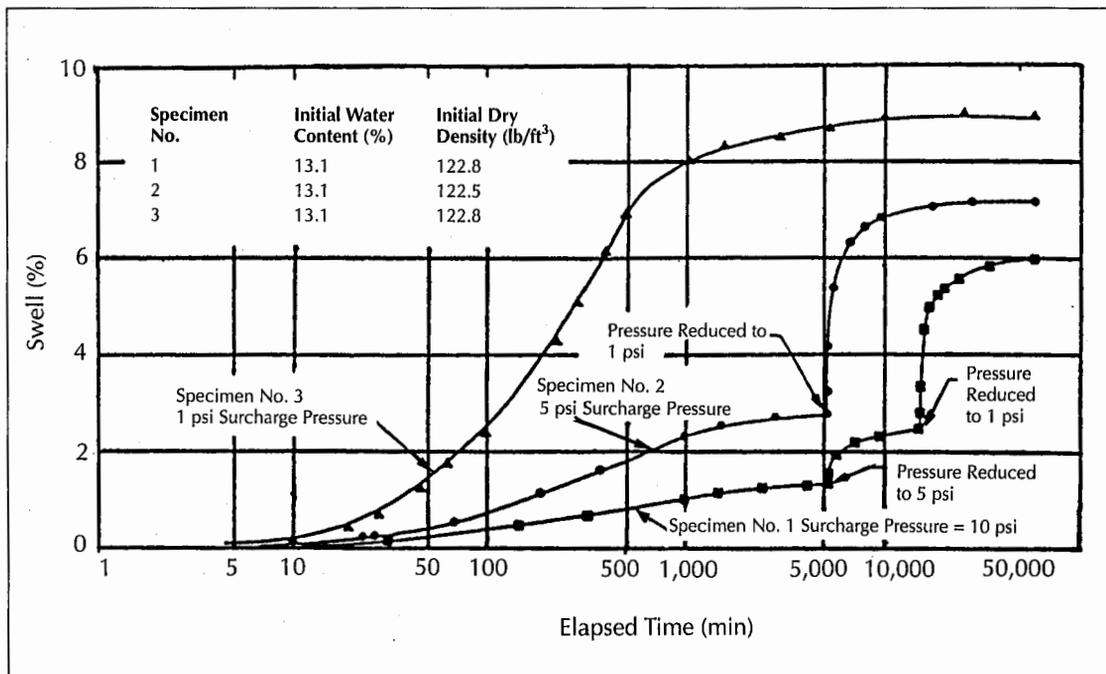


FIGURE 10. The effect of unloading stress path on the swelling of a compacted sandy clay.

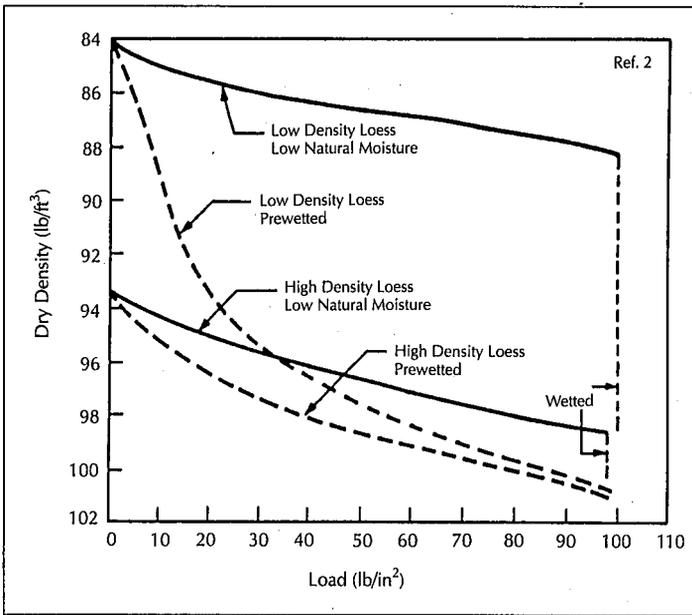


FIGURE 11. Collapse properties of Missouri River basin loess.

they become saturated. Collapse may be triggered by water alone or by saturation and loading acting together. Soils with collapsible grain structures may be residual, water deposited or aeolian. Settlement under increased loading may be small when the soil is at its natural moisture content, but after wetting it may be very large, as shown in Figure 11.

Dispersive Clays. Soils in which the clay particles will detach spontaneously from each other and from the soil structure and go into suspension in quiet water are termed dispersive clays. The consequences of the exposure of dispersive clays to water may include severe surface erosion and the formation of "badlands" topography and the formation of internal erosion tunnels in dams and dikes.

Slaking. Most intact fine-grained soils slake after exposure to air and subsequent immersion unconfined in water — *i.e.*, an initially intact piece of soil will disintegrate into a pile of pieces of sediment of small particles. This disintegration may begin immediately upon immersion or develop slowly with time. Slaking is usually more rapid and vigorous in materials that have been dried prior to immersion as compared to the same material immersed at its initial water content. The slaking of hard clays and clay shales is a concern in the stability of open excavations and the durability of shale is a concern when used as an aggregate or rockfill for construction.

Sensitive & Quick Clays. Sensitivity refers to the loss of

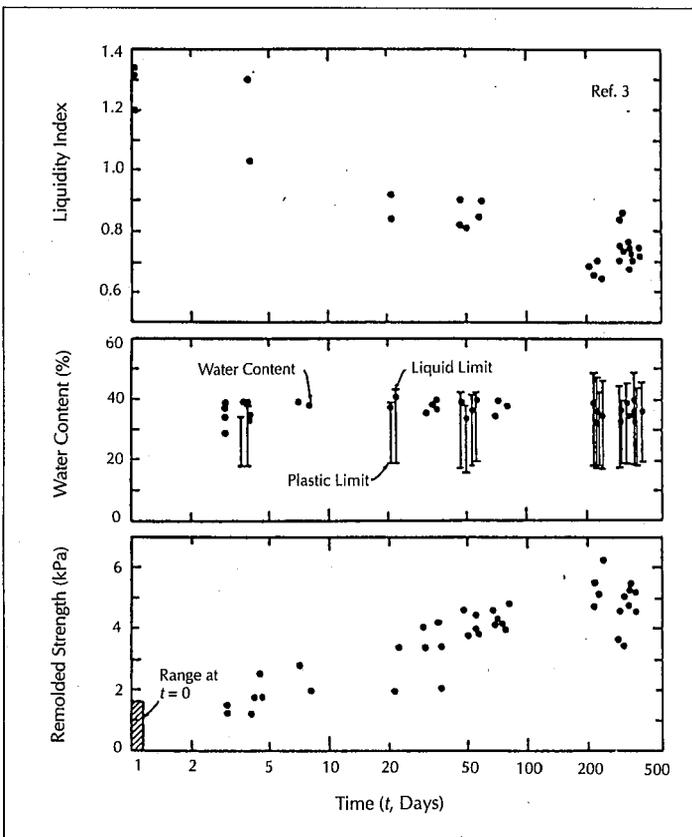


FIGURE 12. Changes in remolded strength and consistency of a Canadian quick clay as a function of time.

strength of an undisturbed clay when it is disturbed and remolded at constant water content. The strength loss may be so great that it will cause the remolded material to behave essentially as a fluid, in which case it is termed a *quick clay*. Early attempts to explain clay sensitivity led to the intensive study of physico-chemical phenomena in fine-grained soils by geotechnical engineers.

Aging of Quick Clays. Significant changes in the properties of quick clays can develop with time after sampling, including an increase in the remolded strength and liquid limit, and a decrease in the liquidity index without a change in water content. The changes in a remolded quick clay from Outardes-2 in Québec over a one-year period are illustrated in Figure 12. When changes such as these occur, the reliability of data obtained from tests on samples that have been stored for more than a few days after removal from the ground becomes questionable unless measures are taken to block the transformations responsible for them.

Thixotropic Hardening. Thixotropy is an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquefies upon remolding. The properties of a purely thixotropic material are shown in Figure 13. Thixotropic hardening may account for low to medium sensitivity and for a part of the sensitivity of quick clays. It can be responsible for time-dependent increases in the strength and stiffness of compacted fine-grained soils. It may be important in influencing the flow properties of drilling muds and slurry wall materials.

Aging of Sands. Many sand deposits and sand fills exhibit "aging" effects wherein their strength and stiffness increase noticeably within periods of weeks to months after deposition, disturbance or densification. Il-

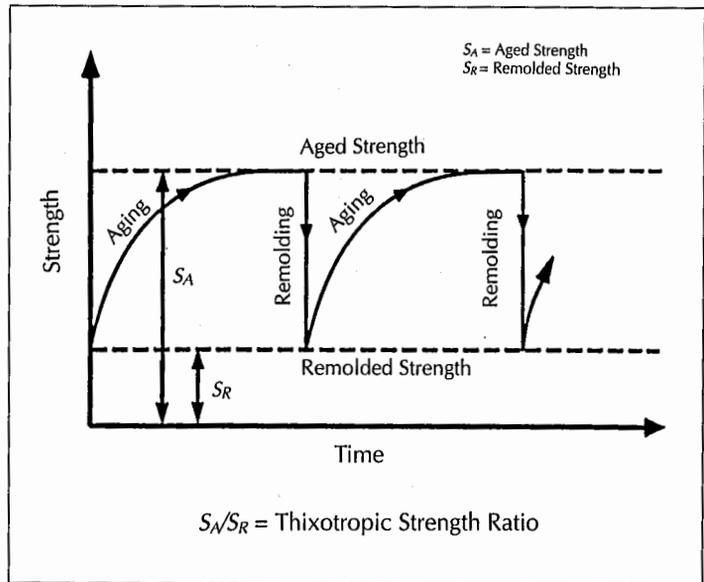


FIGURE 13. Properties of a purely thixotropic material.

lustrations of the effect of time on the penetration resistance of a hydraulic sand fill are shown in Figure 14, and of time on the penetration resistance of an in-situ sand after densification by blasting in Figure 15.

Creep & Stress Relaxation. Soils exhibit both creep and stress relaxation, as shown schematically in Figure 16. The magnitude of these effects increases with the plasticity, activity and water content of the soil. The form of behavior is essentially the same for all soils. Because of the viscous contribution to soil deformation, soil strength is influenced by rate of shear, as indicated by the data in Figure 17.

Coupled Flows. Fluids, electricity, heat and chemicals flow through soils. It has been well established that provided the flow process does not change the state of the soil, each flow rate, or flux, is related linearly to the driving force, or gradient, as shown in Figure 18. In addition, in most soils there are simultaneous flows of different types, even when only one type of driving force is acting. Thus, a driving force of one type can cause a flow of another type. A familiar example is electro-osmosis wherein a DC electrical gradient causes a hydraulic flow. Such flows are termed coupled flows. The different types of direct and coupled flows are listed in Table 1.

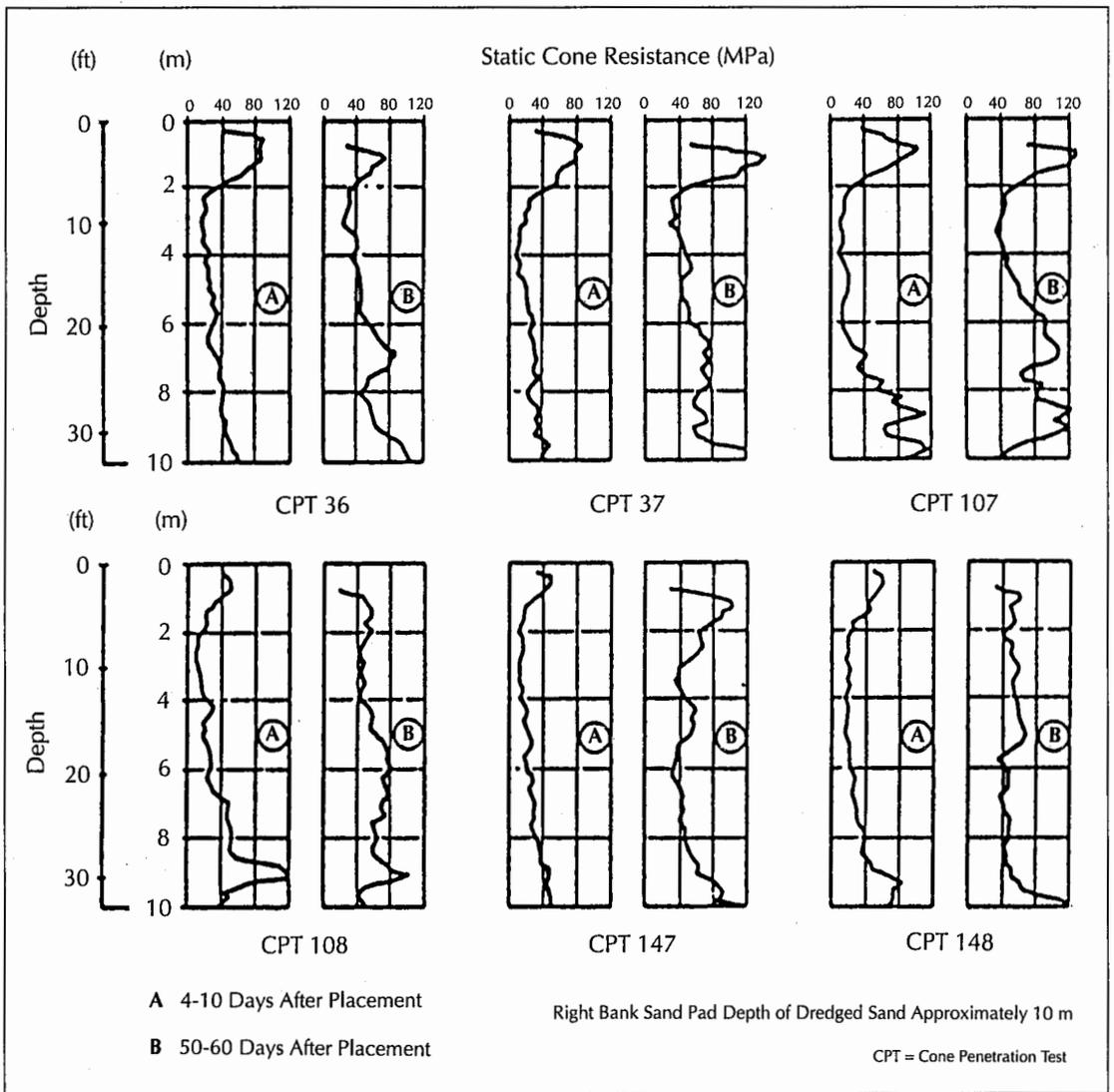


FIGURE 14. Penetration resistance profiles for a hydraulic fill at two times after placement.

Direct and coupled flows — especially advection, diffusion and electro-osmosis — are of particular current interest owing to their importance in geoenvironmental engineering problems.

Developing an Understanding of Soil Behavior

Phenomena of the type described in the preceding section cannot be explained in terms of an inert soil model. Rather, it is necessary to investigate in greater detail:

- Soil grain composition;

- The morphological characteristics of the particles; and,
- How the solid particles interact with the water and chemical environment around them, as well as with each other.

A detailed examination of these topics can be found in Mitchell.⁵

The engineering properties of any soil, and the special phenomena described above, depend on the composite effects of several interacting factors. These factors may be divided into two groups: *compositional factors* and *environmental factors*.

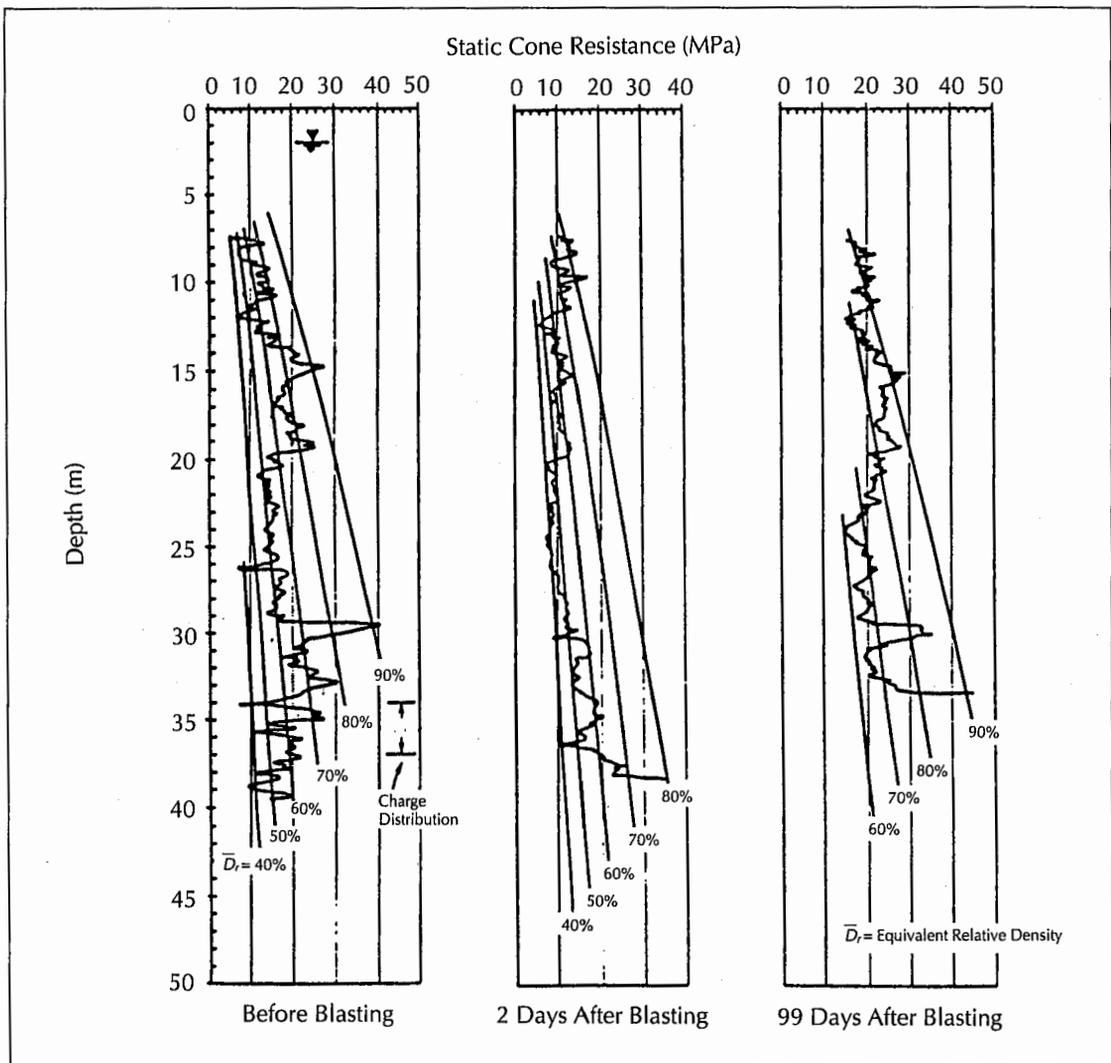


FIGURE 15. Cone penetration test records illustrating penetration resistance increase with time after blasting densification of clean sand at the Jebba Dam, Nigeria.

Compositional factors determine the potential range of values for any property and the extent to which any of the special phenomena could occur. Compositional factors can be studied using disturbed samples. Compositional factors include:

- Type of minerals;
- Amount of each mineral;
- Adsorbed cation types and amounts;
- Particle size, shape and size distribution;
- Pore water composition;
- Free and dissolved gases; and,
- Biological regime.

Environmental factors determine the actual value of any property. Undisturbed samples or in-situ measurements are required for the study of the influences of environmental factors on properties and behavior. Environmental factors include:

- Water content;
- Density;
- Confining pressure;
- Temperature;
- Fabric;
- Availability of water; and,
- Stress history.

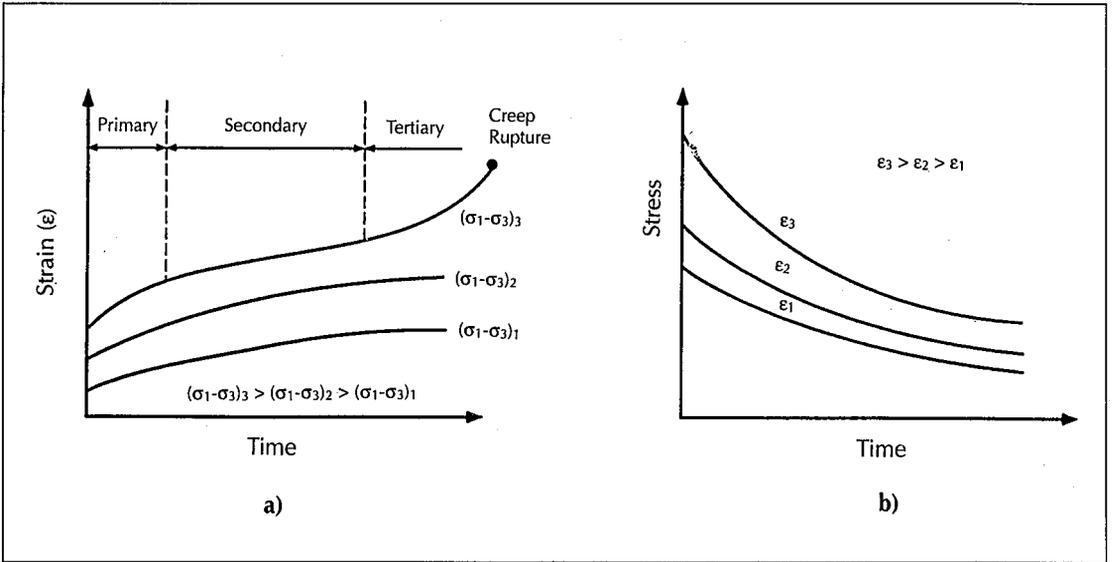


FIGURE 16. Creep and stress relaxation: a) creep under constant stress; and b) stress relaxation under constant strain.

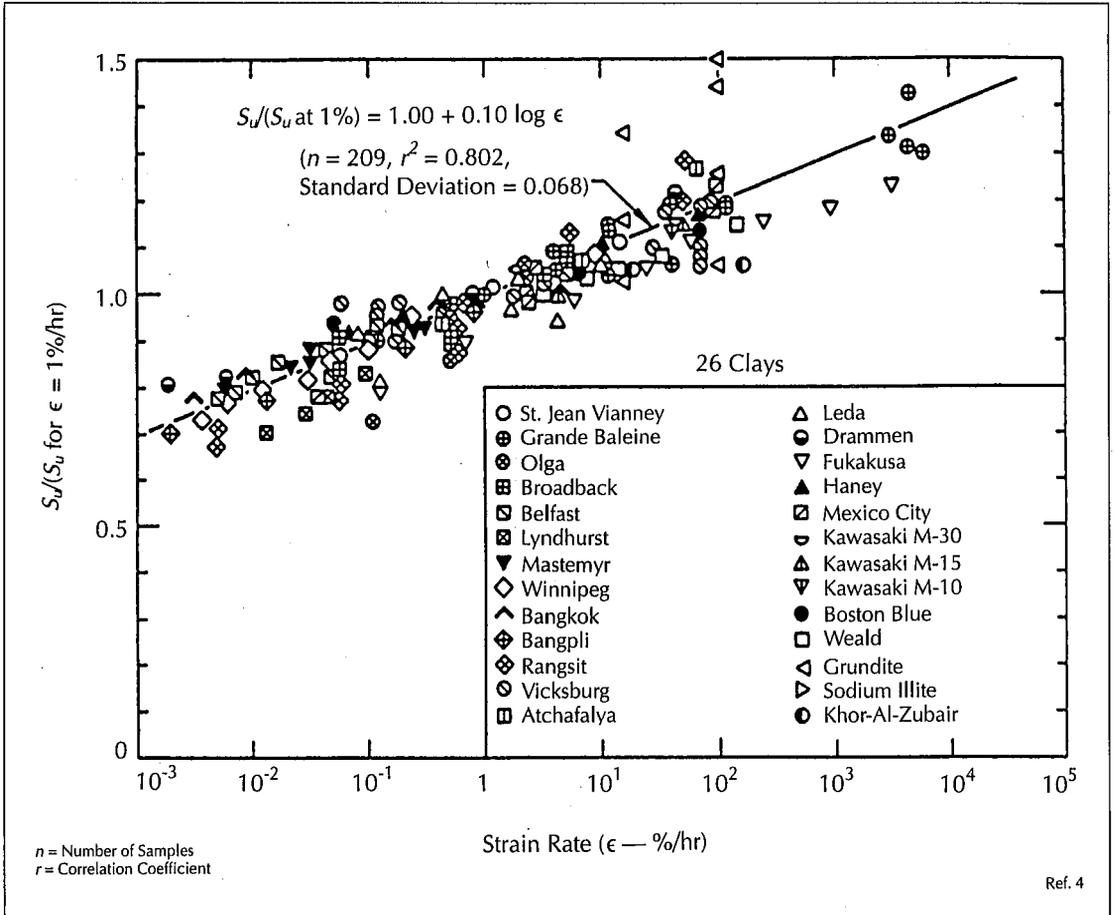
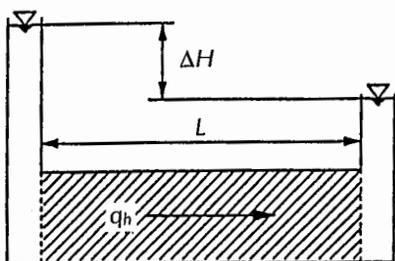


FIGURE 17. Effect of strain rate on undrained strength.

TABLE 1
Direct & Coupled Flow Phenomena

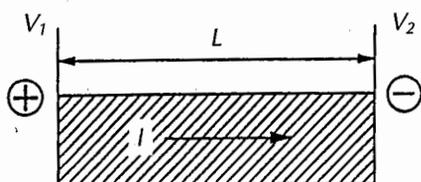
| | Gradient (X) | | | |
|----------|------------------------------------|---|---------------------------------|---------------------------------|
| Flow (J) | Hydraulic Head | Temperature | Electrical | Chemical Concentration |
| Fluid | Hydraulic Conduction (Darcy's Law) | Thermo-osmosis | Electro-osmosis | Chemical Osmosis |
| Heat | Isothermal Heat Transfer | Thermal Conduction (Fourier's Law) | Peltier Effect | Dufour Effect |
| Current | Streaming Current | Thermo-electricity (Seebeck Effect) | Electric Conduction (Ohm's Law) | Diffusion & Membrane Potentials |
| Ion | Streaming Current | Thermal Diffusion of Electrolyte (Soret Effect) | Electrophoresis | Diffusion (Fick's Law) |



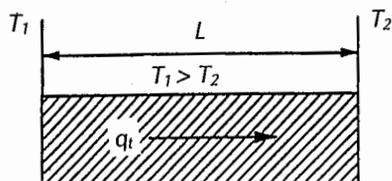
Fluid
 $q_h = k_h(\Delta H/L)A$
 Darcy's Law

In the relationships shown in this figure:

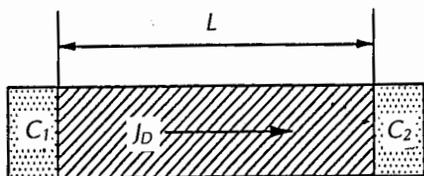
- σ_e = Electrical Conductivity
- q_s = Fluid Flow Rate
- I = Electrical Current
- q_t = Heat Flow Rate
- J_D = Chemical Flow Rate by Diffusion
- A = Cross-Sectional Area
- L = Length
- H = Head
- V = Voltage
- T = Temperature
- c = Concentration
- k_h = Hydraulic Conductivity
- k_e = Thermal Conductivity
- D = Diffusion Coefficient



Electricity
 $I = \sigma_e(\Delta V/L)A$
 Ohm's Law



Heat
 $q_t = k_t(\Delta T/L)A$
 Fourier's Law



Chemicals
 $J_D = D(\Delta c/L)A$
 Fick's Law

FIGURE 18. Four types of direct flows in soils.

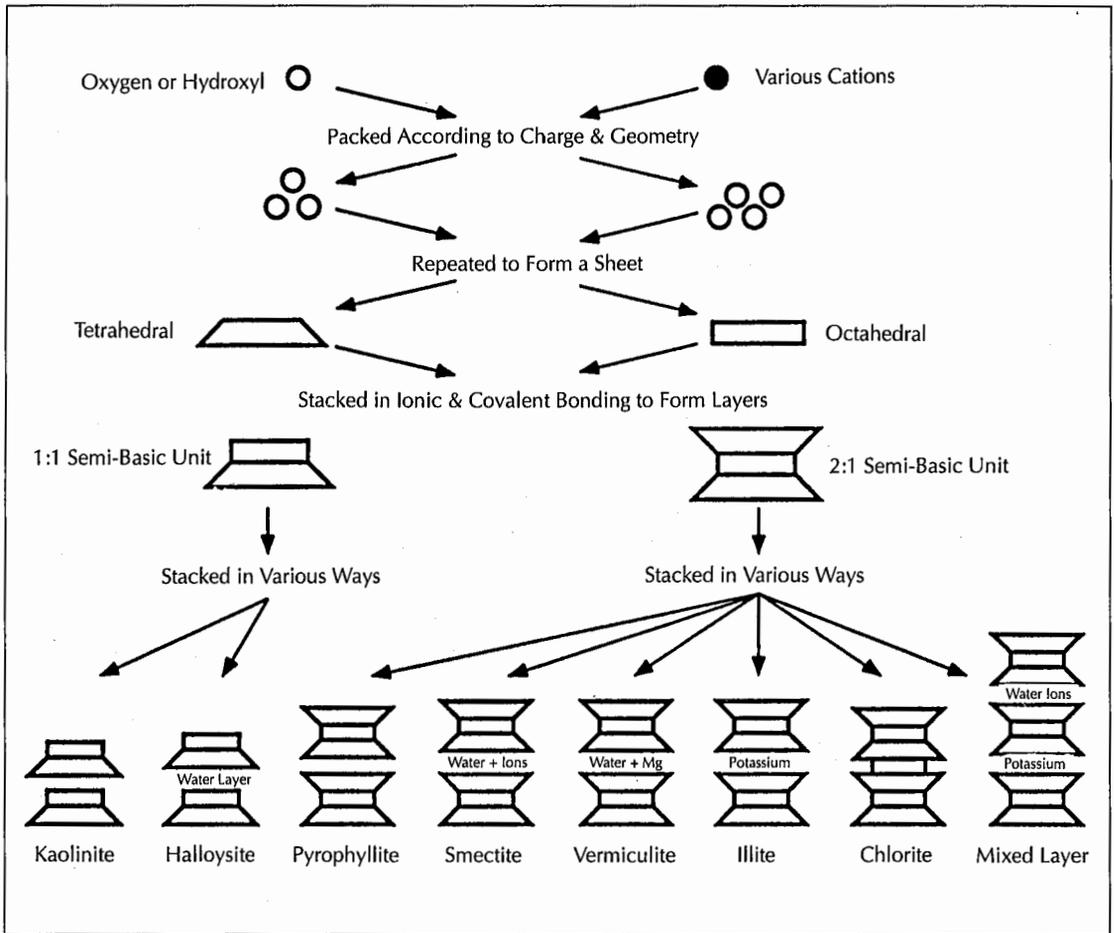


FIGURE 19. The basic structure of the clay mineral groups.

Soil mineralogy is fundamental in that it determines particle size, shape, surface area and the interface properties. Because the clay minerals are very small and they have a platy, tubular or lath-like shape, they have a very large specific surface area, ranging upwards to 800 m²/gm. The general structures for the important clay minerals are shown schematically in Figure 19.

The surface layers of the clay minerals and most of the non-clay silicate minerals are composed of oxygen atoms in tetrahedral coordination held together by silicons. In the kaolin clays one of the surface layers of the platy particles is composed of hydroxyls. The surface oxygens and hydroxyls can be attracted simultaneously to silicons or other cations within the mineral particles or to hydrogens in the adjacent water phase. The resultant hydrogen

bonding means that water is strongly attracted to soil mineral surfaces.

In addition, the mineral structures of the clay minerals are characterized by substantial isomorphous substitution, the end result of which is that the particles have a net negative electrical charge. As a result, cations are attracted to provide electrical neutrality. The quantitative expression of this electro-negativity and the amount of balancing cations is the cation exchange capacity (usually expressed in milliequivalents per hundred grams of dry clay). Important aspects of these conditions are that:

- The balancing cations can be of different types and valences;
- Cations of one type can be replaced by cations of another type; and,

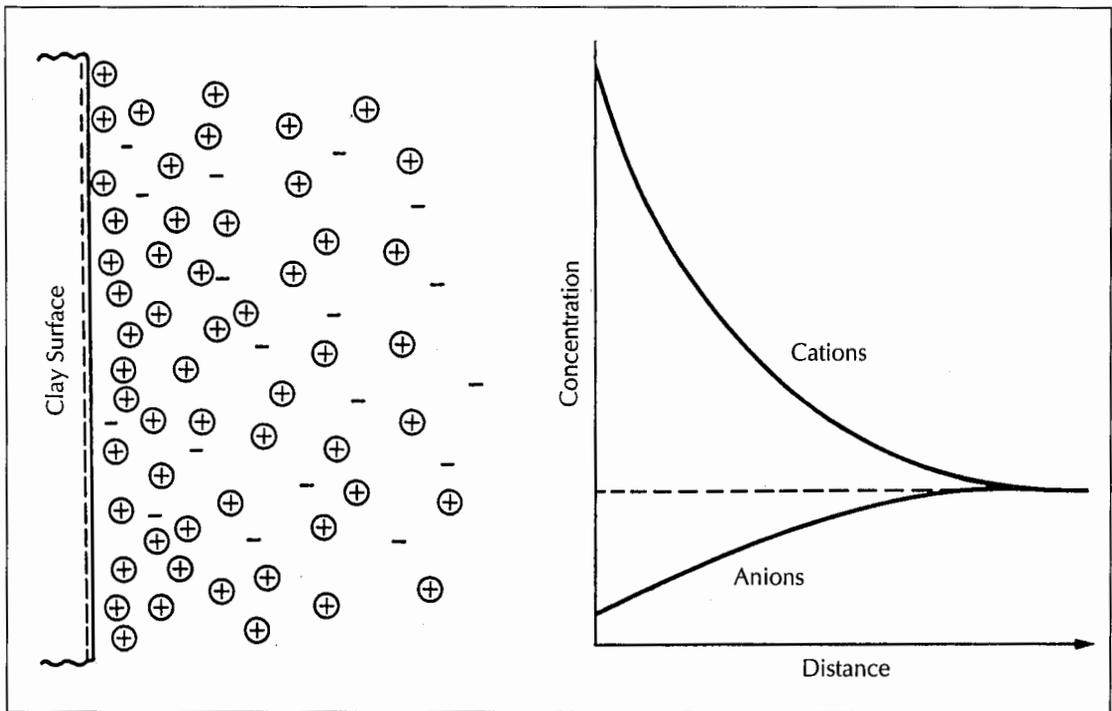


FIGURE 20. Distributions of ions adjacent to a clay surface according to the concept of the diffuse double layer.

- Cations of different types induce different properties.

Furthermore, in the presence of water, adsorbed cations dissociate from clay particle surfaces and form the diffuse part of a double layer of negative and positive charges that extends outward from particle surfaces, as shown in Figure 20.

Owing to their very small size and high specific surface area, clay particles behave as colloids, and many principles from colloid chemistry apply. The thickness and characteristics of this diffuse layer depend on the composition of the fluid phase as well as on the composition and structure of the mineral particles. Thick diffuse layers — which are associated with low cation valence, low electrolyte concentrations and pore fluids of high polarity and dielectric constant — produce strong repulsions between particles; whereas, high cation valence, high electrolyte concentration as well as low pore fluid polarity and dielectric constant are associated with small interparticle repulsions. The magnitude of the interparticle double layer repulsions relative to interparticle

attractive forces (van der Waal's and electrostatic attractive forces) determines whether particles in suspension will flocculate or deflocculate. A wide range of particle associations is possible and the influences of different particle associations on the mechanical properties of a soil can be very large.

In summary, the small particle sizes, the high specific surface area, the attraction of water to soil particle surfaces and the net electrical negativity that leads to the formation of diffuse double layers sensitive to the surrounding environmental conditions mean that:

- Many different particle arrangements are possible for a soil at a given void ratio.
- Different particle arrangements at the same void ratio mean different effective stresses, different mechanical properties and different structural stability.
- Changed environmental conditions may or may not result in changes in the particle associations in a soil mass, depending on how tightly held the particles are. Subsequent mechanical disturbance of a

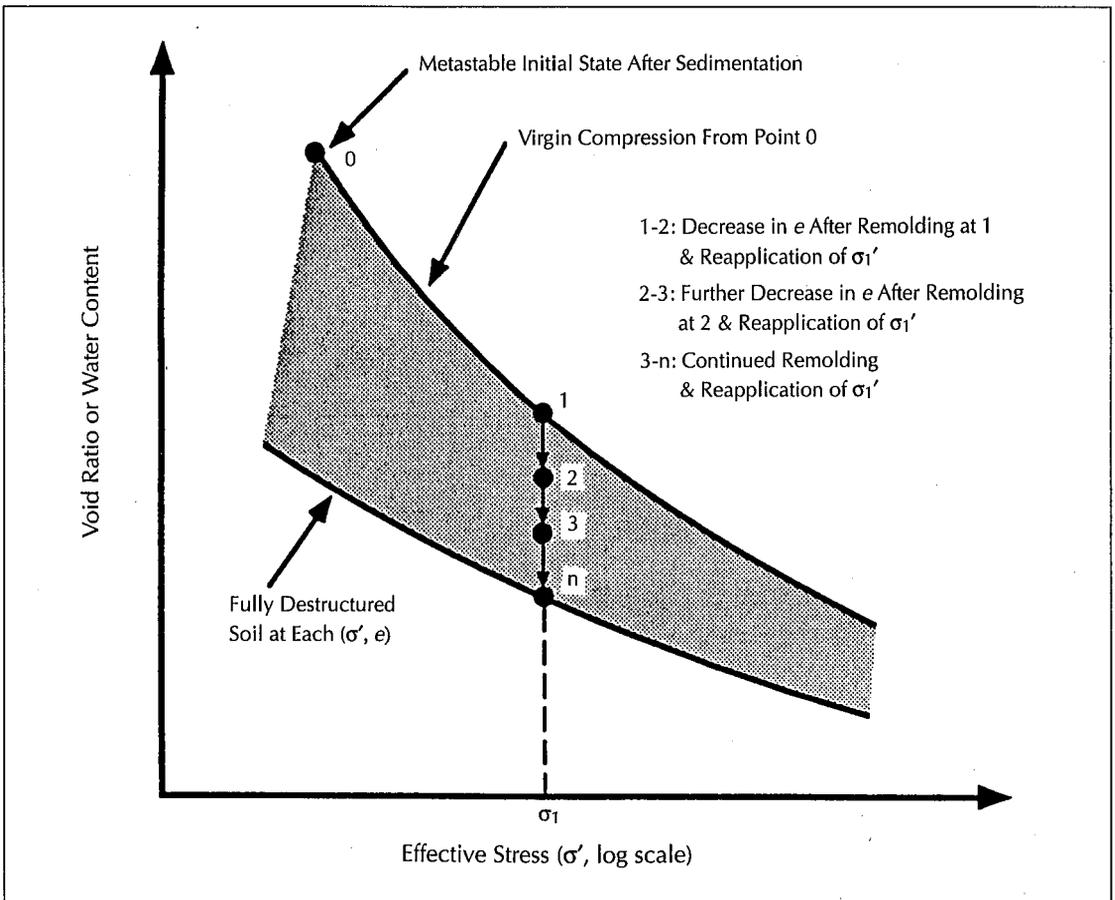


FIGURE 21. The influence of metastable fabric on void ratio under an effective consolidation pressure.

structure's environment that has been substantially changed since the initial structure formation may result in large changes in mechanical properties.

- The dynamic interactions between the solid and fluid phases of a soil mean that time-dependent reactions and interactions are possible.

Structure & Properties

The foregoing considerations can be combined with several principles that relate the fabric and structure of a soil to its mechanical properties in order to develop an understanding of real soil behavior and of the special soil behavior phenomena described earlier.

For any soil in which particles and particle groups flocculate, the initial fabric after soil stratum formation by weathering or sedimen-

tation will be open and involve some degree of edge-to-edge and edge-to-face particle associations producing a fabric that can carry effective stresses at a void ratio higher than would be possible if the particles were arranged in the most efficient possible packing. If, under undrained conditions, the soil is mechanically remolded from a state such as represented by Point 1 in Figure 21, the fabric is disrupted, effective stresses are reduced because of the tendency for the volume to decrease, and the strength decreases.

If the original consolidation stresses are re-applied, then there will be additional consolidation and the void ratio will decrease to a point as represented by Point 2 in Figure 21. Subsequent mechanical remolding and reapplication of stresses will cause consolidation to Point 3, and continued repetition of the process

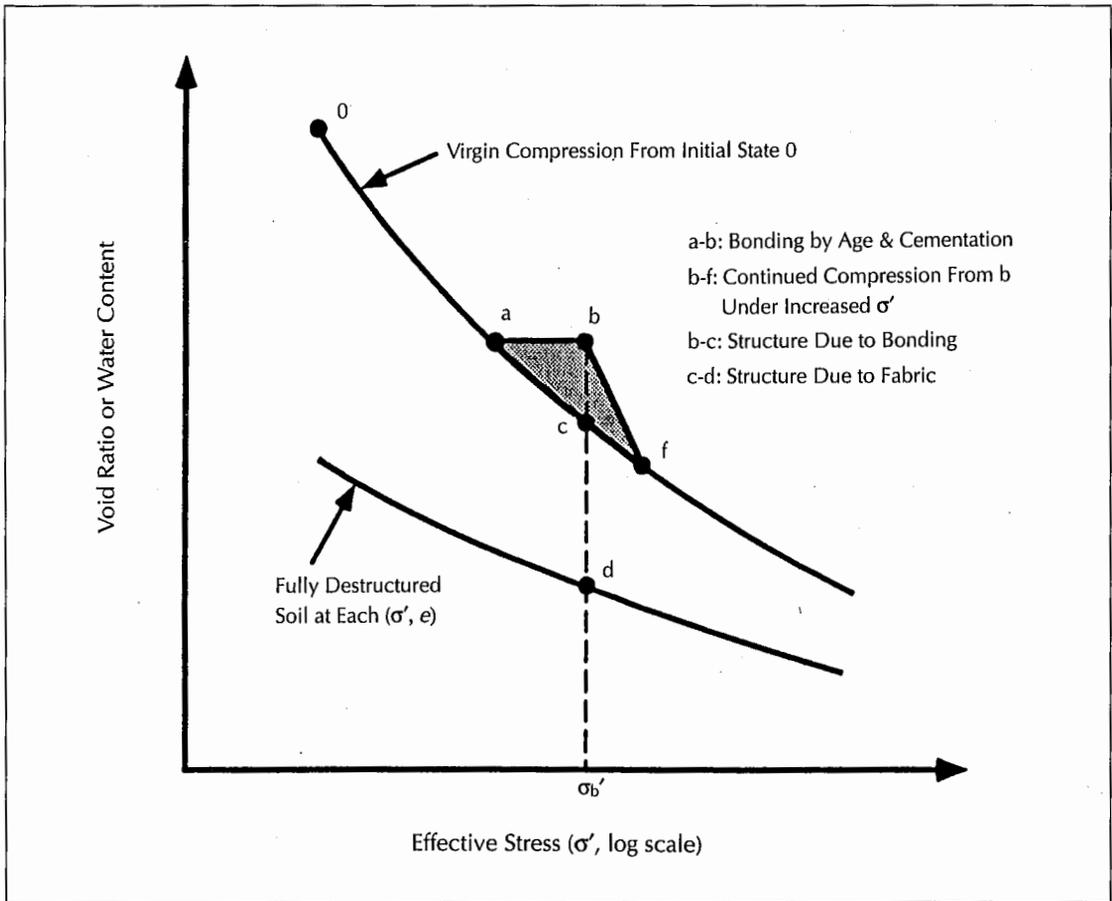


FIGURE 22. Possible states in void ratio-effective stress space.

will lead ultimately to a minimum void ratio for the fully destructured soil at n . Thus, if the soil is at any state within the shaded area depicted in Figure 21, it will have some degree of metastability of structure.

Also, it is possible that a soil can be at a state to the right of the virgin compression curve in Figure 21 as a result of bonding by chemical cementation or aging effects. Thus, the full range of possible states in void ratio-effective stress space is greater (see Figure 22) than shown in Figure 21. Virgin compression from an initial state at 0 to a is followed by the development of bonding, which enables the soil to resist additional compressive stress, a to b . At Point b , the soil is under effective stress, σ'_b . The completely destructured soil under the same stress would be at Point d . The difference in void ratios between the structures soil at b and the destructured soil

at d is made up of a bonding contribution, b to c , and a fabric contribution, c to d .

Several principles relate the fabric and structure of a soil to its mechanical properties. Some of these principles can be used, along with an understanding of the colloidal and chemical behavior of the clay-water-electrolyte system, to explain the special soil behavior phenomena presented earlier:

- Under a given effective consolidation pressure, a soil with a flocculated fabric is less dense than the same soil with a deflocculated fabric.
- At the same void ratios, a flocculated soil with randomly oriented particles and particle groups is more rigid than a deflocculated (destructured) soil.
- Once the maximum precompression stress has been reached, a further incre-

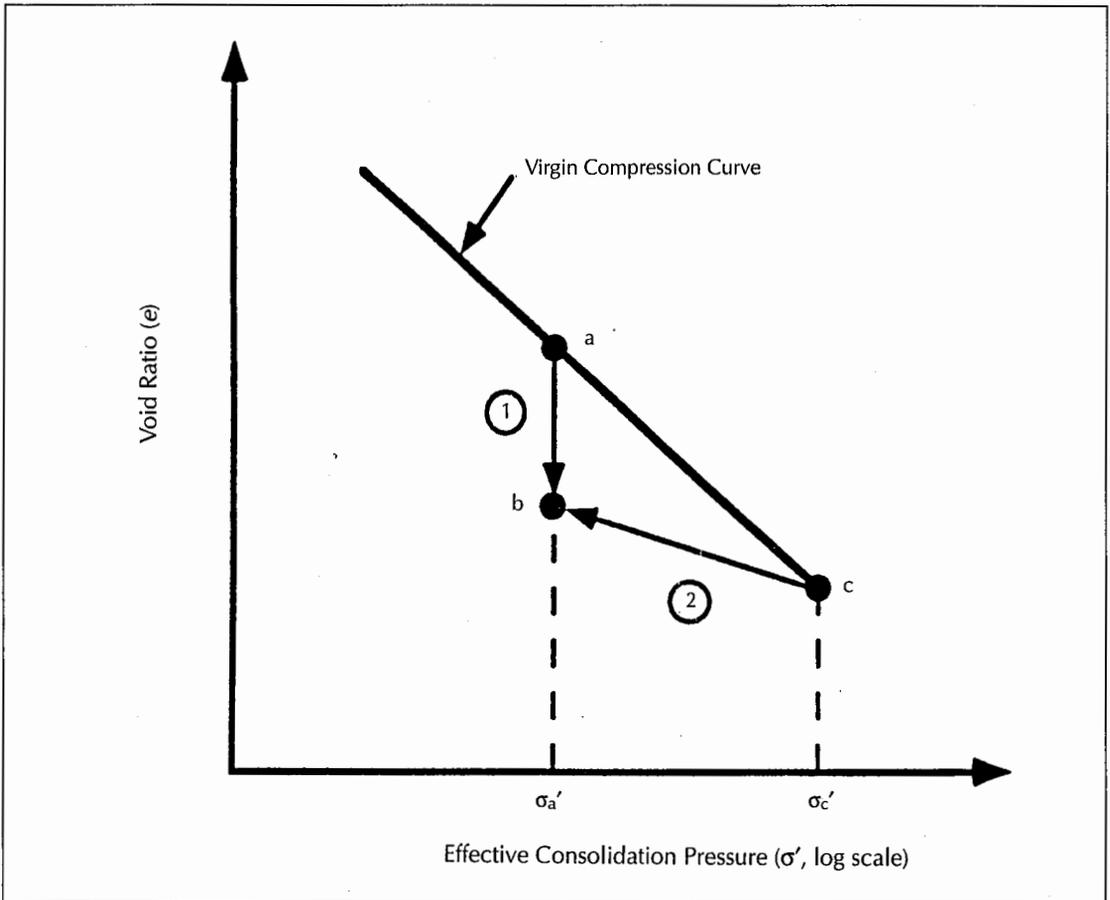


FIGURE 23. Different paths to reach the same void ratio-effective stress state.

ment of pressure causes a greater change in the fabric of a flocculated soil structure than in a deflocculated soil structure.

- At a given porosity, the average pore diameter and range of pore sizes is smaller in deflocculated and/or destructured soils than in flocculated and/or undisturbed soils.
- Shear displacements usually align platy particles and particle groups with their long axes in the direction of shear.
- Anisotropic consolidation stresses tend to align platy particles and particle groups with their long axes in the major principal plane.
- Stresses are not usually distributed equally among all particles and particle groups. Some particles and particle groups may be essentially stress free as a result of arching by surrounding particle groups.
- For an uncemented soil, two samples can have different structures at the same void ratio-effective stress coordinates if they have different stress histories. In Figure 23, a sample initially at Point *a* on the virgin compression curve can deform to Point *b* as a result of disturbance and re-consolidation or by secondary compression (creep) under stress σ'_a sustained for a long time. A sample initially at *c* can reach Point *b* as a result of unloading from σ'_c . The stress-deformation properties of the two samples will differ.
- Volume change tendencies determine pore pressure development during undrained deformation.
- Changes in the structure of a saturated soil at constant volume are accompanied by changes in effective stress. These effective stress changes are immediate.

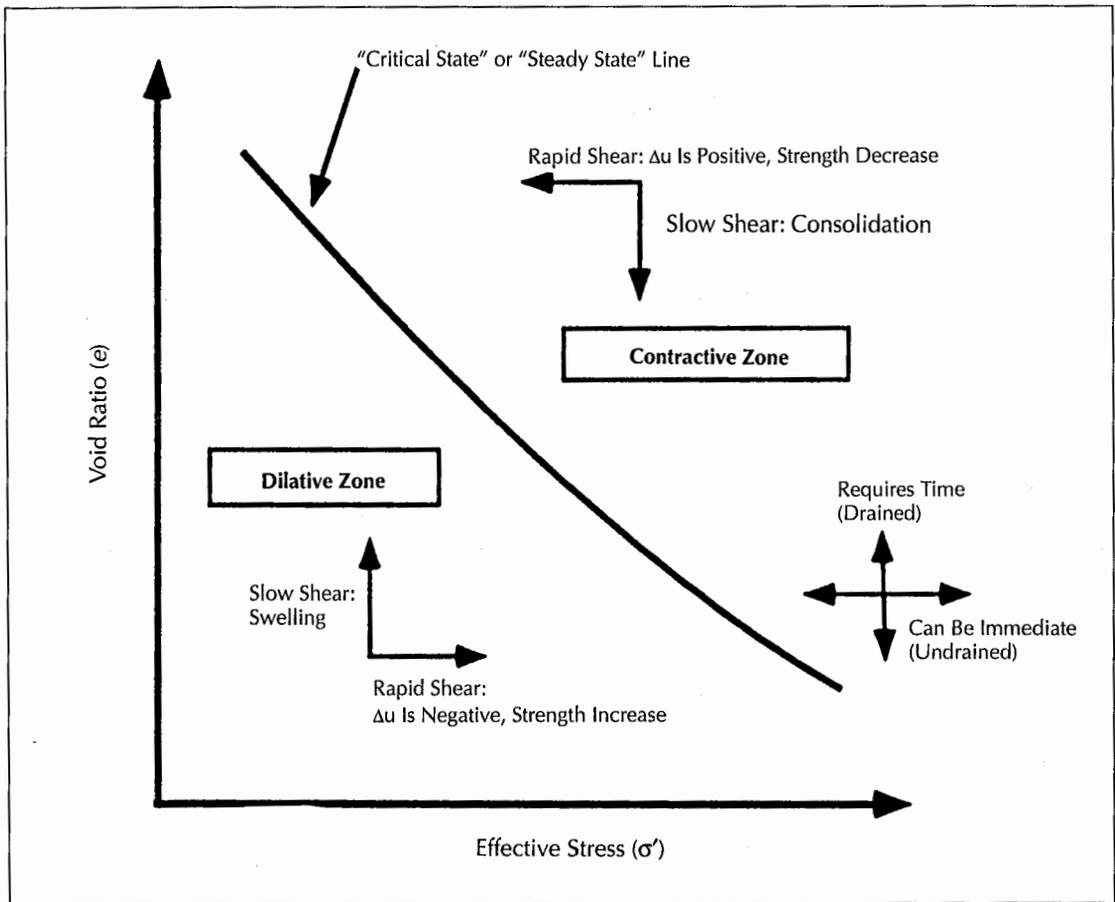


FIGURE 24. Initial state in relation to the critical or steady state line and its influence on pore pressures and volume change during deformation.

- Changes in the structure of a saturated soil at constant effective stress are accompanied by changes in void ratio. The change in void ratio is not immediate, but depends on the time for water to drain from or enter the soil.

The last three principles are illustrated by Figure 24. For any saturated, destructured soil the unique relationship between combinations of void ratio and effective consolidation pressure is commonly referred to as the *critical state* or *steady state* line. If the soil is on this line, there is no tendency for a change in volume during shear deformation. However, if the state of the soil is in the region above and to the right of this line, it will either contract if the rate of deformation is slow, or positive pore pressure will be generated if deformation is rapid. On the other

hand, if the soil is initially at a state in the dilative zone, slow deformation will be accompanied by swelling, and rapid deformation will be accompanied by the generation of negative pore pressures. In general, normally to slightly overconsolidated clays and saturated loose sands are contractive; whereas, heavily overconsolidated clays and dense sands are dilative.

Understanding Unusual Soil Behavior

Nine special types of soil behavior, ranging from swelling and collapsing soils through quick clays and thixotropic hardening to coupled flows, were described above. Some fundamental considerations were reviewed in the previous section that can provide an explanation and understanding for these special types of soil behavior.

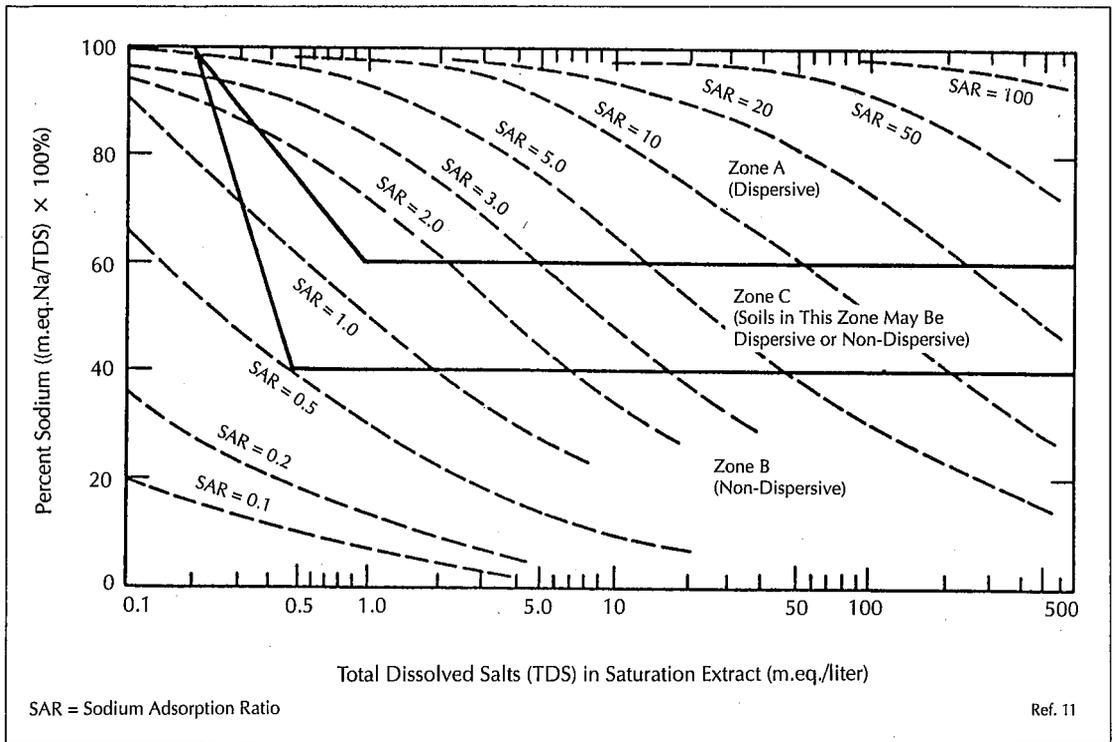


FIGURE 25. The relationship between dispersibility and dissolved pore water salts based on pinhole tests and field observations.

Swelling Soils. Theories for clay swelling have been developed along two rather different lines. The first, based on interparticle repulsions arising from double layer interactions, was introduced into geotechnical engineering literature by Bolt,⁶ and has been extensively evaluated since, as summarized by Mitchell.⁵ This theory is often referred to as the *osmotic pressure theory of swelling*. The second considers that swelling pressures and swelling are caused mainly by surface hydration, and it has been developed mainly by Low.^{7,8}

Purely mechanical contributions that result from the rebound of deformed particles and particle groups when confining pressures are reduced should be added to those possible causes of swelling. As shown by Terzaghi, simple mixtures of sand and mica flakes can be made to duplicate the compression and rebound behavior of a range of different naturally occurring fine-grained soils.⁹ That mechanical factors are significant in swelling is illustrated also by the differences in total swell measured for samples of clay compacted by

different methods to different initial structures as shown in Figure 9.

Without getting into the details and limitations of each approach, an explanation that is consistent with both the influences of the double layer/osmotic pressure theory and the water adsorption theory can be outlined as follows.

Clay particle charge density, adsorbed cation type, pore fluid electrolyte concentration and pore fluid dielectric constant determine the proportions of fully expandable and partially expandable layers in a potentially swelling clay. For example, calcium montmorillonite does not swell to interplate distances greater than about 0.9 nm (at which the particles stabilize). This particle structure is stabilized by attractive interactions between the basal planes of the unit layers. In contrast, the individual unit cell layers in sodium montmorillonite (bentonite) may separate completely in low electrolyte concentration solutions. In the presence of high electrolyte concentrations or pore fluids of low dielectric constant, interlayer

swelling is suppressed. In such cases, the effective specific surface is greatly reduced relative to that for the case where interlayer swelling occurs. As a result, the amount of water required to satisfy surface hydration is reduced greatly.

For example, data from Low suggest that a hydration water layer thickness on smectite surfaces of about 10 nm is needed to reach a distance beyond which the water properties are no longer influenced by surface forces,⁷ and the swelling pressure of montmorillonite is about 100 kPa for a water layer thickness of about 5 nm.¹⁰ For a fully expanding smectite having a specific surface of 800 m²/gm, this latter water layer thickness would correspond to a water content of 400 percent. Thus, a material such as sodium montmorillonite would be expected to be expansive over a very wide range of water contents, and experience shows clearly that it is.

On the other hand, consider an illite or a smectite made up of partially expanded clay particles (termed *quasi crystals*) so that interlayer swelling is negligible. Since both clays have surface structures that are essentially the same, it would be expected that the hydration forces should be similar. Thus, an adsorbed water layer thickness of 5 nm would also be reasonable. However, the specific surface areas of pure illite and nonexpanded smectite are only about 100 m²/gm, which corresponds to a water content of 50 percent for a 5-nm thick water layer. For a pure kaolinite having a specific surface of 15 m²/gm, the water content would be only 7.5 percent for a 5-nm thick adsorbed layer.

It is evident, therefore, that the specific surface dominates the amount of water required to satisfy forces of hydration. Except for very heavily overconsolidated clays and those soils that contain large amounts of expandable smectite, there is sufficient water present even at low water contents to satisfy surface hydration forces, and swelling is small. On the other hand, when the clay content is high and particle dissociation into unit layers is extensive, the effective specific surface area is large and swelling can be significant. The tendency for smectite clays to dissociate into unit layers can be evaluated through consideration of double layer interactions, with those conditions that

favor the development of high repulsive forces leading to greater dissociation. The interlayer bonding is too strong in the other clay mineral groups (see Figure 19) for separation of particles into individual unit layers so that the surface area would be large enough to give high swell or swell pressure.

Collapsing Soils. Collapsing soil behavior such as that exhibited by loess, debris flow material and other soils with a loose and metastable structure (as illustrated by the test results in Figure 11) can be understood on the basis of the considerations in the previous sections (see especially Figures 22 and 24). Owing to deposition in a loose state — *i.e.*, the contractive zone depicted in Figure 24 — and the subsequent development of weak bonding by clay or light cementation at silt and sand grain contacts, the soil reaches condition *b* in Figure 22. Then, exposure to water can cause a decrease in void ratio along *b* to *c* under self weight, or along *b* to *f* if additional loading is applied.

Dispersive Clays. The susceptibility of a fine-grained soil to dispersive clay behavior is not indicated by usual soil classification tests such as grain size distribution and Atterberg limits. The stability of clay soil structure against breakdown and particle dispersion, at least for non-marine clays, is better indicated by the proportion of sodium in the adsorbed cation complex. Chemical and physical tests for the identification of dispersive clays have been developed that are based on such determinations as the sodium concentration in relation to the total dissolved salts in the pore water (see Figure 25)¹¹ and the pinhole test. In the pinhole test distilled water is allowed to flow through a 1.0-mm diameter hole drilled through a compacted specimen. The water becomes muddy and the hole rapidly increases in size in dispersive clays. Additional tests include the Soil Conservation Service Dispersion Test in which the percentage of particles finer than 5 μm is determined by hydrometer analyses of samples with and without the use of a dispersing agent. The higher the ratio of percentage material finer than 5 μm by weight measured in the test without a dispersing agent to that measured in the test with a dispersing agent, the greater the probability of dispersion will be in the field.

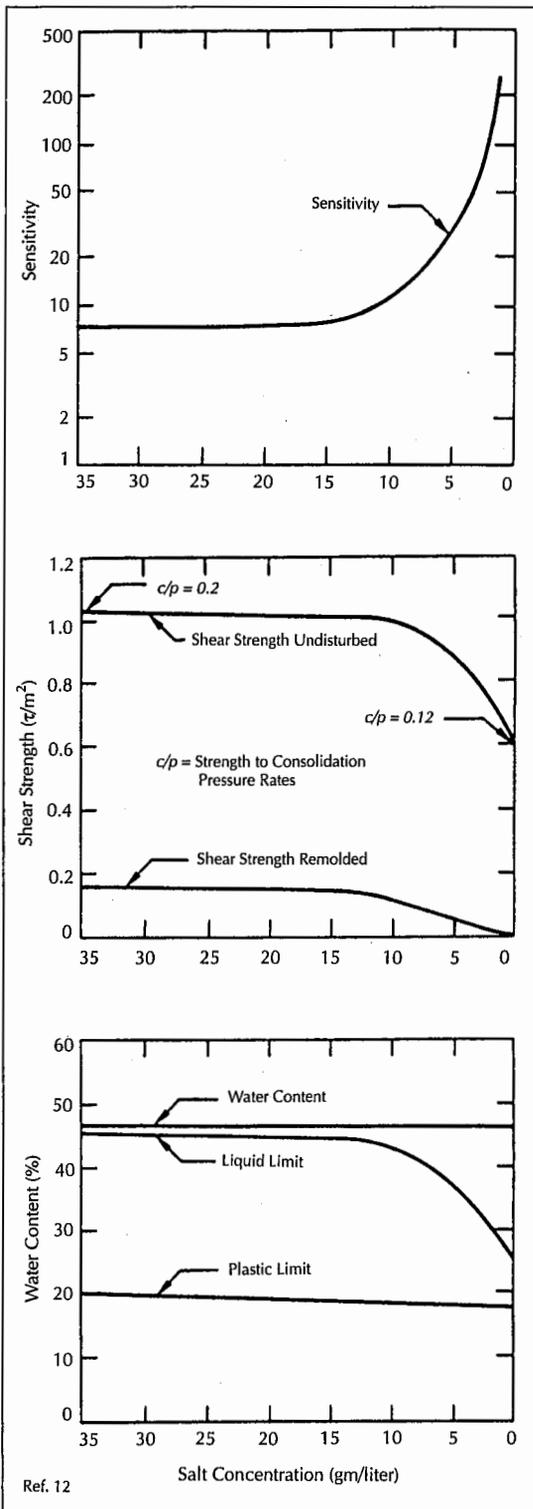


FIGURE 26. Changes in properties of a normally consolidated marine clay as a result of freshwater leaching.

Unfortunately, however, the tests as designed do not always reliably classify soils as to their dispersivity. In reality, this is not surprising, because whether or not a soil will exhibit dispersive behavior depends not only on its chemical and mineralogical composition, but also on its state (as reflected by water content, density and structure), on the chemistry of the water to which it is exposed and on the specific conditions of exposure such as temperature, confining pressure and velocity of flowing water. It is important, therefore, that in testing for dispersivity the sample state and water chemistry duplicate to the greatest extent possible those conditions to be expected in the field.

Slaking. There are three mechanisms responsible for slaking. Dispersion, which depends on the clay and water chemistry, was described above. Swelling, resulting from stress relief and water adsorption due to surface hydration and osmotic forces, can weaken the structure and lead to disintegration. Compression of entrapped air in partly saturated materials as a result of rapid water absorption can exert tensile stresses on the structure leading to splitting.

Sensitive & Quick Clay Formation. Most very sensitive and quick clays are post-glacial marine clays. Reduction in salinity, usually by leaching, is the first step in their development. Leaching by freshwater occurs after a drop in sea level or rise in land level. The presence of percolating water in sand and silt lenses is sufficient to remove salt from the clay by diffusion without requiring the water to flow through the pores of the intact clay. Although this leaching causes little change in fabric, the interparticle forces may be changed, resulting in a change in undisturbed strength of up to 50 percent, and such a large reduction in the remolded strength that a quick clay may form. The large increase in interparticle repulsion results from the decrease in electrolyte concentration and is responsible for deflocculation and dispersion of the clay on mechanical remolding. The changes in properties that accompany reductions in salt concentration in a normally consolidated marine clay are shown in Figure 26.

The details of the formation of a quick clay are somewhat more complex than indicated by

a simple reduction in salt content.^{5,13} However, the necessary conditions for the formation of a quick clay may be stated: they are low salt content, high percentage of monovalent cations in the adsorbed layers on the clay particles, and high pH.

Quick Clay Aging. Changes in the remolded strength and consistency of a Canadian quick clay as a function of time were shown in Figure 12. These aging effects can be explained as follows.

When a quick clay is sampled or exposed, some contact with air and oxygen is inevitable. Small amounts of organic matter present in the clay are oxidized to form carbonic acid which, in turn, dissolves calcium carbonate, thus increasing the concentrations of calcium and bicarbonate in the pore water. Oxidation of pyrite (small amounts are present in the rock flour that comprises much of the solid content of quick clays) forms sulfuric acid and ferric hydroxide. The sulfuric acid reacts with magnesium calcite to increase the concentrations of dissolved calcium and magnesium in the pore water and in the adsorbed cation complex on the clay particles. Sodium and potassium are displaced from the double layer to the pore water.

The chemical changes developed in LaBaie, Québec, quick clay as a function of time are illustrated in Figure 27.¹³ The different curves in Figure 27 refer to different conditions of sample storage. The salinity increase and the increase in concentrations of the divalent cations cause increases in the remolded strength and the liquid limit as well as decreases in the sensitivity and liquidity index.

The aging of quick clays is an excellent example of how even seemingly small changes in environmental conditions can result in significant changes in properties. These changes can occur over times typical of those associated with the field and laboratory phases of many projects — *i.e.*, several weeks to a few months. To minimize aging effects, the exposure of samples to air should be minimized, thick wax caps should be used with rust-free sample tubes, and samples should be stored at low temperatures to slow down reaction rates.

Thixotropic Hardening. A basic outline of thixotropic hardening (see Figure 13) first

should be presented. Sedimentation, remolding, and compaction of soils produce structures compatible with the processes that are acting on the soils. Once the externally applied energy of remolding or compaction is removed, however, the structure is no longer in equilibrium with the surroundings. If the interparticle force balance is such that attraction is in excess of repulsion, there will be a tendency for the flocculation of particles and particle groups and for a reorganization of the water-cation structure to a lower energy state. Both effects, which have been demonstrated experimentally, take time because of the viscous resistance to particle and ion movement.

Several studies have shown that there is a continual decrease in pore water pressure, or an increase in pore water tension, with time after compaction or remolding. The concurrent increase in effective stress accounts for the observed increase in strength.

Aging of Sands. The effect of time on the penetration resistance of a hydraulic fill was depicted in Figure 14, and the effect of time on the penetration resistance of a natural sand deposit after densification by blasting was shown in Figure 15. Unfortunately, the specific mechanisms responsible for strength and stiffness increases associated with the aging of sands are not yet known in detail.

There has not been clear resolution of the relative importance of chemical factors (such as silicate precipitation at interparticle contacts and changes in surface characteristics) and mechanical factors (such as time-dependent stress redistribution and particle reorientations) in causing the observed behavior. Evidence and arguments in support of chemical hardening mechanisms are given by Mitchell and Solyman;¹⁴ whereas, Mesri *et al.* and Schmertmann present a case for mechanical mechanisms, particularly secondary compression-like processes.^{15,16} More recent analyses and experiments have failed to yield unambiguous results concerning mechanisms,¹⁷ and reliable means for predicting the magnitude and rate of strength and stiffness increase remain to be developed.

Creep & Stress Relaxation. The general characteristics of creep and stress relaxation were shown in Figure 16, and the effect of shear rate

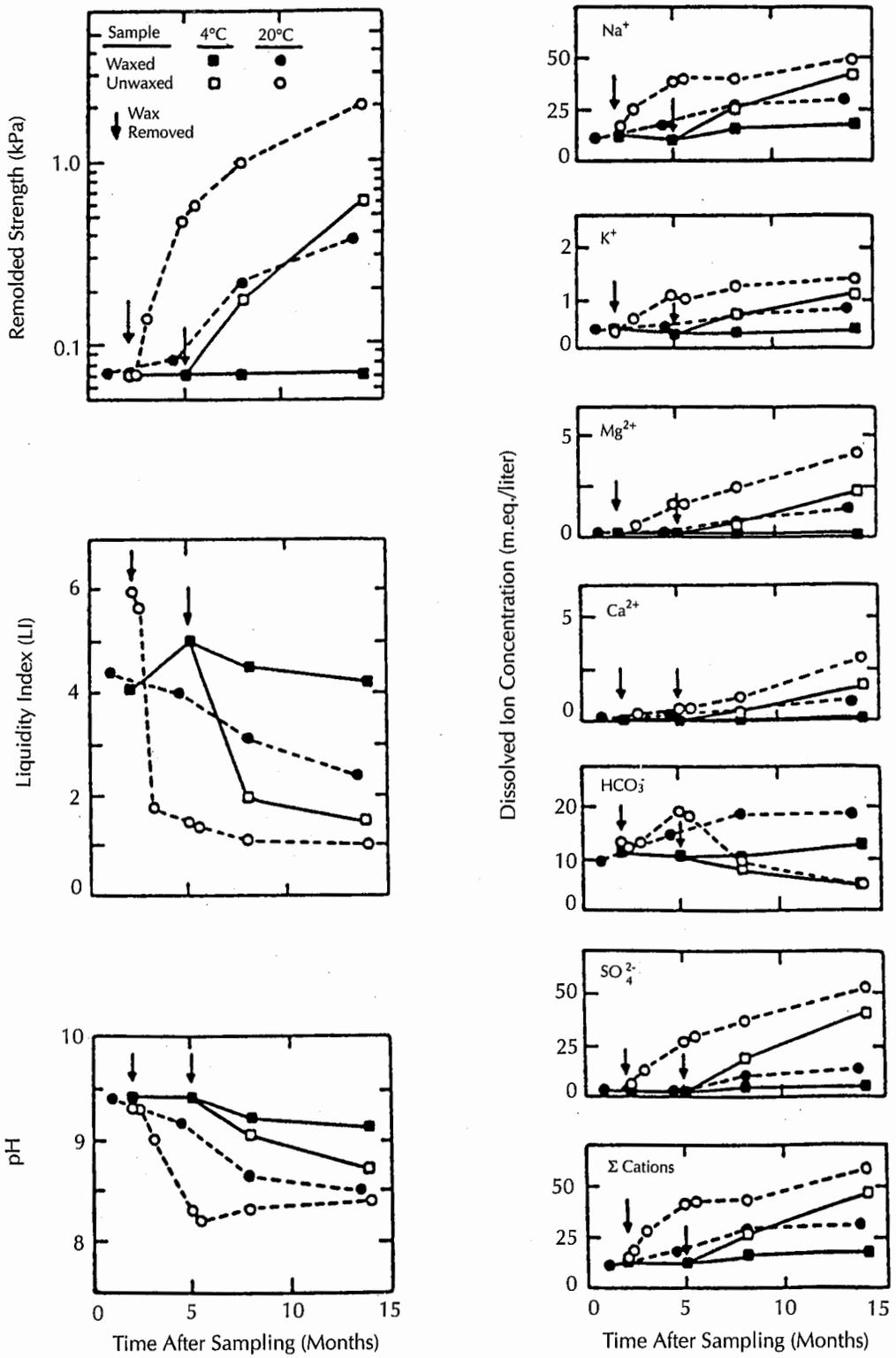


FIGURE 27. The effect of time and storage conditions on properties of LaBaie quick clay.

on shear resistance was portrayed in Figure 17. The results of many studies have revealed that these time-dependent changes in stress and strain are the result of thermally activated processes. Analyses within the framework of rate process theory have provided insights into the stress and time dependencies of creep and stress relaxation rates. Such studies have also enabled development of consistent hypotheses for bonding, effective stresses and strength of soils that are suitable for most soil types, soil states and loading conditions. A detailed review of this subject may be found in Mitchell.⁵

Coupled Flows. The last of the special soil behavior phenomena described earlier was coupled flow processes. Types of coupled flow processes that may occur in soils were summarized in Table 1. A comprehensive treatment of coupled flows in terms of mechanisms, theories and applications can be found in Mitchell.^{5,18}

Coupled flows are generated because of the non-uniform distributions of cations and anions and electrical charges (as shown in Figure 20) and because of the small pore sizes in fine-grained soils. Clays can exhibit membrane properties, which means that the passage of certain ions and molecules through the clay may be restricted in part or in full at both microscopic and macroscopic levels. These restrictions mean that the application of gradients of different types — hydraulic, thermal, electrical or chemical — can induce local and global imbalances in pressure, temperature, electrical potential and/or chemical concentration. These imbalances result in flows of different types in addition to the direct flow of the type corresponding to the applied gradient.

Some recent applications of coupled flow considerations in geotechnical problems are described in Mitchell¹⁸ and include thermally driven moisture flow and its consequences relative to the thermal stability of buried electrical cable backfills, in-situ determination of clay consolidation properties by electro-osmosis, the relative importance of diffusive and advective chemical transport through clays, waste containment and site remediation by electro-osmosis, potential consequences of in-situ potentials in relation to slope stability, and

the possibility for stabilizing the foundations of existing structures using electro-osmosis.

Conclusion

In most geotechnical engineering projects and problems, correct site characterization and property evaluation are the two most critical elements. If they are not done reasonably and reliably, neither an understanding of or confidence from subsequent soil mechanics analyses can be obtained, no matter how sophisticated the analyses may be or how powerful the computer that performs them. Thus, running through every aspect of our work is the need for sufficient understanding of what soils are and the principles that govern their important geotechnical properties, which in most cases are those pertaining to flows through them, to volume change, to stress-deformation behavior and to strength.

Real soils are not composed of inert materials, but their state and properties depend importantly on their composition and the environmental conditions in which they exist. Simple soil models should be used for analysis only with an appreciation of the assumptions and approximations that they contain. Some unusual, but generally well-known types of soil behavior have been described herein that are important in practice, but which, while understood, cannot be represented by simple equations or empirical correlations. Nonetheless, it is important that their consequences be considered in practice.

No one has stated the importance of soil mechanics in geotechnical engineering better than Professor Arthur Casagrande. In his closing remarks to an engineering conference held by the Corps of Engineers in 1938, Casagrande said:

Soil mechanics is a complex subject because the materials with which we deal are so utterly complex. In addition, to penetrate deeper and deeper into the mysterious behavior of soils, we need a much greater variety of tools than, for example, are required in structural engineering. A man who desires to work in soil mechanics needs not only a thorough knowledge of the properties of materials in general, of mathematics, and at least the principles of the theory of elasticity,

but also knowledge of physical chemistry and geology, and in addition the mastery of some entirely new conceptions and avenues of approach which form the fundamentals of soil mechanics.

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JAMES K. MITCHELL is the Charles E. Via, Jr., Professor of Civil Engineering at Virginia Polytechnic Institute and State University, where he moved in 1994 after 36 years at the University of California, Berkeley, where he held the Edward G. Cahill and John R. Cahill Chair in Civil Engineering. His teaching, research and consulting activities have focused on soil behavior, soil stabilization and ground improvement, and environmental geotechnics. He has published numerous papers on this work, as well as the graduate level text and reference, *Fundamentals of Soil Behavior*.

REFERENCES

1. Carrier, W.D., III, Olhoeft, G.R., & Mendell, W., "Physical Properties of the Lunar Surface," Chap. 9, *The Lunar Sourcebook*, Cambridge University Press, 1991, pp. 475-594.
2. Clevenger, W.A., "Experiences With Loess as a Foundation Material," *Transactions*, ASCE, Vol. 123, 1958, pp. 151-180.
3. Lessard, G., "Traitement chimique des argiles sensibles d'Outardes-2," *Memorie de M. Sc. A.*, Ecole Polytechnique de Montreal, 1978.
4. Kulhawy, F.H., & Mayne, P.W., *Manual for Estimating Soil Properties for Foundation Design*, Final Report, Project 1493-6, EL-6800, 1990, Electric Power Research Institute, Palo Alto, CA.
5. Mitchell, J.K., *Fundamentals of Soil Behavior*, 2nd ed., John Wiley & Sons, New York, 1993.
6. Bolt, G.H., "Physico-Chemical Analysis of the Compressibility of Pure Clays," *Geotechnique*, Vol. 6, No. 2, 1956, pp. 86-93.
7. Low, P.F., "Structural Component of the Swelling Pressure of Clay," *Langmuir*, Vol. 3, 1987, pp. 18-25.
8. Low, P.F., "Interparticle Forces in Clay Suspensions: Flocculation, Viscous Flow and Swelling," *Proceedings of the 1989 Clay Minerals Society Workshop on the Rheology of Clay/Water System*, 1992.
9. Terzaghi, K., "The Influence of Elasticity and Permeability on the Swelling of Two-Phase Systems," in J. Alexander, ed., *Colloid Chemistry*, Vol. III, Chemical Catalog Co., New York, 1931, pp. 65-88.
10. Low, P.F., "The Swelling of Clay: II Montmorillonites," *Journal of the Soil Science Society of America*, Vol. 44, 1980, pp. 667-676.
11. Sherard, J.L., Dunnigan, L.P., Decker, R.S., & Steele, E.F., "Identification and Nature of Dispersive Soils," *Journal of the Geotechnical Division*, ASCE, Vol. 102, No. G T 4, 1976, pp. 287-301.
12. Bjerrum, L., "Geotechnical Properties of Norwegian Marine Clays," *Geotechnique*, Vol. 4, 1954, pp. 49-69.
13. Lessard, G., & Mitchell, J.K., "The Causes and Effects of Aging in Quick Clays," *Canadian Geotechnical Journal*, Vol. 22, No. 3, 1985, pp. 335-346.
14. Mitchell, J.K., & Solymar, Z.V., "Time-Dependent Strength Gain in Freshly Deposited or Densified Sand," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 110, No. G T 11, 1984, pp. 1559-1576.
15. Mesri, G., Feng, T.W., & Benak, J.M., "Postdensification Penetration Resistance of Clean Sands," *Journal of Geotechnical Engineering*, ASCE, Vol. 116, No. 7, 1990, pp. 1095-1115.
16. Schmertmann, J.H., "The Mechanical Aging of Soils," *Journal of Geotechnical Engineering*, ASCE, Vol. 117, No. 9, 1991, pp. 1288-1330.
17. Human, C.A., *Time Dependent Property Changes of Freshly Deposited or Densified Sands*, Doctoral Dissertation in Civil Engineering, University of California, Berkeley, 1992.
18. Mitchell, J.K., "Conduction Phenomena: From Theory to Geotechnical Practice," *Geotechnique*, Vol. 41, No. 3, 1991, pp. 299-340.