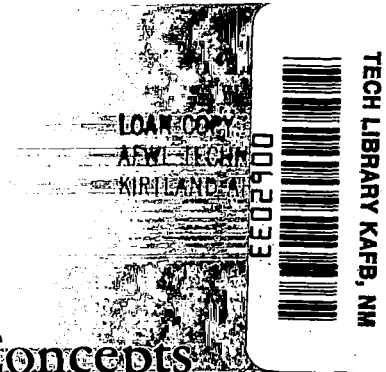


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NASA Contractor Report 3365



Development of Experimental Concepts
for Investigating the Strength
Behavior of Fine-Grained Cohesive
Soil in the Spacelab/Space
Shuttle Zero-G Environment

Rudolph Bonaparte and James K. Mitchell

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Prepared for
Marshall Space Flight Center
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National Aeronautics
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**Scientific and Technical
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FOREWORD

The studies reported herein were performed under a Consulting Agreement between Professor James K. Mitchell, Department of Civil Engineering, University of California, Berkeley, and The Virginia Polytechnic Institute and State University (VPI), Blacksburg, Virginia, as partial fulfillment of the scope of work of Contract NAS8-33449, "Phase-I Spacelab Experiment Definition Studies Related to Soil Behavior," between VPI and the George C. Marshall Space Flight Center.

This study effort was sponsored by the NASA Office of Aeronautics and Space Technology, Physics and Chemistry Experiments in Space Working Group. The Principal Investigator for VPI was Professor Stein Sture, currently with the Department of Civil, Environmental and Architectural Engineering, University of Colorado at Boulder; and the NASA Technical Monitor for the Same Studies was Dr. Nicholas C. Costes, Space Sciences Laboratory, Marshall Space Flight Center.

DEVELOPMENT OF EXPERIMENTAL CONCEPTS FOR INVESTIGATING THE STRENGTH BEHAVIOR OF FINE-GRAINED COHESIVE SOIL IN THE SPACELAB /SPACE SHUTTLE ZERO-G ENVIRONMENT

I. INTRODUCTION

The development of the National Aeronautics and Space Administration's (NASA) Spacelab system presents for the first time the opportunity for scientists and engineers to carry out long-term scientific and technical experiments in space. Space offers the advantages of a near-zero gravity environment as well as an ultra-high vacuum. This combination of experimental conditions is virtually impossible to duplicate on earth over an extended time period.

This report attempts to ascertain the possible benefits that might accrue to geotechnical engineers and soil behavior specialists through utilization of the unique environment of space to study fine-grained, cohesive soil deformation and strength behavior, particularly under conditions of very low confining pressure. On earth, the presence of gravitational body forces puts a lower bound on the stresses that can be applied in an experiment. Thus, it is extremely difficult, if not impossible, to study the shear strength characteristics of cohesive soils in the region of the effective stress origin. Furthermore, the presence of gravity leads to stress nonuniformities and fabric anisotropy within test specimens. All of these limitations of terrestrial soil mechanics testing can be overcome in the space environment.

The work presented in this report has been motivated by the knowledge that the behavior of cohesive soils in the region of the stress origin is important for the solution of several types of problems that occur in geotechnical engineering. Engineers are concerned with the development of transverse and longitudinal cracking in earth dams. Several studies which will be described within this report have shown the importance of an understanding of cohesive soil behavior under low effective stresses and in tension in attempting to predict zones of cracking in earth dams. Recently, tensile creep rupture has been suggested as a possible cause of cracking in earth dams. To date there has been very little laboratory work done on the tensile stress-strain-time properties of soils.

Related to the problem of cracking in earth dams is the problem of hydraulic fracturing of earth dams as well as the underlying foundations. Other geotechnical problems in which an understanding of cohesive soil behavior at low stresses and in tension is important include the development of tensile cracks at the crests of slopes, in pavement subgrades, and in areas undergoing subsidence. Recently, attention has been focused on the consequences of cracking in soil deposits that are being considered for use in the containment of low-level radioactive wastes and hazardous industrial wastes.

An understanding of the conditions of failure at low effective stresses and in tension will be useful in developing constitutive relationships to mathematically describe the stress-strain-strength behavior of cohesive soils in the region of the stress origin. Tests at low effective stresses will be of assistance in developing yield criteria, plastic potentials, and work hardening rules. Constitutive relationships developed as a result of this work will be useful in the analyses of submarine slope stability problems.

From a scientific viewpoint the testing of cohesive soils in the region of the stress origin will provide information on the existence of true cohesion and true tensile strength, as well as on the mobilization of these quantities with strain. The tests will also be useful in understanding the role of interparticle attractive and repulsive forces in the generation of shearing resistance, as well as the development of anisotropy and preferred particle orientation due to anisotropic stress states.

The approach that has been taken in this report has been to first review as thoroughly as possible, fine-grained cohesive soil behavior in order to develop insight into the mechanisms that govern the strength and deformation behavior of these materials. In addition, this review will illuminate areas of cohesive soil behavior in which our understanding is incomplete. The review points towards areas in which the NASA Spacelab experimental program could make significant contributions.

As a starting point fundamental aspects of clay mineralogy, adsorbed water, the clay-water-electrolyte system and interparticle forces are reviewed. Various models that have been developed to describe the development of shearing resistance in soils will then be presented. Concepts and theories from disciplines other than geotechnical engineering that have shown themselves to be useful in describing soil behavior will also be presented. Two theories from outside of geotechnical engineering that have been shown to be particularly useful in describing the development of shearing resistance in soils are the adhesion theory of friction and the theory of rate processes. These will be discussed in detail. Fundamental studies of cohesion and friction will also be reviewed as will experimental studies aimed at understanding the mechanisms controlling cohesive soil strength and deformation behavior at low effective stress levels and investigations into the tensile strength of cemented and non-cemented soils.

The results of the literature review have been used to formulate a comprehensive hypothesis for strength generation in fine-grained cohesive soils. This hypothesis presents a mechanism for strength generation and identifies the possible causes of true cohesion. Through the hypothesis the roles of overconsolidation, physico-chemical variables, fabric, surface contaminants and time effects can be understood. The comprehensive hypothesis is also used as a guide in identifying potentially fruitful experiments for the NASA Spacelab program.

Finally, potential experiments involving both in-space and terrestrial laboratory testing will be recommended. These tests will be primarily concerned with the strength and deformation behavior of fine-grained cohesive soils. Recommendations for testing will be based on the contributions that a test can make to both the scientific aspects of soil behavior as well as to the practice of geotechnical engineering.

II. REVIEW OF THE NATURE OF COHESIVE SOILS

Introduction

In this section a relatively brief review of the geotechnical literature pertaining to the nature of fine-grained cohesive soils will be presented. Emphasis will be placed on those aspects of behavior most important to an understanding of shear strength generation in soils. The review will proceed from fundamental considerations of clay mineral structure and the nature of adsorbed water on clay particles. Based on the structure of the various clay minerals, interparticle attractive and repulsive forces will be identified, and their potential magnitudes in various clay-water-electrolyte systems will be discussed. Of primary concern will be the discerning of those forces that are responsible for controlling the behavior of a soil's interparticle contacts, since it is at these contacts that resistance to shear is generated. Differences in the roles of short-range and long-range interparticle forces in influencing cohesive soil behavior will also be discussed.

Clay Mineral Structure

The clay minerals commonly found in soils belong predominantly to a class of minerals termed hydrous aluminum silicates which are part of a larger family of layered silicates called phyllosilicates. Other members of the phyllosilicate family include serpentine, pyrophyllite, talc, mica, and chlorite. The three most common groups of clay minerals found in soils are kaolinite, hydrous micas (illites) and smectites (montmorillonite)¹.

The structure of the clay minerals can be conveniently described in terms of two different structural units, these being sheets and layers. The clay mineral groups are differentiated by the way in which two or three sheets combine to form a layer and the manner in which two successive layers are held together. Two different types of sheet structures are found in the clay minerals, silica sheets and octahedral sheets.

Silica sheets are made up of silica tetrahedra in which a silicon ion is tetrahedrally coordinated with four oxygen atoms. Three of the four oxygen atoms of each tetrahedron are shared to form a hexagonal network, as shown in Fig. 1. As can be seen, the bases of the tetrahedra are all in the same plane, and the tips all point in the same direction. The oxygen-to-oxygen distance in the silica sheet is 2.55 Å (Å = angstrom = 1×10^{-10} meters), and the space available for the cation in tetrahedral coordination is about .55Å. The thickness of the sheet is 4.65Å. The silica sheet structure can be repeated indefinitely and has the composition $(\text{Si}_4\text{O}_{10})^{4-}$.

¹The term illite will be used interchangeably with hydrous mica herein, as will montmorillonite with smectite.

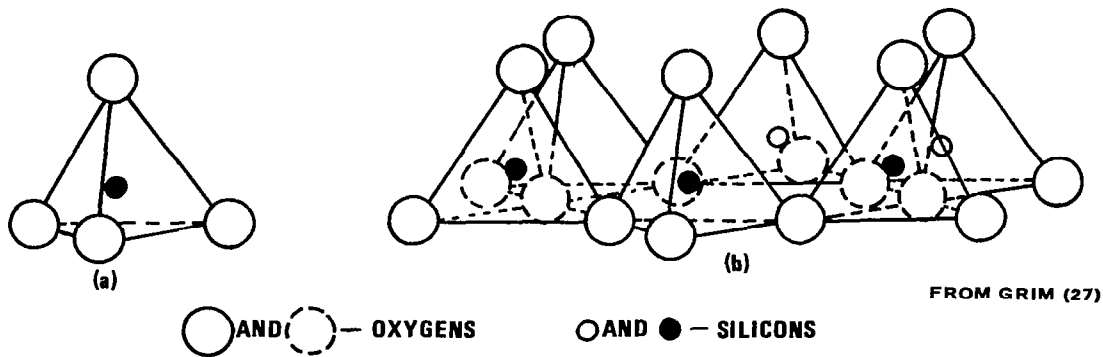


Figure 1. Diagrammatic sketch showing (a) a single silica tetrahedron and (b) the sheet structure of silica tetrahedrons arranged in a hexagonal network.

Octahedral sheets are composed of two planes of closely packed oxygen atoms or hydroxyl ions in which aluminum, iron, magnesium or other cations are embedded in octahedral coordination. An octahedral sheet is shown schematically in Fig. 2. If a trivalent cation is present in the octahedral structure, only two-thirds of the cationic spaces are normally filled, and the structure is termed dioctahedral. If the trivalent cation is aluminum, the chemical composition is $Al_2(OH)_6$, which is the mineral gibbsite. If the cations present in the octahedral sheet are divalent, all of the cationic sites are normally filled and the structure is termed trioctahedral. If the cations are all magnesium, the chemical composition is $Mg_3(OH)_6$, which is the mineral brucite.

A basic clay mineral layer is made up of a stack of two or three tetrahedral and octahedral sheets. The stacking of these sheets is such that a single plane of atoms is common to both the tetrahedral and octahedral sheets. Bonding between sheets is of the primary valence type and is quite strong. Bonding between unit layers of the different clay mineral kaolinite is composed of a single dioctahedral sheet combined with a single tetrahedral sheet. The tips of the silica tetrahedra and one of the planes of the dioctahedral sheet are common. Thus the tips of the silica tetrahedra point towards the center of the unit layer. Fig. 3 shows a diagrammatic sketch of the structure of a kaolinite unit layer. The structural formula is $(OH)_8Si_4Al_4O_{10}$. The unit layers of kaolinite are stacked one on top of another. Bonding between layers is by hydrogen bonding between oxygen atoms in the tetrahedral sheet and hydroxide ions in the octahedral sheet. The hydrogen bonds are relatively strong and are of sufficient strength to prevent any interlayer swelling. The amount of isomorphous substitution² is very small, with an occasional aluminum ion

²Isomorphous substitution is defined as "the substitution of ions of one kind by ions of another type, with the same or different valence, but with retention of the same crystal structure" (71). Isomorphous substitution leads to net negative charges on the clay particles. The net negative charge, in turn, strongly influences the clay-pore fluid system behavior.

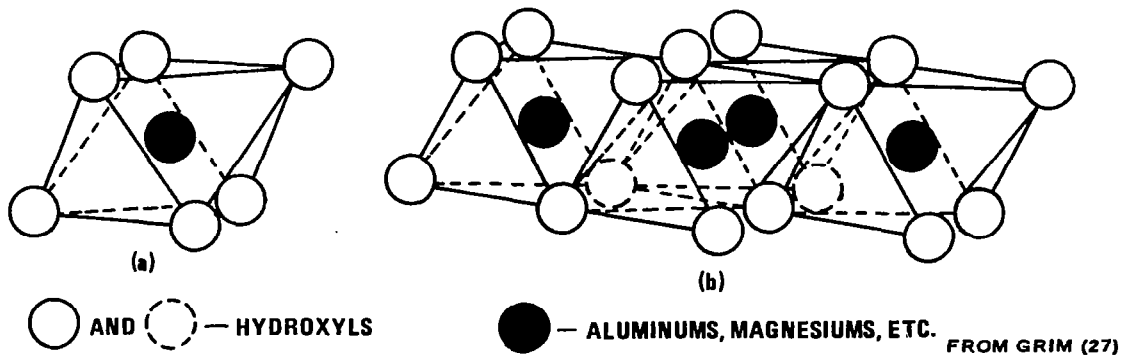


Figure 2. Diagrammatic sketch showing (a) a single octahedral unit and (b) the sheet structure of the octahedral units.

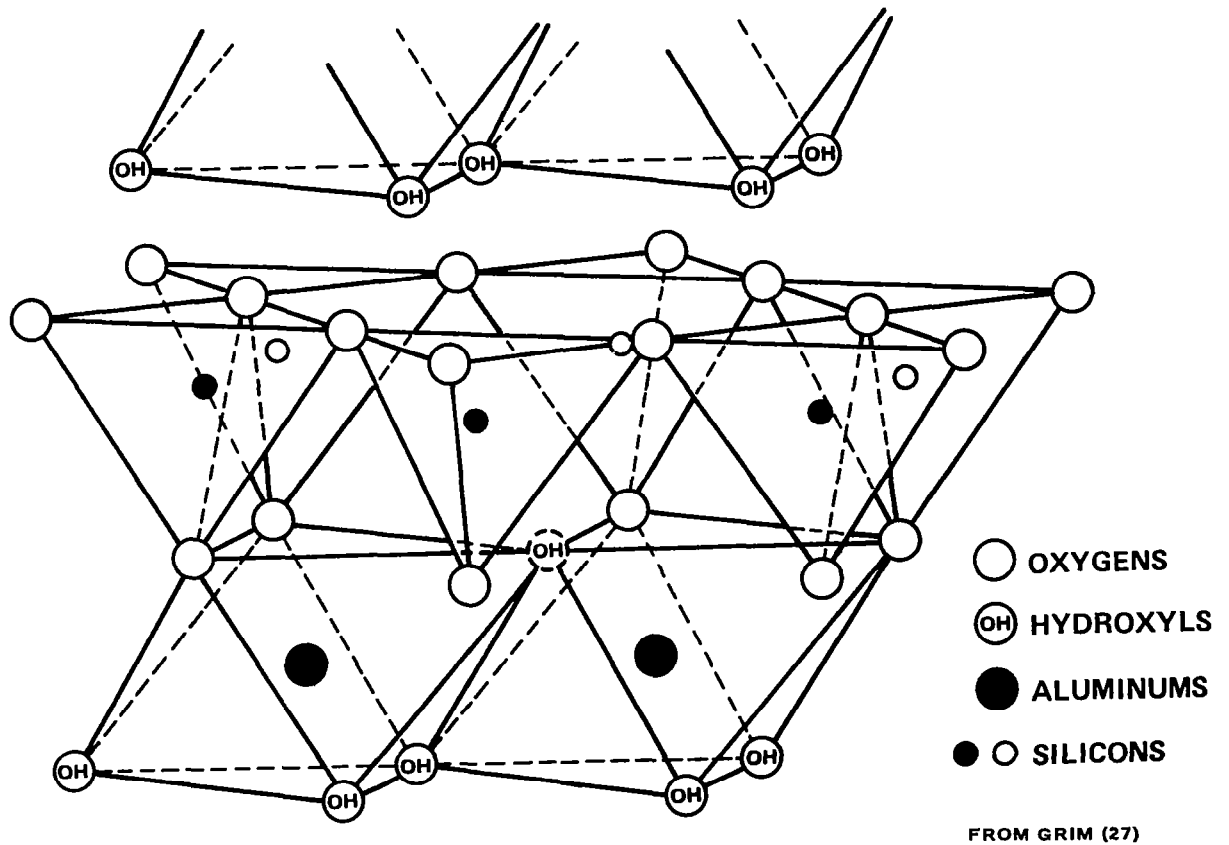


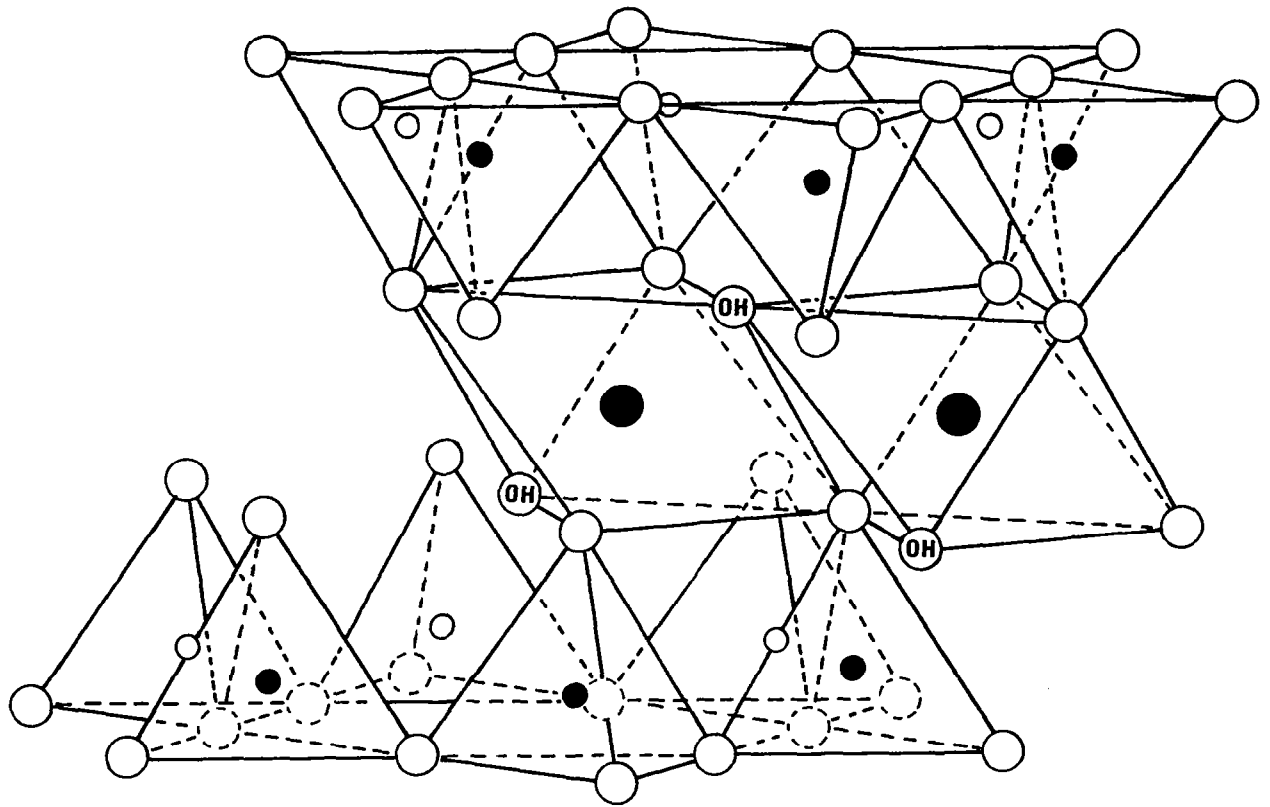
Figure 3. Diagrammatic sketch of the structure of the kaolinite layer.

replacing a silicon ion in the tetrahedral sheet or a Fe^{2+} ion replacing an Al^{3+} ion in the octahedral sheet. The cation exchange capacity (quantitative measure of the number of cations required to balance the resulting negative particle charge) of kaolinite ranges between 3-15 meq/100 grams. The particle size of kaolinite ranges between 0.5 and 2 μm in thickness and .1 and 4 μm in lateral dimension. The specific surface area of kaolinite is about 10 to 20 m^2 per gram of dry clay (71).

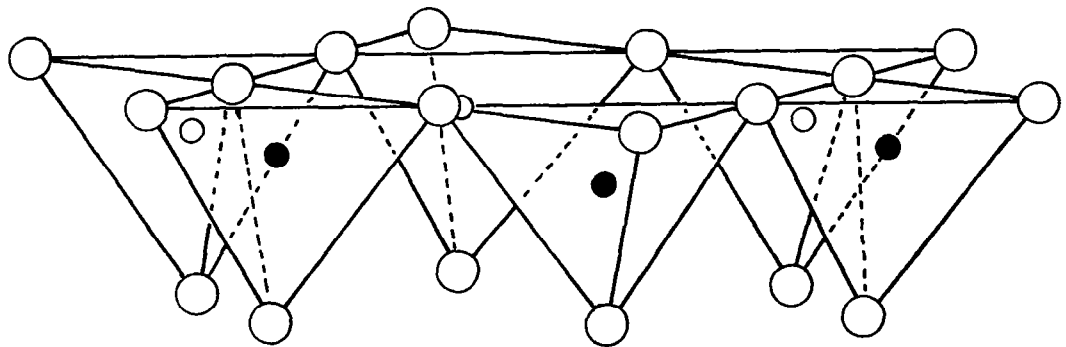
The structure of the smectite group minerals consists of a single octahedral sheet sandwiched between two silica sheets. The tips of the silica tetrahedra point towards the center of the unit cell, with the oxygens forming the tips being common to the octahedral sheet. The unit layers of the smectite minerals are stacked one on top of another. As a consequence, there are two planes of oxygen atoms stacked next to each other at the interface between unit layers. This structure leads to weak bonds between layers and excellent cleavage. Bonding between successive layers is by van der Waals forces and by cations that may be present to balance charge deficiencies in the structure (71). Extensive isomorphous substitution exists within the minerals of the smectite group, leading to substantial charge deficiencies. These deficiencies are balanced by exchangeable cations that take up positions between the unit cell layers and on the surfaces of particles (71).

The theoretical composition of smectite, in the absence of lattice substitutions, is $(\text{OH})_4\text{Si}_8\text{Al}_4\text{O}_{20}\cdot n\text{H}_2\text{O}$. A diagrammatic sketch of the structure of smectite is shown in Figure 4. Due to the large amount of isomorphous substitution, the smectites exhibit high cation exchange capacities, in the range of 80-150 meq/100 g. Because of the very weak inter-layer bonds, water and other polar fluids can enter between layers, causing the smectite particles to break down into particles that are usually only one or two unit layers thick (64). They usually occur as roughly equidimensional flakes and are typically less than 1-2 μm in length.

The hydrous mica clay mineral, illite, is the most common of the clay minerals found in soils and has a structure similar to muscovite. The basic illite layer is composed of two silica tetrahedral sheets between which is sandwiched an octahedral sheet. The structure is similar to that of smectite with the tips of the tetrahedra pointing towards the center of the unit layer. The hydrous mica structure differs from that of smectite with the tips of the tetrahedra pointing towards the center of the unit layer. The hydrous mica structure differs from that of smectite, however, because in hydrous mica some of the silicon atoms are replaced by aluminum atoms and the resulting charge deficiency is balanced by potassium ions which take up positions between unit layers. In muscovite about one-fourth of the silicon positions are occupied by aluminum. Illite differs from muscovite in that there is less substitution of aluminum for silicon, there is less interlayer potassium, there is some randomness in the stacking of layers in illite and the size of illite particles occurring naturally is very small (71).



EXCHANGEABLE CATIONS
 nH_2O



○ OXYGENS ○(OH) HYDROXYLS ● ALUMINUM, IRON, MAGNESIUM
 ○ AND ● SILICON, OCCASIONALLY ALUMINUM

FROM GRIM (27)

Figure 4. Diagrammatic sketch of the structure of smectite.

A diagrammatic sketch of the structure of muscovite is shown in Figure 5. The unit cell is electrically neutral and has the formula $(\text{OH})_4\text{K}_2(\text{Si}_6\text{Al}_2)\text{Al}_4\text{O}_{20}$. Because some of the charge deficiency in hydrous mica is balanced by the interlayer potassium ions, its cation exchange capacity is relatively low, ranging between 10 and 40 meq/100 grams. In addition, the interlayer bonding by the potassium is sufficiently strong that swelling does not occur in the presence of a polar fluid.

Soil Water

Water is strongly attracted to clay mineral surfaces. Evidence for this lies in the observations that dry clays adsorb water from the atmosphere even at low relative humidities, and temperatures in excess of 100°C are needed to remove all of the water from soil. In addition, as will be shown, the structure of water in the vicinity of the surfaces of the clay minerals is different from that of normal water (57, 71).

The importance of these observations to the present study lies in the fact that shearing resistance in soils is developed at interparticle contacts. The nature of the water in the vicinity of the contacts and the strength with which the soil holds the water to its surface will strongly influence the behavior of the contacts when subjected to normal and shear stresses.

Water (H_2O) is a polar molecule with the sites of the two hydrogen nuclei being poles of positive charge and the sites of two pairs of outer shell oxygen electrons being poles of negative charge. In water the positive corner of one molecule attracts the negative corner of another. This results in hydrogen bonding between molecules and causes a tendency for each molecule to bond to four neighboring molecules which surround it tetrahedrally (71). Most of water's special properties are due to hydrogen bonding. Interestingly, there is as yet no clearly correct, rigorous theory for the structure of liquid water. Several theories have been proposed. These have been discussed by Eisenberg and Kauzman (21) and Mitchell (71).

Due to their dipolar character, water molecules are attracted to ions in solution. If the attraction is strong enough, ion hydration will occur. Most common cations found in soils hydrate, a condition in which water molecules close to the ion are strongly oriented in the ion's electromagnetic field and have little kinetic energy. The influence of the ion on the water structure decreases with increasing distance from the ion.

As previously noted, clay mineral particles attract water. Lambe (46) suggests that soil-water interactions arise from the attraction of the dipolar water molecule to the electrically charged soil particle, as well as the attraction of the dipolar water to the cations in the double layer, which are in turn attracted to the particle surfaces. Low (54) suggested

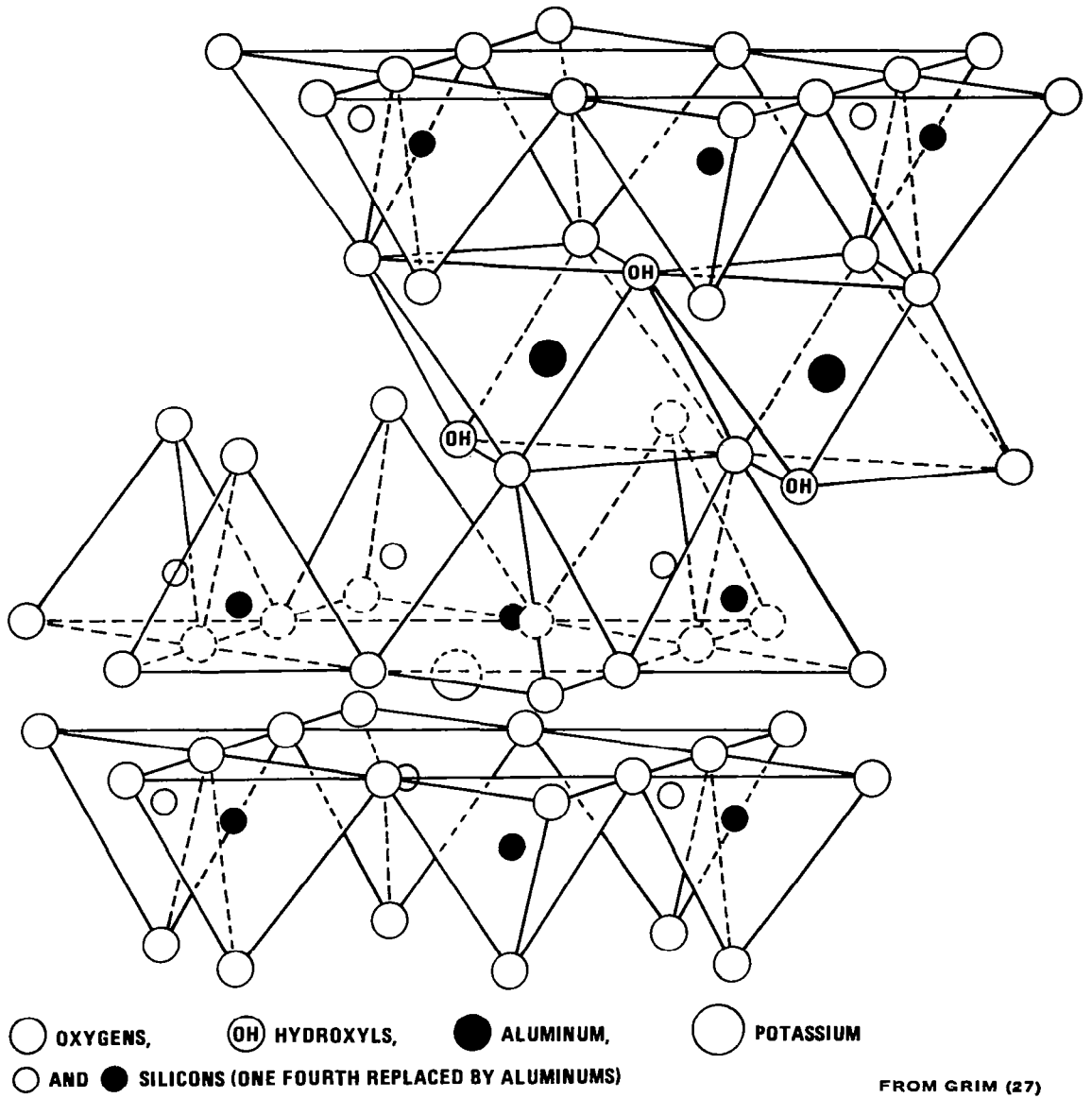


Figure 5. Diagrammatic sketch of the structure of muscovite.

four possible mechanisms for clay-water attraction. These mechanisms, summarized by Mitchell (71), and illustrated in Figure 6, include hydrogen bonding between the clay particle surfaces and the water molecules, attraction of hydrated ions by the clay particles, attraction by osmosis due to concentration gradients of cations and anions within the diffuse double layer and charged surface-dipole attraction. It appears that hydrogen bonding is a major contributor to the total water adsorption. It should also be noted that van der Waals attractive forces may also make some contribution to the attraction between the water molecules and the clay particles.

In order to fully understand the role of soil water in determining the stress-deformation behavior of cohesive soils it is necessary to understand the physical behavior of the strongly adsorbed water layer which coats the clay mineral surfaces to a thickness of several Angstroms (56, 93)³. Unfortunately the exact nature and structural properties of the adsorbed water layer remain unclear.

There have been a relatively large number of studies of the adsorbed water layer, including studies of its density, thermodynamic properties, dielectric and magnetic properties and its X-ray diffraction properties, along with diffusion and fluid flow experiments. These have been summarized by Martin (57), Low (54) and Mitchell (71). As Mitchell points out, however, most of the data from water structure studies or the interpretations of the data may be questioned or are ambiguous.

Due to these experimental uncertainties several different models for the structure of adsorbed water have been proposed, none of which have received unequivocal support as yet. It is definitely known that the structure of adsorbed water is different than that of normal water. The two soil water models which seem to describe the experimental data best are the two-dimensional fluid model and the solid water model. According to Martin (57), in the two dimensional fluid model the adsorbed water can resist appreciable normal stresses. However, when subjected to a shear force the adsorbed water will deform, perhaps even more easily than normal water.

Solid water is seen as having a more ordered structure than liquid water. Mitchell (70) notes that the structure definitely isn't that of ice. The bonds in solid water are presumed to be stronger than in normal water owing to the polarization of the clay particle surfaces (57). Because of these bonds, solid water would be able to resist both normal and shear forces to a larger degree than normal water.

³In addition to the layer of strongly adsorbed water, soil water is also composed of double-layer water and free water (46). The double-layer water component is all of the water in the diffuse double layer excluding the strongly adsorbed water. This water, while attracted to the clay particles, is not nearly as strongly held as the adsorbed water nor does it have the unusual properties attributed to the strongly adsorbed water. Free water is sufficiently distant from the clay particle surface that it is not influenced by the presence of the clay particle surfaces.

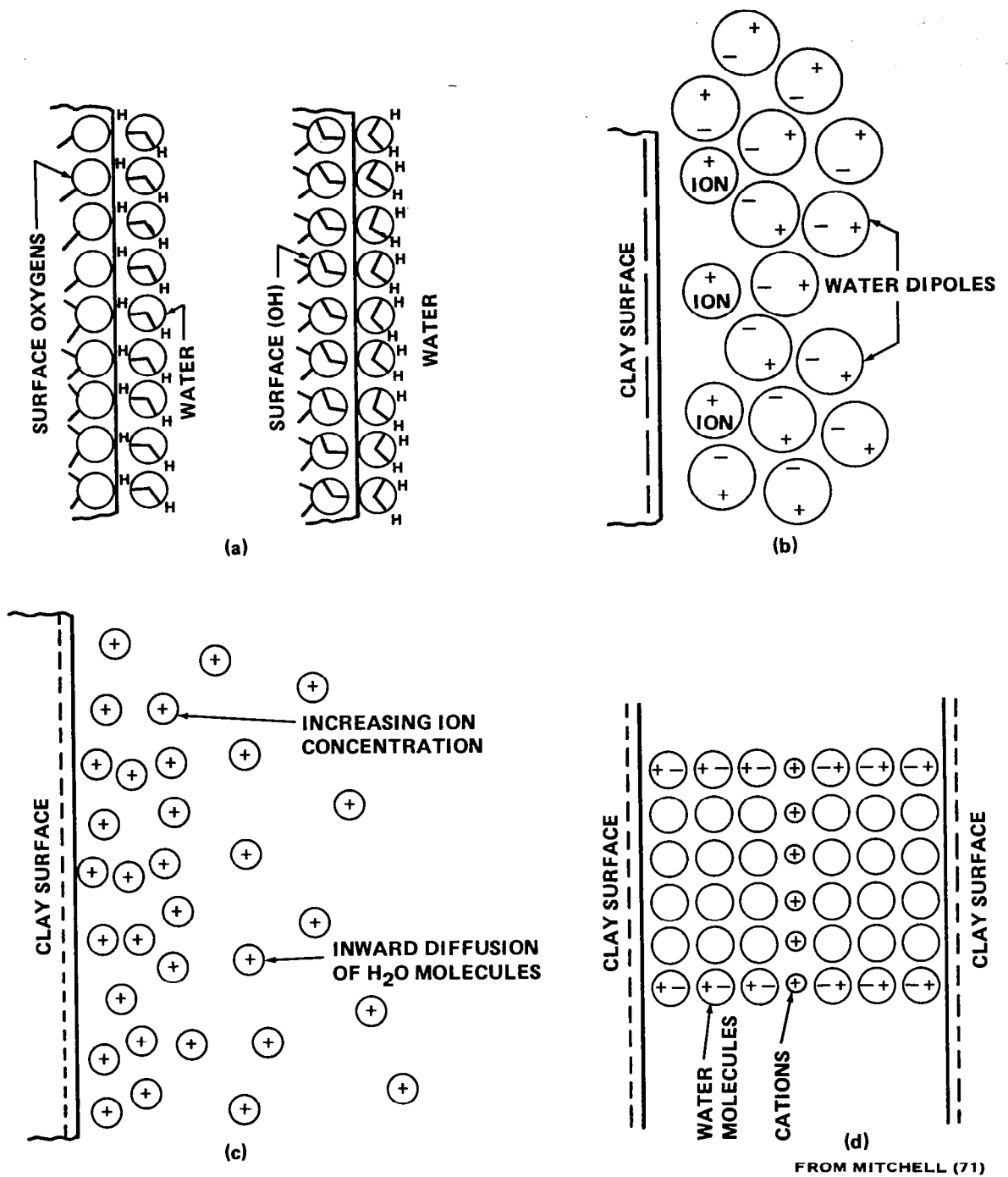


Figure 6. Possible mechanisms of water adsorption by clay surfaces. (a) hydrogen bonding. (b) ion hydration. (c) attraction by osmosis. (d) dipole attraction.

Mitchell (71) notes that the structure of adsorbed water could be different in soils at low water contents than in soils at high water contents (saturated soils). At very low water contents there would exist intense competition between the clay surfaces and the adsorbed cations for the available water molecules. This competition might lead to a large degree of disorder in the arrangement of water molecules and a high lateral mobility, analogous with the two-dimensional fluid model. At higher water contents, the adsorbed cations diffuse from the surface, the normal water structure exerts a stronger influence on the adsorbed water structure, and, therefore, there is less disorder in the adsorbed water layer at high water contents than at low water contents.

The exact role of the adsorbed water layer in determining the stress-deformation behavior of cohesive soils is largely dependent on the nature of the interparticle contacts. If the contacts are largely solid to solid, as a considerable body of evidence seems to show, then the adsorbed water will be of only secondary importance (70). However, if contacts are through adsorbed water layers, as described by Pusch (82) for instance, the adsorbed water layer will be of primary importance in defining the stress deformation behavior. This important question, concerning the nature of interparticle contacts, will be discussed in detail in a subsequent section of this report.

Interparticle Forces

It was pointed out in the preceding section that a soil develops the ability to resist both normal and shear stresses at its interparticle contacts. The strength of these contacts is, in turn, largely controlled by the interparticle forces that are acting at them. Because of the complex nature of the soil-water system there is a fairly large number of both attractive and repulsive forces that may influence the net force acting at an interparticle contact. These forces will be discussed below. For the present discussion it is convenient to think of the clay particles as having a plate-like shape.

1. Long-Range Attractive Forces. Long-range (in the present context long-range forces act over distances greater than 20\AA , short-range forces over distances less than 10\AA) attractive forces arise due to fluctuating electrical moments which develop in all units of matter. The forces associated with these statistical dipoles are called van der Waals forces.

There are three components to the van der Waals forces acting in soils, the orientation component, the induction component and the dispersion component. Only the dispersion effect is thought to be of importance in non-polar clay particles. The equation for the force acting between two parallel infinite flat plates due to the dispersion component is (106):

$$V_a = \frac{A}{48\pi} \left[\frac{1}{d^2} + \frac{1}{(d + \delta)^2} - \frac{2}{(d + \frac{\delta}{2})^2} \right] \quad (1)$$

where d is the half spacing between plates, δ is the thickness of the plates and A is the van der Waals constant, which is in the range of 10^{-11} to 10^{-14} ergs (Mitchell, 1976).

Mitchell (71) summarized much of the work that had been done on van der Waals forces between soil particles. The instantaneous dipoles are frequency dependent. Although this invalidates equation 1 on theoretical grounds, the equation is a good approximation for particle separations of less than about 1000A. In addition, for the general case in which two bodies are separated by a pore fluid, Lifshitz (1955, 1961), cited by Mitchell (71), shows that the resulting attractive van der Waals forces are strongly dependent on the dielectric constants of the pore fluids. Moore and Mitchell (75) presented the results of triaxial and vane shear tests on soils with pore fluids at various dielectric constants which appear to support the Lifshitz theory.

2. Long-Range Repulsive Forces. The only repulsive forces that are thought to be significant over interparticle spacings greater than 20A are the diffuse double-layer forces. The diffuse double layer consists of a particle's negatively charged surface and the region around the particle in which the concentration of ions is different than that of the bulk pore fluid. The negative surface potentials exhibited by the clay minerals are largely the result of isomorphous substitution. Cations are attracted, and anions are repelled from the clay particle surfaces in order to establish electrical neutrality. This leads to an increase in the concentration of cations in the vicinity of the particle in excess of the concentration existing in the bulk pore fluid. Similarly, there is a decrease in the concentration of anions.

The distribution of ions within the diffuse double layer is governed by two opposing tendencies. Firstly, there is a Coulombic attraction of the positively charged cations to the negatively charged particle surfaces and a repulsion of negatively charged anions. In opposition to the Coulombic forces there exists within the diffuse double-layer diffusion gradients caused by the uneven distribution of ions. These gradients cause cations to try to diffuse from regions of high cation concentrations to regions of low cation concentrations in order to achieve the condition of minimum free energy. The net result of these interacting forces is a distribution of ions adjacent to the clay particle surfaces of the form shown in Figure 7.

Mathematical descriptions for the ion distribution within the diffuse double layer have been developed through application of the Boltzmann equation, which relates the concentration of ions at equilibrium to their positions within an electric field and Poisson's equation, which relates the field strength to the charge density at a point in an electric field. The equation that results when the Boltzmann equation is combined with the Poisson equation has been solved for the ideal cases of either planar or spherical surfaces.

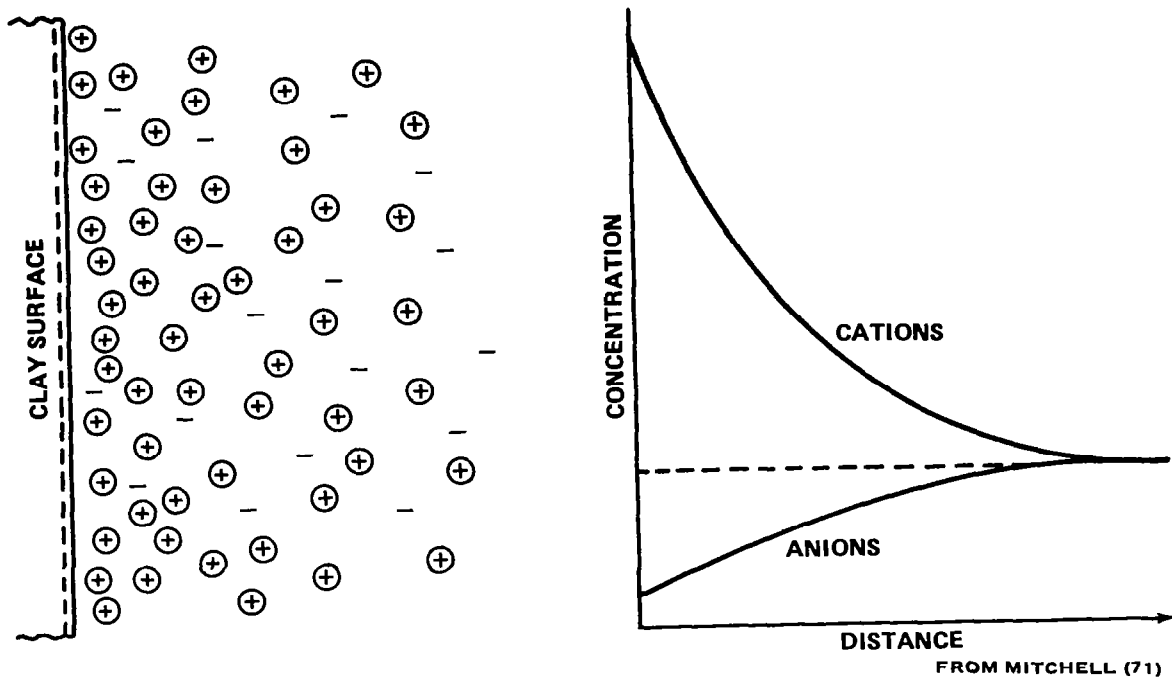


Figure 7. Distribution of ions adjacent to a clay surface according to the concept of the diffuse double layer.

Several simplifying assumptions are made in deriving the equation for the thickness of the double layer. These assumptions have been discussed by Mitchell (71). For the case of a planar clay particle, the "thickness" of a single (no interaction with adjacent double layers) diffuse double layer can be expressed approximately by (71):

$$x = \left(\frac{DKT}{8\pi\eta_0 e^2 v^2} \right)^{1/2} \quad (2)$$

where

- x = thickness of the double layer
- D = dielectric constant of the pore fluid
- K = Boltzmann constant = 1.38×10^{-16} erg/°K
- η_0 = bulk pore fluid electrolyte concentration
- e = unit electric charge = 1.6×10^{-19} coulomb
- v = cation valence.

Long-range repulsive forces develop in clay-water-electrolyte systems due to interactions of double layers of like charge. The magnitude of the repulsive forces between two parallel particles can be calculated by applying the van't Hoff equation to the midplane ion concentration between the particles. The van't Hoff equation describes the osmotic pressure existing at the midplane between particles due to the difference in ion concentration between the midplane and the bulk pore fluid. The osmotic pressure is equated with the repulsive pressure between particles. It should be pointed out that although the osmotic pressure approach has been successful in predicting interparticle repulsive forces under some very specialized conditions, its limitations are great enough that it cannot predict repulsive forces between soil particles in most soils under most conditions. These limitations have been discussed by Quirk (89), Bailey (8) and Mitchell (71).

Although the quantitative usefulness of the various double layer equations is limited, qualitatively they provide much insight into fine-grained, cohesive soil behavior. In addition, the importance of the double layer is greatest in clays at very low stress levels, when the applied stresses are small, and in the determination of initial soil fabric and structure. Therefore, the understanding of double-layer forces even qualitatively, is important to an understanding of the strength-deformation behavior of clays at low stress levels.

The effect of an increase in the thickness of the double layer is to increase the midplane ion concentration between particles at a given separation distance relative to the ion concentration in the bulk pore fluid and thus to increase interparticle repulsive forces. A decrease in double-layer thickness leads to decreased interparticle repulsions.

Equation 2 can be analyzed to see which variables significantly affect the diffuse double-layer thickness. It can be seen that an increase in the pore fluid electrolyte concentration leads to a decrease in double-layer thickness. Similarly, the larger the valence of the adsorbed cations, the smaller the double layer. It has been shown for instance, that multivalent cations are preferentially adsorbed by clay particles and that the addition of only a small amount of di- or trivalent cations to a clay-water-monovalent cation system can have a large effect on the physical properties of that soil-water system. Increases in the dielectric constant lead to increased double-layer thicknesses and increased repulsions. Increases in temperature should theoretically lead to increases in double-layer thickness. However, an increase in temperature also leads to a decrease in dielectric constant and thus the net effect of temperature is small.

In addition to the variables used in equation 2, there are other factors which influence double-layer behavior (71). These factors include the effects of secondary energy terms, adsorbed water, ion size, pH, and anion adsorption. Of these, the effects of ion size and pH appear to be the most important.

The double-layer equation presented previously treats ions as point charges and does not take into account the fact that they have a finite size. Due to this assumption equation 2 predicts the same double-layer thickness for cations of the same valency but with different diameters. In actuality, the cations with the smaller diameter will have a double-layer thickness which is less than that for cations with the larger diameter.

Low pore fluid pH may lead to positive edge charges on the clay particles, particularly with kaolinite, and subsequent edge-to-face flocculation of particles in clay suspensions. High pH leads to negative surface potentials over the entire particle surface and the development of dispersed fabrics during sedimentation.

3. Short-Range Attractive Forces. Short-range forces or contact forces are considered to act within about 10\AA of a clay particle's surface. These forces include van der Waals attractions and edge-to-face electrostatic attractions, which were discussed previously, along with possible primary valence bonding, ionic lattice attraction and cation linkage, and cementation.

It is not clear if primary valence bonding occurs between soil particles. If it does, these bonds would be very strong. If it does occur, it is thought that the hydrogen and oxygen surface atoms of the clay particles (and possibly the strongly adsorbed water atoms) participate in the bonding. Ionic and covalent bonds typically occur at interatomic spacings of less than 3\AA . Bailey (8) attributes ionic lattice attractions to an electrostatic linking of the negatively charged clay particles by the cations between them. In a sense, the cations take up positions in a crystalline arrangement between particles. This crystalline arrangement can only take place at small interparticle spacings. If the particles are spaced much more than 10\AA apart, cation linkage will not take place.

4. Short-Range Repulsive Forces. Repulsive forces which are important at interparticle spacings of less than 20\AA include Born repulsions, close range interactions of diffuse double layers, the hydration of adsorbed ions and clay particle surfaces, and the forces associated with the orientation of water molecules in an electric field (42).

Born repulsions result when two particles are brought into mineral-to-mineral contact. This short-range repulsive force prohibits the interpenetration of matter by postulating that an infinitely large repulsive force develops when the distance between the nuclei of two approaching atoms is approximately equal to the sum of their radii.

Adsorptive forces cause water molecules to be attracted to clay surfaces, and for closely spaced particles, lead to the development of large hydrostatic pressures in the fluid separating the particles. This effect is somewhat similar to the water pressures caused by osmotic gradients. The energy associated with surface hydration can be quite large. Mitchell (71) notes that the pressure required to remove the last few molecular layers of water between two clay plates which are being pressed together may be as high as 4000 atmospheres.

Short-range repulsive forces may also develop due to the hydration energies associated with interlayer cations in close proximity to interparticle contacts. Hydration forces decay inversely as the square of the distance. Unfortunately, many of the details of both surface hydration and ion hydration remain unknown.

Net Force of Interaction

The various long-range and short-range forces discussed on the previous few pages combine to form a net attractive or repulsive force between adjacent clay particles at any interparticle spacing. A summary of these forces is given in Table 1. It is important to realize that the net force between particles is not constant; instead, it is a very sensitive function of many environmental and compositional factors, the most important of which include particle spacing and arrangement, pore fluid chemistry and soil mineralogy. Even in a given soil with a fixed pore fluid, the net interparticle force changes every time there is a change in particle spacing or arrangement.

A large body of research exists which has been devoted to developing an understanding of the roles of the various long- and short-range interparticle forces in influencing the behavior of fine-grained, cohesive soils. Because of the many forces that are present in the clay-water-electrolyte system and the complex manner in which these forces interact and respond to changes in the chemical and physical environment this understanding is still far from complete.

Early studies into the nature of interparticle forces centered mainly on the roles of diffuse double-layer repulsive forces and van der Waal's attractive forces. It was originally thought that many of the properties of fine-grained soils could be interpreted quantitatively in terms of these forces and their variation due to changes in compositional and environmental factors (70). More recent studies have shown that this approach was an over-simplification.

Presently, the view is held that in large measure the long-range and short-range forces play fundamentally different roles in controlling soil behavior. The long-range interparticle forces are seen as being critical in the establishment of the initial fabric in high water content clay systems. They also play a role in determining the compressibility of a few types of soils under very specialized conditions (8, 71, 89), in the development of sensitivity (81, 94) and thixotropy (67, 83), and in the rearrangement of particles during shear (63). Short-range forces are seen as controlling the mechanical behavior of clays in the stress ranges of usual engineering interest (70).

Probably the most important role of long-range interparticle forces is the formation of a soil's initial fabric. In clay-water-electrolyte systems in which the long-range attractive forces exceed the long-range repulsive forces flocculated fabrics are likely. If, instead, the repulsive forces exceed the attractive forces, dispersed fabrics are likely. Flocculated fabrics are characterized by relatively large aggregates of particles connected to each other through linkage assemblies (85). Within the aggregates particle contacts are edge-to-face, edge-to-edge and face-to-face.

TABLE 1. TYPES OF INTERPARTICLE FORCES

FROM LADD AND KINNER (45)

- I. Long-Range (Interparticle spacing of greater than 20\AA)
 - A. Repulsive (Double-layer osmotic repulsion)
 - B. Attractive (van der Waals dispersion component)
- II. Short-Range or Contact Forces (Interparticle spacing of less than 20\AA)
 - A. Repulsive
 1. Osmotic repulsion
 2. Surface hydration
 3. Ion hydration
 4. Orientation of water in an electric field
 5. Born repulsion (if actual mineral to mineral contact occurs)
 - B. Attractive
 1. van der Waals dispersion component
 2. Edge-to-face electrostatic attraction
 3. Primary valence bonding (if actual mineral-to-mineral contact occurs)
 4. Ionic lattice attraction, cation linkage (only for parallel particles)
 5. Cementation

Dispersed fabrics contain a large number of small aggregates which take up positions in close proximity to each other. The close packing of aggregates in dispersed structures leads to small pore spaces and lower void ratios than found in flocculated structures at the same effective stress (71).

The role of long-range forces in determining initial fabric is well illustrated by Fig. 8, which shows the volume of kaolinite that was obtained from an initial mixture of one gram of dry clay and 30 grams of water. As can be seen, at low values of pH the sediment volume is much larger than at high values of pH. This is a reflection of a flocculated soil fabric at low pH and a dispersed fabric at high pH. The flocculated fabric is the result of strong edge-to-face electrostatic attractions in acidic environments. At low pH values the edges of the kaolinite particles take on a positive charge and are thus attracted to the negatively charged faces of neighboring clay particles. At high values of pH, the edge charge on the particles becomes negative and instead of having an edge-to-face attraction there is a net edge-to-face repulsion between the kaolinite particles. These increased repulsive forces result in a dispersed fabric, small pore spaces and therefore a smaller sediment volume.

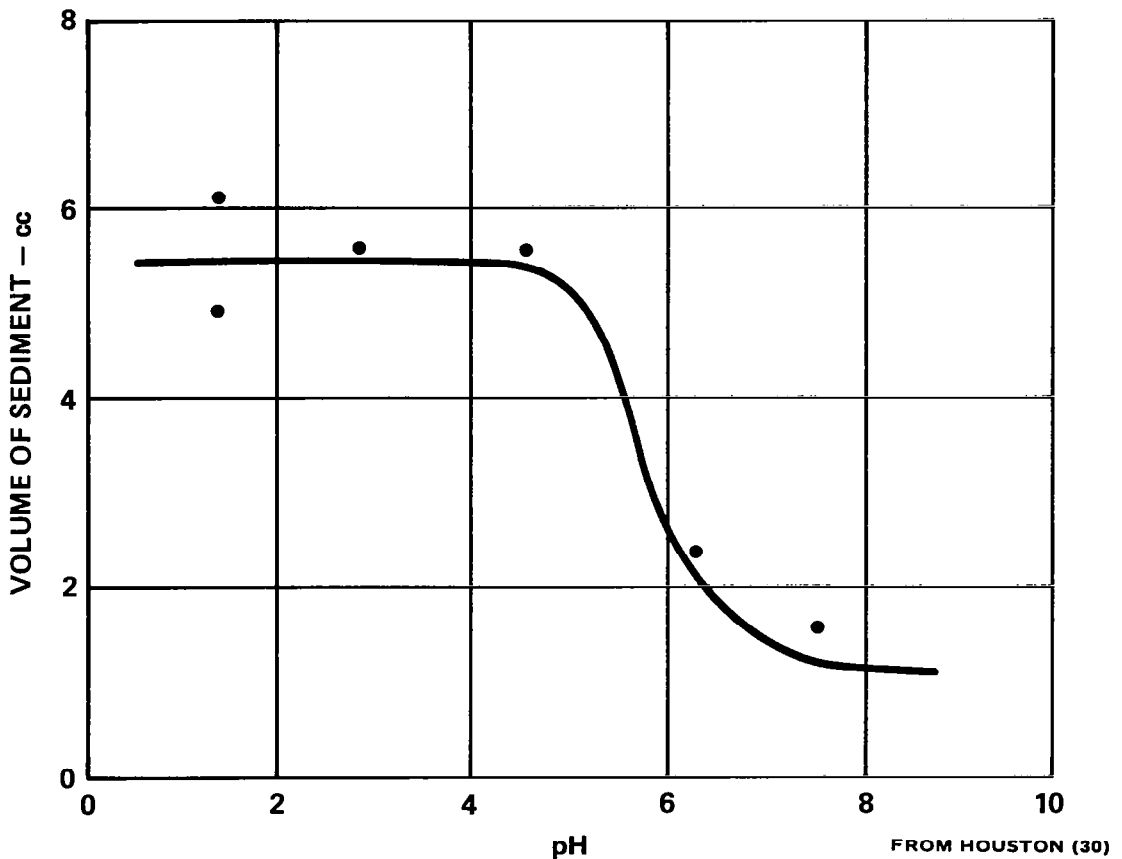


Figure 8. Volume of sediment versus pH of mixtures for kaolinite.

Mitchell (71) reviewed the work of several researches and found that while theoretically and experimentally determined consolidation-rebound curves may be in good agreement for very fine ($< .2 \mu\text{m}$) fractionated pure clays, diffuse double-layer theories cannot predict the consolidation-rebound behavior of larger size clay particles and most natural clays. Mitchell cited several reasons for the discrepancy between theory and experiment, including the deviations from assumed parallelism between clay plates, cross-linking, the effects of other long- and short-range forces, physical interference between particles and the possible effects of impurities such as organic matter.

Short-range interparticle forces are primarily responsible for strength generation in cohesive soils (63, 70, 80). Short-range forces are transmitted between particles at interparticle contacts. The short-range forces control the intergranular stresses that develop at contacts and thus the true area of contact. This, in turn, strongly influences the contact shear strength.

Except in relatively rare circumstances, the knowledge gained by studying the various interparticle forces is qualitative in nature. The use of theoretical force-distance equations to predict the shear strengths of soils is not possible for many reasons. One reason is that the theories used to describe the various forces found in soils are often only approximate and make assumptions that are not met in most soil-water systems. As an example, due to uncertainty in the value of the van der Waals constant, van der Waals forces between adjacent particles cannot be predicted within an order of magnitude (8). Lambe (46), Mitchell (71), Bailey (8), Quirk (89), Ladd and Kinner (45) and Ingles (31) outline other limitations and inaccuracies associated with the various force theories.

Another reason for our inability to calculate interparticle forces is our lack of knowledge of interparticle distances, orientations and groupings. As Ingles (31) points out, the strengths developed by the various force mechanisms are strongly dependent on interparticle spacings. The complexities of many particle fabrics would make an accurate assessment of interparticle spacings and orientations almost impossible. Finally, difficulties in quantifying the magnitudes of interparticle forces arise due to the uncertain roles played by adsorbed water and surface contaminants.

Even though interparticle forces cannot be quantified to the degree desirable, a qualitative understanding of the various forces is an important component in understanding soil behavior at very low applied stress levels. Experiments in which the various factors which influence interparticle forces—such as pore fluid chemistry—are altered yield results which give information not only on the relative importance of the various interparticle forces, but also information on fundamental mechanisms which might contribute to true cohesion and true tensile strength in soils.

III. STRENGTH GENERATION IN FINE-GRAINED, COHESIVE SOILS

Introduction

There have been only a limited number of experimental investigations concerned with the strength and stress-strain behavior of cohesive soils under conditions of very low effective stress. These tests have been performed on undisturbed, laboratory sedimented, and remolded soils. The results of tests on undisturbed soils are often dominated by cementation bonds, which can account for most, if not all of the cohesion or tensile strength in some cases.

Tests on remolded soils or laboratory sedimented soils offer the advantage of minimizing the cementation component of strength. This is a significant advantage in a study to ascertain mechanisms which might account for true cohesion in soils. Obviously one disadvantage associated with testing remolded soils is that the behavior of the remolded soil is substantially different than that of the undisturbed soil and therefore is not representative of the field behavior of the soil. As an illustration of the potential difference in the behavior of a soil in the remolded state and in the undisturbed state, at low effective stress, the failure envelopes for intact and remolded blue London Clay are compared in Figure 9. These results were obtained by Bishop and Garga (10) using both undrained unconfined compression tests with negative pore pressure measurement and drained direct tension tests. The authors attribute the difference in failure envelopes to the destruction of cementation bonds caused by remolding.

Evaluation of the stress-strain-strength behavior of cohesive soils at low stress levels is generally quite difficult because the strengths are usually quite small and because it is difficult to devise a testing system that can accurately apply the small normal and shear forces on a potential failure plane that are needed to define the failure envelope near the stress origin. Ideally, in order to determine the true cohesion of a soil, the soil should be tested in a manner such that the effective normal stress on the failure plane at failure is zero. Conventional triaxial compression tests cannot be performed while maintaining zero effective stress on the failure plane.

In addition to the many testing difficulties associated with trying to determine a soil's true cohesion, the evaluation of the results of any tests performed at low stress levels is made difficult due to the presence of gravity-induced body stresses within the soil specimen. As an example, in a typical triaxial specimen having a height of 8.9 cm and a diameter of 3.6 cm the normal stress on the failure plane in a test in which the soil's angle of internal resistance is 30° , varies by 4.5 g/cm^2 between the top and bottom of the specimen, while the shear stress varies by 9 g/cm^2 . The effects of gravitational body forces can be minimized by using small samples; however, the use of small samples may lead to non-uniform stress and strain fields within the specimen, nonuniform boundary conditions and difficulties in measuring significant parameters (102, 104).

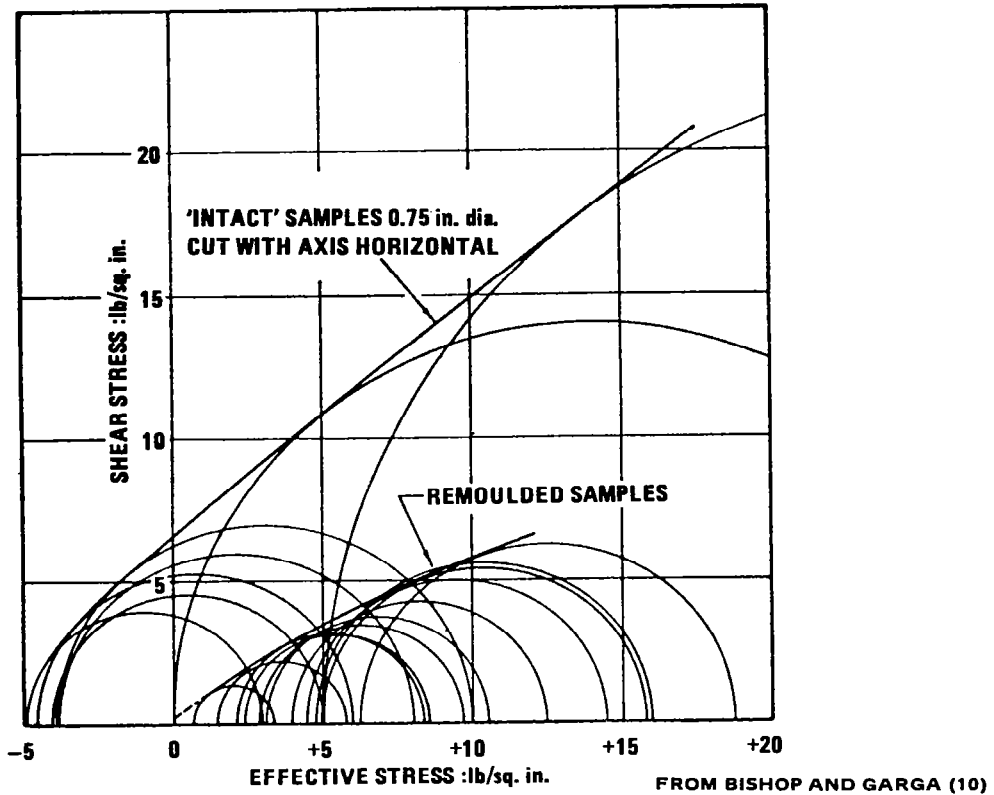


Figure 9. Results of drained tension and compression tests on 'intact' Blue London Clay compared with the results of compression tests on remoulded samples.

From the foregoing discussion and from other discussions in the literature (41, 71, 102, 104), it can be seen that measuring soil strength in the region of the stress origin is a difficult proposition.

Fundamental Mechanisms of Shear Strength Generation in Fine-Grained Soils

Studies of the fundamental mechanisms of shear strength generation in fine-grained, cohesive soils are based on the nature of interparticle contacts and adsorbed water, microscopic aspects of friction and adhesion and the role of interparticle forces and soil fabric in developing shearing resistance.

1. Rosenqvist's Model. One of the early microscopic models for the development of shearing resistance in cohesive soils was the one developed by Rosenqvist (93) and others at the Norwegian Geotechnical Institute. Rosenqvist postulated that clay particles in a clay-water system are surrounded by a tightly held, highly structured layer of adsorbed water. This postulate was based on measurements of the heats of wetting of various clays. He found that the free energy of adsorbed water in

clay is lower than the free energy of normal water but higher than that of ice. In addition, the free energy increased with distance from the particle surface. Based on exchange reaction experiments (77) between hydroxyl ions on the surfaces of dry clay minerals and the deuterium atoms of added heavy water (and assuming a relationship between diffusion rates and viscosity), Rosenqvist concluded that the viscosity of the adsorbed water layer increases with decreasing distance from the mineral surface. Rosenqvist suggested that clay particles in a clay water system be thought of not as rigid silicate sheets but instead as combination clay-water particles. Rosenqvist's clay-water particle consisted of a clay particle surrounded by immobilized hydrogen and oxygen atoms in a more or less rigid arrangement. This "adsorbed water envelope" was thought to have a yield strength which decreased with distance from the soil particle surface.

Rosenqvist incorporated his findings with Bowden and Tabor's (14) adhesion theory of friction to postulate the following strength mechanism. When two grains of soil are brought into close proximity, adhesion bonds will form independent of whether water is present or not. In massive structured minerals such as quartz, or in clays in which the pore fluid is nonpolar, elastic stresses will arise at the interparticle contact points. In these materials, when the external stresses are released, the stored elastic strain energy will tend to separate the mineral grains and the soil will behave like a frictional material.

If the clay particles are in an aqueous solution, they will be surrounded by an adsorbed water layer. When two particles are pressed together, they will contact first through the adsorbed water. The adsorbed water will yield plastically and little or no elastic stresses will arise in the contact region. During unloading there is no stored elastic energy to separate the particles (although as discussed earlier, other repulsive forces may be present which would cause the particles to move apart), and the clay particles will stick together due to adhesional forces. In dry clays or clays with nonpolar pore fluids, contacts would be mineral to mineral, and contact stresses would therefore be predominantly elastic. These materials would behave similarly to quartz, that is, as purely frictional systems with no cohesive component of strength.

Rosenqvist's view that cohesion was the macroscopic manifestation of the plastic yielding of adsorbed water layers did not find complete acceptance among engineers and scientists. Michaels (65) argued that while the physical state of adsorbed water was certainly different than that of normal water, the experimental evidence did not fully support Rosenqvist's view of a highly structured, highly viscous, immobile adsorbed water layer. Instead, he suggested that the adsorbed water might be anisotropic in nature. Michaels also doubted that adsorbed water functioned as a molecular glue which was directly responsible for cohesion.

In support of this position he noted that dry clays are stronger than wet clays, that the adsorption of water reduces the surface forces on the clay particles, and thus should reduce adhesional forces between particles, and that the attraction between two dipolar surfaces separated by less than about 10Å will be reduced when separated by a dipolar fluid. Michaels argued that interparticle adhesion occurs in spite of, rather than because of the presence of water.

In Michaels view, adhesion will occur simply if two clay particles are brought into sufficiently close proximity. Thus, the main strength-determining factor is the number of interparticle adhesive bonds, which is mainly a function of the geometric arrangement of particles and their spacings. The pore fluid chemistry is seen as being primarily responsible for the initial soil structure, which controls the initial number of interparticle contacts. Chemistry usually plays only a secondary role in determining the strength of a contact. Michaels also noted that under many circumstances capillary tension in the pore fluid would make an important contribution to soil cohesion, although by definition, capillary tension is not considered to be a component of true cohesion.

2. Lambe's Model. Lambe (47) presented a picture of shear strength development in cohesive soils based on considerations of interparticle forces, interparticle contacts and particle kinematics. Strength was seen to be generated by three different mechanisms, cohesion, friction and dilation. True cohesion, which Lambe defined as shear strength in the absence of any externally derived normal pressure, was attributed to net interparticle attractive forces. He outlined five interparticle forces which might contribute to a true cohesion. These included:

a. Salt flocculation. This condition prevails when the pore fluid electrolyte concentration is high enough to allow the clay particles to move into face-to-face, or more typically edge-to-face contact.

b. Edge-to-face flocculation. This type of flocculation occurs when the edges of the clay particles take on a positive charge. These edges are then attracted to the negatively charged faces of other clay particles.

c. Hydrogen bonding between kaolinite sheets. This results when two adjacent kaolinite particles are brought into close proximity with the outermost sheet of one of the particles being composed of oxygen atoms, while the outermost sheet of the other particle is composed of hydroxyl ions. The net tendency of this force is to make large particles out of several smaller ones.

d. Bonding by potassium ions between unit layers of hydrous mica. The net effect of this force is also to make large particles out of several smaller ones.

e. Cementation.

In Lambe's model, cohesion is mobilized at very small strains. After some critical strain level, the cohesive bonds are broken and the subsequent contribution of cohesion to the shearing resistance developed by a soil is zero. This concept is illustrated in Figure 10. Lambe points out that cohesion can be variable. It is sensitive to several compositional and environmental factors.

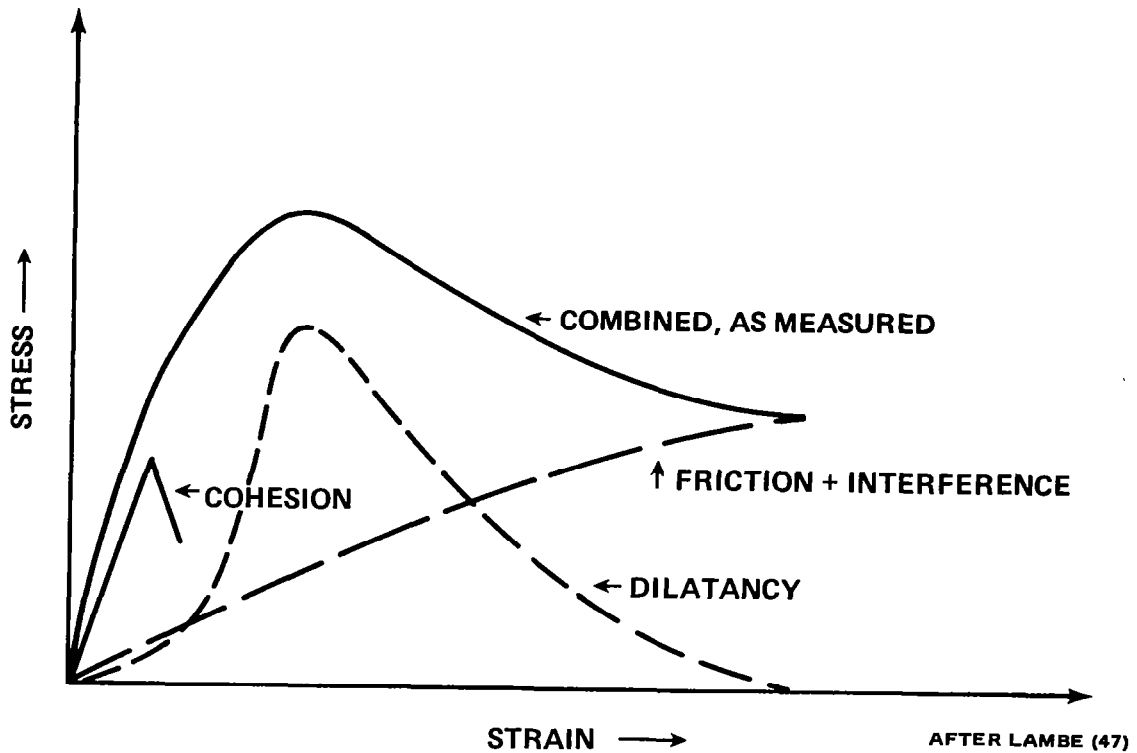


Figure 10. Components of shear resistance.

Lambe also notes that not all of the effects of cohesion are observable as a cohesion intercept on a Mohr-Coulomb plot (a diagram showing combinations of shear stress and normal stress that cause friction). He suggests that an important role of cohesion is to make larger soil particles out of smaller ones. These larger particles, when sheared, develop more frictional resistance and physical interference and therefore reflect an increase in shearing resistance through an increase in friction angle rather than an increase in cohesion intercept.

Lambe attributes interparticle friction and dilatancy to the relative movements of soil particles. In clays, dilation and friction arise from several sources, including particle rearrangements, macro- as well as microdilatancy and true frictional resistance between particle surfaces. He visualizes true friction as being due to local charge variations on adjacent clay particle surfaces. To move one particle with respect to another requires that a shear force be applied in order to move charged

surface atoms through a varying electric field. Both dilatancy and friction are seen as being direct functions of the normal force acting on a shear surface. The normal force, in turn, is considered to be a function of the externally applied stresses, the pore air and water pressures and the attractive and repulsive forces acting between particles.

In the late 1950's and early 1960's geotechnical engineers borrowed theories from physical chemistry, statistical mechanics and surface physics in order to better understand the fundamental mechanisms of shearing resistance in soils. Two of these theories in particular, the theory of absolute reaction rates and the adhesion theory of friction, are germane to the present study and will be discussed in some detail.

3. Rate Process Theory. The rate process approach to soil deformation is based on the theory of absolute reaction rates first proposed by Arrhenius to describe the rates of chemical reactions. The theory which has evolved since Arrhenius' time is generally accepted by physical chemists as a relationship which describes the temperature dependence of the rates of most chemical reactions as well as certain physical processes (26).

In the application of rate process theory, a system is seen as being made up of a large number of flow units. These flow units could be atoms, molecules or larger particles. The theory stipulates that in order for these flow units to react, bond, or engage in some other physical process, they must first ascend to a certain energy level, termed the energy of activation of the reaction, or activation energy. The activation energy represents the energy that a flow unit in the initial state of a process must acquire before it can take part in the reaction, whether it be physical or chemical (26).

The flow units of any system have, at any instant in time certain vibrational energies associated with them. These vibrational energies are not constant but vary continuously in time. The average thermal energy of a flow unit is KT , where K is the Boltzmann constant (1.38×10^{-16} erg/°K) and T is the absolute temperature of the flow unit. From the theory of statistical mechanics the theoretical distribution of particle energies can be derived. This distribution, known as the Maxwell-Boltzmann distribution, indicates that the probability of any flow unit having an energy level equal to or greater than ΔF be given by the equation:

$$p(\Delta F) = C \times \exp(-\Delta F/KT) \quad (3)$$

where $p(\Delta F)$ denotes a probability, ΔF represents the activation energy of a flow unit and C is a constant. Glasstone, Laidler and Eyring (26) indicate that within moderate temperature ranges C and ΔF are constants

and C can be taken to be about 1. The Maxwell-Boltzmann probability function can be interpreted as being either the probability of any one flow unit having energy greater than ΔF or, alternatively, the fraction of the total number of flow units that possess energy greater than ΔF .

In a system satisfying the Maxwell-Boltzmann distribution the mean frequency of thermal vibration of the flow units is given by KT/h , where h is Planck's constant (6.624×10^{-27} erg/sec). The mean frequency with which a given flow unit reaches or exceeds the activation energy, ΔF , is given by the product of the mean frequency of vibration and the probability that on any given vibration the flow unit energy is greater than ΔF . Thus, the frequency, ν , with which a flow unit obtains an energy level equal to or greater than the activation energy is given by the equation:

$$\nu = (KT/h) \text{EXP}^{-\left(\frac{\Delta F}{KT}\right)} . \quad (4)$$

In the application of rate process theory to the deformation of soils it is postulated that within a soil mass, shearing stresses are resisted by interparticle bonds which develop within interparticle contact zones. A single contact zone probably consists of a number of interparticle bonds dependent on the size of the contact area. The interparticle contact area is assumed to be proportional to the normal force being transmitted through the contact. These concepts have been supported experimentally (73).

In soils, interparticle bonds are considered to be flow units. Bonding reduces the free energy of a system. Therefore, each interparticle bond is assumed to represent a local minimum potential energy region. As illustrated in Fig. 11, this equilibrium position of minimum potential energy can be thought of as an energy valley which is surrounded by energy barriers of equal height in all directions (16). The displacement of a flow unit, i.e., the breaking of a bond requires the introduction of an activation energy, ΔF , which can be visualized as the energy required to climb over the energy barrier in Figure 11.

In reality, the energy barriers which flow units must surmount may vary throughout a material, thus the hill in Figure 11 represents the mean barrier height. The vibrational energy of a flow unit, which varies with time, can be thought of as movement of the flow unit up and down the sides of the surrounding energy barrier.

Acknowledging the statistical distribution of barrier heights and thermal energies, the activation energy can be thought of as the mean of the increments in energy that a flow unit must acquire in order to surmount the mean energy barrier height. In a Maxwell-Boltzmann distribution of energies there are always at least a few flow units that have enough thermal energy to surmount a surrounding energy barrier. However, in the absence of a directional potential, such as an externally applied shear force or a thermal gradient, bonds are assumed to rupture, and reform, at the same or adjacent equilibrium positions with equal frequency in all directions.

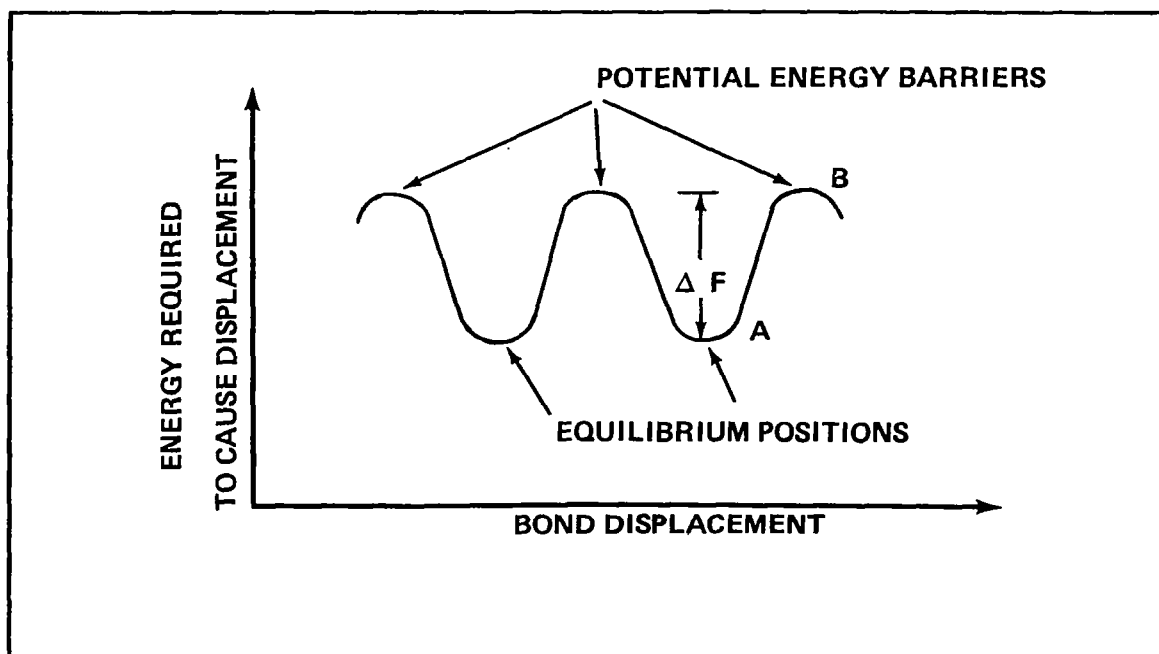


Figure 11. Pictorial representation of energy barrier and valley concept of bond displacement.

When a shearing force is applied at an interparticle contact, it has the net effect of lowering the energy barrier in the direction of the force and increasing the barrier in the direction opposite the force. The net result of the altered barrier heights is the existence of a directional potential parallel to the applied shear force. This is illustrated in Figure 12. The directional potential has the effect of changing the frequency with which flow units surmount energy barriers in the direction of the shear force.

The distorted barrier heights result because of the mechanical energy that is imparted into the flow unit by the shear force. As the flow unit vibrates around its equilibrium position, the applied shear force, f , does work on the flow unit equal to the forces times the distance of movement. If during a particular vibration, the oscillation is in the direction of the applied shear force, the work done by the shear force contributes to the energy of the flow unit. This effectively reduces the amount of energy that must be put into the flow unit to allow it to surmount the energy barrier. If instead, the flow unit is moving in a direction opposite to the shear force, the work that the flow unit must expend in order to do work against the shear force effectively increases the height of the energy barrier. Based on the situation illustrated in Figure 12, it is clear that a shear force, f , acting on a flow unit, does work on that flow unit equal to $f\lambda/2$ in moving it from its equilibrium position to the top of the energy barrier. λ is the distance between adjacent equilibrium positions.

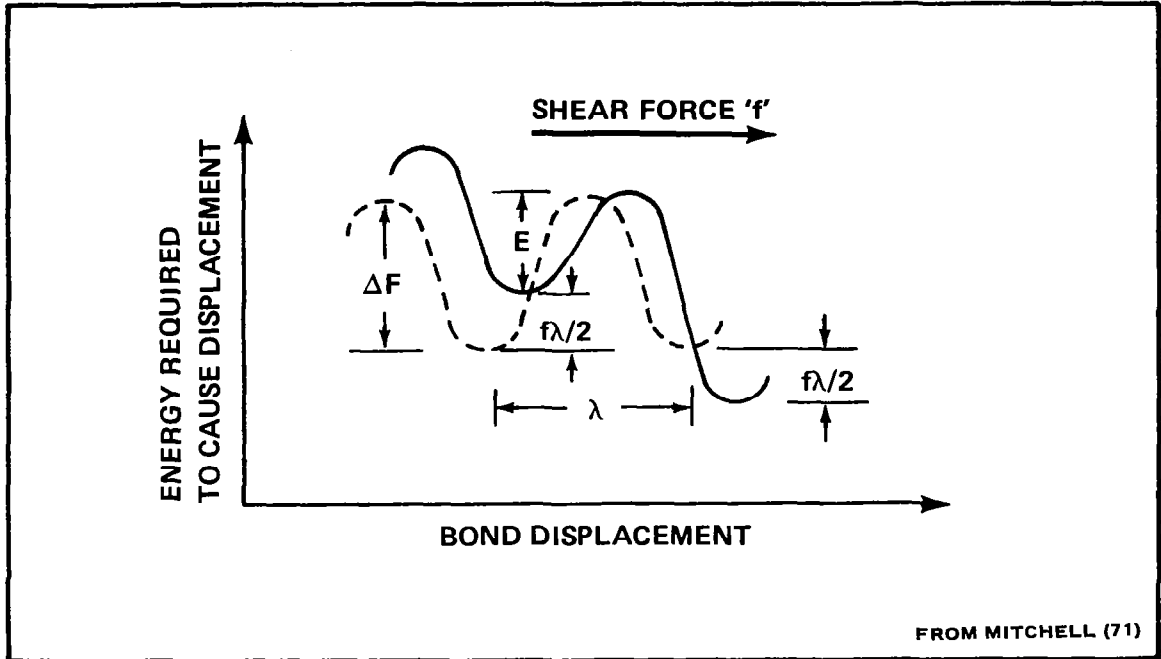


Figure 12. Distorted energy barrier resulting from a shear force acting across a bond.

As a consequence of the shear force, the energy barrier height is reduced to $\Delta F - \left(\frac{f\lambda}{2}\right)$ in the direction of the shear force and is increased to $\Delta F + \left(\frac{f\lambda}{2}\right)$ in the direction opposite to the shear force. Mitchell (68) developed equations to describe the frequency with which a flow unit is activated when subjected to a directional potential caused by a shear force. In the direction of the force the frequency is given by,

$$v_f = \left(\frac{KT}{h}\right) \text{EXP} \left[- \frac{\left(\Delta F - \frac{f\lambda}{2}\right)}{KT} \right] \quad (5)$$

and in the direction opposite the shear force,

$$v_o = \left(\frac{KT}{h}\right) \text{EXP} \left[- \frac{\left(\Delta F + \frac{f\lambda}{2}\right)}{KT} \right] \quad (6)$$

It is clear that V_f will be greater than V_o , and therefore the net frequency with which a flow unit will surmount an energy barrier in the direction of a shear force is,

$$v_f - v_o = \frac{KT}{h} \left\{ \text{EXP} \left(- \frac{\left[\Delta F - \frac{f\lambda}{2} \right]}{KT} \right) - \text{EXP} \left(- \frac{\left[\Delta F + \frac{f\lambda}{2} \right]}{KT} \right) \right\} \quad (7)$$

$$= \frac{2KT}{h} \text{EXP} \left(\frac{-\Delta F}{KT} \right) \text{SINH} \left(\frac{f\lambda}{2KT} \right) . \quad (8)$$

Mitchell (69) presented experimental evidence to show that the value of $\left(\frac{f\lambda}{2KT} \right)$ ranged between 9 and 58. When $\left(\frac{f\lambda}{2KT} \right)$ is this large, the term $\text{sinh} \left(\frac{f\lambda}{2KT} \right)$ in equation (8) can be replaced by the approximation $\frac{1}{2} \exp \left(\frac{f\lambda}{2KT} \right)$ without introducing significant error. Making this substitution, equation (8) becomes,

$$v_f - v_o = \left(\frac{KT}{h} \right) \text{EXP} \left[- \frac{\left(\Delta F - \frac{f\lambda}{2} \right)}{KT} \right] . \quad (9)$$

Of the total number of activated flow units at any instant in time, some will move to new positions of equilibrium and some may fall back into their original positions. For each unit that is successful in crossing a barrier, there will be some small displacement. Thus the rate of displacement or strain will be proportional to the net frequency of activation in a given direction. Mathematically, this proportionality can be expressed through the equation

$$\dot{\epsilon} = X(v_f - v_o) \quad (10)$$

and upon substituting equation (9) for $v_f - v_o$,

$$\dot{\epsilon} = X \frac{KT}{h} \text{EXP} \left[- \frac{\left(\Delta F - \frac{f\lambda}{2} \right)}{KT} \right] . \quad (11)$$

In these equations $\dot{\epsilon}$ represents the rate of strain and X is a constant of proportionality that may be a function of several factors (73). Equation (11) (or some other rate process equation of slightly different form) has been used by a number of investigators concerned with fundamental aspects of soil behavior. It has shown to be applicable to both volumetric (17, 107) and deviatoric (6, 68, 72) soil deformation, as well as successfully describing the erosion rates of cohesive sediments (33) and the viscous behavior of clay pastes (6, 60).

The most common application in soil mechanics of rate process theory has been in the understanding and prediction of deformation rates of soils subjected to shear stresses. In these formulations it is typically assumed that the shear force per bond is proportional to the shear stress acting on an element of soil and inversely proportional to the total number of bonds. The assumption is also made that the maximum displacement will occur along the plane of maximum shear stress. In a triaxial test the maximum shear stress is $\frac{1}{2} \sigma_d$, where σ_d denotes the deviator stress. If S denotes the number of interparticle bonds, the average shear force per interparticle bond can be written as,

$$f = \frac{\sigma_d}{2S} \quad (12)$$

which leads to an expression for the strain rate given by,

$$\dot{\epsilon} = \frac{XKT}{h} \text{ EXP } \left[- \left(\frac{\Delta F - \frac{\sigma_d \lambda}{4S}}{KT} \right) \right] \quad (13)$$

The temperature and stress dependence of strain rate that is suggested by equation (13) has been shown to be correct for soils (16, 72, 98). It should be noted that both boundary normal and shear stresses produce shear forces at interparticle contacts. Equation (13) accounts for the boundary shear forces only.

Campanella (16) and Mitchell (65) both pointed out that the shearing resistance of a soil is controlled by the number of bonds. They postulated that the number of bonds at a contact is proportional to the contact area, which in turn is proportional to the normal force being transmitted through the contact. The normal force acting across a contact is a function of both the externally applied effective stresses and the internal stresses acting across the contact. Based on these ideas, Campanella and Mitchell postulated that the number of bonds was proportional to the mean normal effective stress, σ'_m , acting on the soil. Campanella presented the equations,

$$s = a + b\sigma_M' \quad (14)$$

$$\sigma_M' = \frac{1}{3} (\sigma_1' + \sigma_2' + \sigma_3') \quad (15)$$

to describe the number of bonds, S , in a unit cross section of soil. In equation (14), a and b are constants. The constant b defines the relationship between the change in the number of bonds and the change in applied mean normal effective stress. Campanella suggested that the constant a be thought of as the number of interparticle bonds existing at zero externally applied mean normal effective stress. This represents a shearing resistance under conditions of zero applied effective stress, and thus represents a true cohesion. Neither a nor b are truly constant but are instead functions of stress history, structure and possibly other factors.

Singh (98) and Mitchell et al. (72) presented techniques for calculation of the number of interparticle bonds, S , and the experimental activation energy, E , defined as,

$$E = \Delta F - \frac{\sigma_d \lambda}{4S} \quad (16)$$

To determine the number of interparticle bonds it was assumed that X , λ , ΔF and S were constant for a set of two triaxial creep tests evaluated at the same consolidation pressure, temperature, and time, but at different deviator stress levels. An assumption also had to be made as to the value of λ , the separation distance between adjacent equilibrium points. Several authors (6, 62, 72) have presented arguments supporting separation distances of 2.8 \AA . This distance corresponds to the center-to-center spacing of oxygen atoms that comprise the plane of atoms at the surface of the clay-forming silicate minerals, and it implies that deformation involves the displacement of oxygen atoms along contacting particle surfaces.

Through the use of creep tests at two different deviator stresses, relationships such as those shown in Figure 13 and Figure 14 can be developed. Figure 13 illustrates the relationship between number of bonds and consolidation pressure for normally consolidated San Francisco Bay Mud, while Figure 14 illustrates the relationship between number of bonds and water content for a normally consolidated illite.

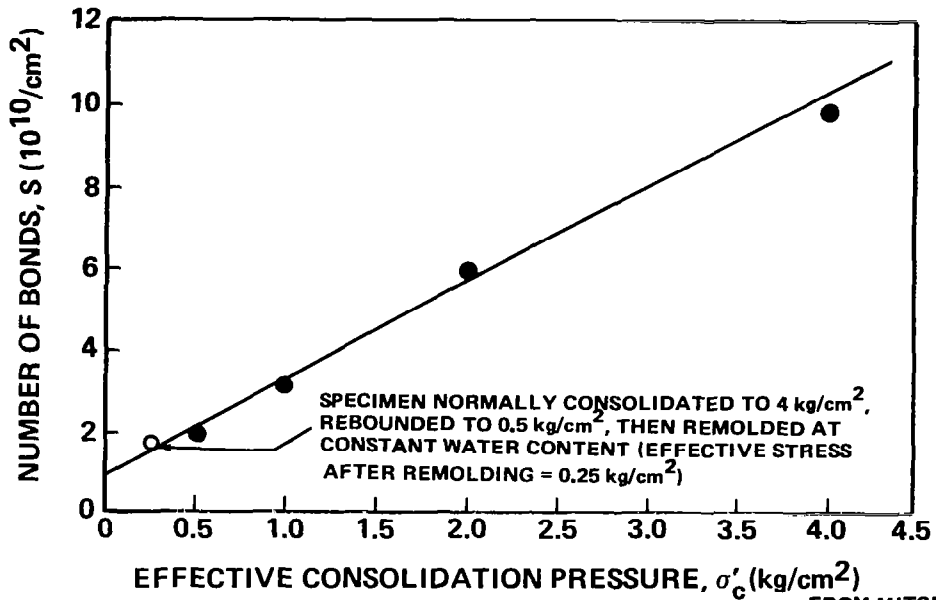


Figure 13. Number of interparticle bonds as a function of consolidation pressure for normally consolidated San Francisco Bay Mud.

FROM MITCHELL (71)

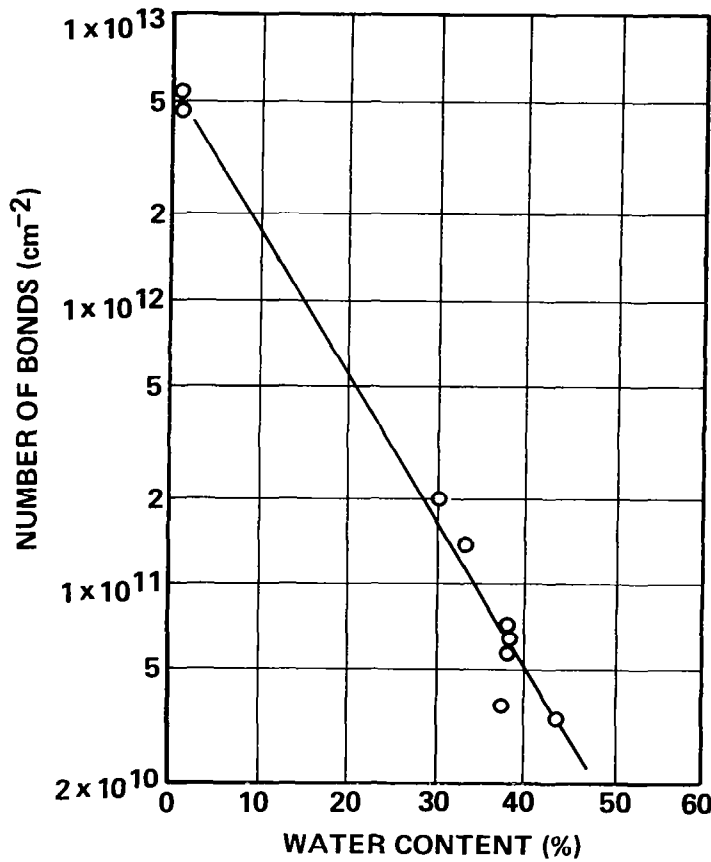


Figure 14. Number of bonds as a function of water content of illite.

FROM MITCHELL (71)

The experimental activation energy, E , can be determined from a triaxial creep test in which the temperature is rapidly changed from some initial value, T_1 , to some final value, T_2 , while the structure remains constant. Procedures for the determination of E have been outlined in the literature (72). Mitchell (71) presented activation energies for a variety of soils and other engineering materials, Table 2. He also drew four significant observations from Table 2:

a. The activation energies for soils are relatively high, much higher than for the viscous flow of water.

b. Variations in water content, ionic form, consolidation pressure, void ratio and pore fluid have no significant effect on the activation energies calculated for soils.

c. The activation energies for sand and clay are similar.

d. Clays in suspension with insufficient solids to form a continuous structure deform with an activation energy equal to that of water.

Mitchell (71) summarized the results of his and his co-workers (16, 69, 72, 73, 98) research into rate process theory with a list of 10 significant findings relating to bond numbers and activation energies, as listed below.

1. The high values of activation energy (30 to 45 kcal/mole) in soils in comparison with other materials suggest breaking of strong bonds during shear.

2. Similar creep behavior for wet and dry clay and for dry sand indicates deformation is not controlled by viscous flow of water.

3. Comparable values of activation energy for wet and dry soil indicate that water is not responsible for bonding.

4. Comparable values of activation energy for clay and sand support the concept that interparticle bond strengths are the same for both types of material. This is supported also by the uniqueness of the strength vs. number of bonds relationship for all soils.

5. The activation energy and presumably, therefore, the bonding type are independent of consolidation pressure, void ratio, and water content.

6. The number of bonds is directly proportional to effective consolidation pressure for normally consolidated clays.

7. Overconsolidation leads to more bonds than for a normally consolidated clay at the same effective consolidation pressure.

TABLE 2. ACTIVATION ENERGIES FOR CREEP OF SEVERAL MATERIALS

Material	Activation Energy (kcal/mole) ^a	Reference
(1) Remolded illite, saturated, water contents of 30 to 43%	25 to 40	Mitchell, Singh, and Campanella (1969)
(2) Dried illite: samples air-dried for saturation, then evacuated	37	Mitchell, Singh, and Campanella (1969)
(3) San Francisco Bay Mud, undisturbed	25 to 32	Mitchell, Singh, and Campanella (1969)
(4) Dry Sacramento River sand	~25	Mitchell, Singh, and Campanella (1969)
(5) Water	4 to 5	Glasstone, Laidler, and Eyring (1941)
(6) Plastics	7 to 14	Ree and Eyring (1958)
(7) Montmorillonite-water paste, dilute	20 to 26	Ripple and Day (1966)
(8) Soil asphalt	27	Abdel-Hady and Herrin (1966)
(9) Lake clay, undisturbed and remolded	23 to 27	Christensen and Wu (1964)
(10) Osaka clay, overconsolidated	29 to 32	Murayama and Shibata (1961)
(11) Concrete	54	Polivka and Best (1960)
(12) Metals	50+	Finnie and Heller (1959)
(13) Frozen Soils	94	Andersland and Akili (1967)
(14) Sault Ste. Marie clay, suspensions, discontinuous structures	Same as water	Andersland and Douglas (1970)
(15) Sault Ste. Marie clay, Li ⁺ , Na ⁺ , K ⁺ forms, in H ₂ O and CCl ₄ , consolidated	28	Andersland and Douglas (1970)

^aThe first four values are experimental activation energies, E. Whether the remainder are values of ΔF or E is not always clear in the references cited.

FROM MITCHELL (71)

8. Strength depends only on the number of bonds.
9. Remolding causes a decrease in the effective consolidation pressure which means also a decrease in the number of bonds.
10. There are about 100 times as many bonds in dry clay as in wet clay.

Christensen and Wu (17), Andersland and Douglas (6), and Matsui and Ito (60) came to essentially the same conclusion as Mitchell regarding the nature of interparticle bonding. Christensen and Wu found the activation energies of clay soils to vary between 23 and 27 kcal/mole. They noted that the agreement between rate process theory and the behavior of dry clay suggests "that the adsorbed water is not the primary source of visco-elastic behavior."

Andersland and Douglas found activation energies equal to about 28 kcal per mole and noted that this value did not change with either the degree of consolidation or the nature of the adsorption complex. They concluded that the bonding mechanism is not related to the adsorbed water layer but instead must consist of ionic bonding at points of mineral-to-mineral contact. Matsui and Ito also concluded that the adsorbed water layer did not take part in interparticle bonding since the activation energies they calculated did not depend on the existence of an adsorbed water layer nor its thickness or characteristics.

Not everyone agrees, however, with the interpretation that interparticle contact is essentially mineral to mineral. It has already been pointed out that Rosenqvist (93) visualized bonding as occurring through adsorbed water layers. Bjerrum (11) suggested that at least some of the bonding between particles is through adsorbed water. Like Rosenqvist, Bjerrum maintained that it was bonding through the adsorbed water layers that was responsible for cohesion.

Foster (24) concluded that in the stress range of 10 to 40 percent of the failure load interparticle bonds derive their strength from the adsorbed water layer. At stress levels much in excess of 40 percent of the failure stress, Foster inferred that the seat of bonding passes from the adsorption phase to the solid mineral. Pusch (82) too argued for bonding through an adsorbed water layer 20 to 30Å thick. Shearing resistance is generated through hydrogen bonding in Pusch's view. He suggests that a model for soil deformation in which bonding is through adsorbed water layers makes the assumptions used in the basic rate process theory much more realistic.

This question of the nature of interparticle bonds is an important one if an improved understanding of the fundamentals of soil behavior is going to be obtained. As the previous discussion has suggested, there is at present no complete consensus concerning the nature of these bonds. The most recent evidence, presented by Matsui, Ito and Abe (61), seems to confirm direct mineral-to-mineral contact under at least some conditions. Whether the mineral-to-mineral contact found by Matsui et al. is the exception, or instead the rule, is unfortunately not entirely clear.

4. Adhesion Theory of Friction. Most modern studies of friction use as a starting point the adhesion theory of friction based for the most part on the work of Bowden and Tabor (14), although early work by Terzaghi contributed to the development of the theory as applied to soils.

An understanding of the mechanism of friction between solids begins with the realization that the real area of contact between two contacting surfaces is only a small fraction of the gross cross-sectional area of the contact. As Bowden and Tabor illustratively stated, "even the smoothest surfaces are rough on an atomic scale and placing them together is rather like turning Switzerland upside down and putting it on top of Austria." Electrolytically polished aluminum surfaces, which by almost any standard must be considered smooth, have surface asperities between 100 and 1,000Å in height (14). Mica flakes, which are among the smoothest surfaces obtainable, may contain steps ranging between 10 and 1,000Å because of their cleavage properties and the unit layer thickness of 10Å. Thus, contacts between particles are through asperities which represent only a small fraction of the gross cross-sectional area.

Since contact takes place only at asperities, the intergranular pressures at these points are extremely high. Bowden and Tabor assert that over the region where intimate contact occurs, strong adhesion takes place and the specimen becomes, in effect, a continuous solid. Evidence for these adhesion bonds is readily available in metals (13, 14). As previously discussed, the situation in soils is not as certain. The activation energies measured in evaluating the application of rate process theory to soils support the concept of adhesion at contacts since the measured energies are in the range of primary valence bonding. So do the track marks in the sheared kaolinite particles studied by Matsui et al. (61). Lee (50) provided indisputable evidence for the adhesion bonding of Sacramento River Sand when consolidated above pressures of 60 kg/cm².

In the adhesion theory of friction, sliding between two surfaces will occur when the tangential force acting between surfaces exceeds the collective shear strength of all of the asperities. The shear strength of a single asperity is dependent on the contact area at the asperity and the shear strength per unit area of the material comprising the junction. The contact area is dependent on whether the asperity deforms plastically or elastically under normal load, on the presence or absence of surface contaminants and on whether or not there is any amount of gross seizure.

For the case of a clean, perfectly plastic material, the contact area (A_c) at adjoining asperities is equal to the normal force being transmitted at the contact (N) divided by the yield strength of the material comprising the junction. If the shear strength of the junction material is represented by τ_m , then the following equations describe the behavior of the contact,

$$A_c = N/\sigma_y \quad (17)$$

$$T = A_c \tau_m \quad (18)$$

where σ_y is the yield strength of the material and T is the shear strength of the junction. Equations (17) and (18) explain two basic laws of friction, known as Amonton's Laws first proposed in 1600. They are that the frictional force is proportional to the normal load and that the frictional force is independent of the total apparent area of contact.

The analysis of friction at plastic junctions is complicated by at least two factors. The first stems from the observation that the critical shear stress of a metal is typically about one-fifth of its yield stress. Based on equations (17) and (18), a coefficient of about .2 would be expected. In reality, most unlubricated metals give a value for their coefficients of friction of about 1 (14). The problem lies in the fact that equations (17) and (18) treat the yield stress and shear strength of a material as being independent properties. However, plasticity theory dictates that under combined compressive and tangential stress conditions deformation occurs according to a plastic yield criterion. Examples of plastic yield criteria include the Mohr-Coulomb criterion, the von Mises criterion and the Tresca criterion. Bowden and Tabor suggest that a typical yield criterion for the plastic deformation of metals might be of the form,

$$p^2 + \alpha\tau^2 = \sigma_y^2 \quad (19)$$

where p is the normal stress, τ is the shear stress and α is a constant. In the absence of any shear stress the normal stress at a yielding asperity is equal to the initial yield stress $p = \sigma_y$. When a small tangential stress, τ , is applied, further plastic yielding occurs. Although the stress increment is tangent to the contact, plasticity theory dictates that the direction of the strain increment be in the direction of the stress resultant (14). For only a small increase in shear stress, the initial plastic displacement tends to push the two contacting asperities closer together, which leads to an increase in the area of contact. As the shear stress is increased, the plastic strain increment rotates from an initial direction perpendicular to the contact to a final direction which may be parallel to the contact.

The vertical component of any plastic yielding leads to an increase in the area of contact and thus an increase in the frictional strength of the contact. This process is called junction growth and is responsible for cold welding in some materials. If the shear strength of the junctions equals that of the bulk solid, gross seizure occurs. Mitchell (71) notes that if the ratio of junction strength to bulk material strength is less than .9, as it probably is for most soils, the amount of junction growth will be small.

The second factor that complicates the analysis of plastic junctions (and elastic junctions) is surface contamination. Due to the presence of adsorbed contaminants, actual solid-to-solid contact may develop over only

a fraction of the contact area, as shown in Figure 15. If the fraction of the contact that is solid to solid is denoted by δ , and the contaminant film shear strength is τ_c , the strength of the contact will be given by the equation,

$$T = A_c [\delta \tau_m + (1 - \delta) \tau_c] \quad . \quad (20)$$

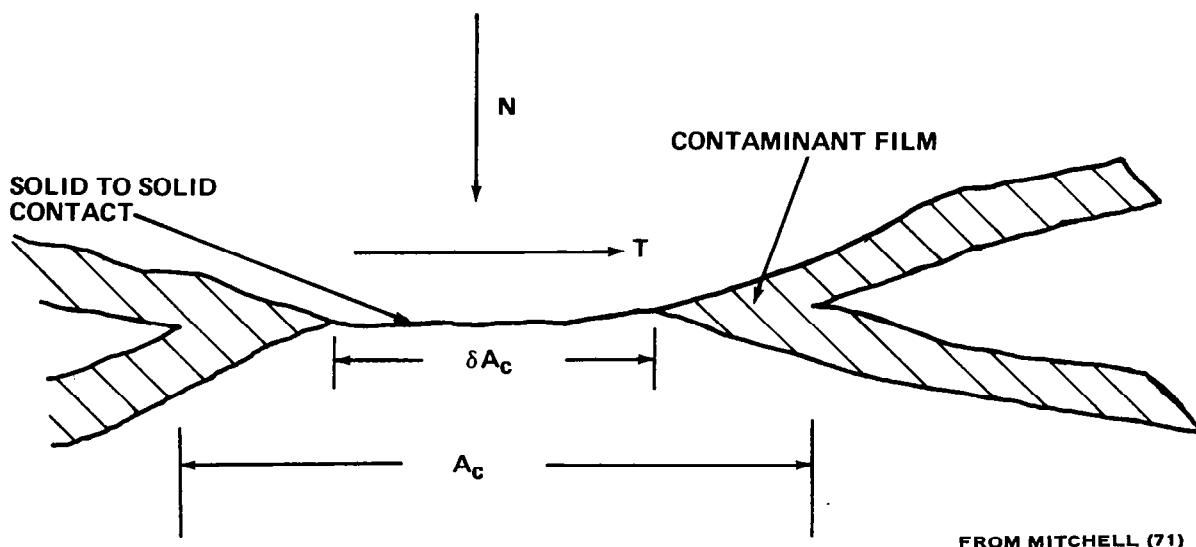


Figure 15. Plastic junction between asperities with adsorbed surface films.

Mitchell (71) notes that equation (20) may provide a basis for explaining the fact that the measured friction angles for bulky minerals, such as quartz and feldspar, are greater than those measured for platy minerals, such as clay. He notes that the "small particle size of clays means that the load per particle, for a given effective stress, will be small relative to that in silts and sands composed of bulky minerals. The surfaces of platy silt and sand-size particles are smoother than those of bulky mineral particles. The asperities, caused by waviness of the surface, are more regular but not as high as those for the bulky minerals.

"Thus it can be postulated that for a given number of contacts per particle the load per asperity decreases with decreasing particle size and, for particles of the same size, is less for platy minerals than for bulky minerals. Because δ should increase as the normal load per asperity increases, and it seems reasonable that the adsorbed film strength is less than the strength of the solid material ($\tau_c < \tau_m$), it follows that (ϕ_u) for small and platy particles is less than (ϕ_u) for large and bulky particles."

Not all contacting asperities deform plastically. Depending on the material and stress range, none, some or all contacts may behave elastically. If the asperities are assumed to have spherical tips, the Hertz analysis may be used to calculate the area of contact, A_c , which is given by,

$$A_c = \frac{\pi}{4} (\delta RN)^{2/3} \quad (21)$$

$$\delta = \frac{12(1 - \nu^2)}{E} \quad (22)$$

where R is the radius of the asperity, N is the normal force, ν is Poisson's ratio and E is the modulus of elasticity. The shear strength of the contact, T , will be equal to,

$$T = \tau A_c = \tau \frac{\pi}{4} (\delta NR)^{2/3} \quad , \quad (23)$$

while the coefficient of friction will be given by,

$$u = T/N = \tau (\delta R)^{2/3} N^{-1/3} \quad . \quad (24)$$

This equation predicts the coefficient of friction will vary as the minus one-third power of the normal load. Mitchell (71) shows that under a certain set of assumptions this is the same as predicting that the coefficient of friction will vary as the minus one-third power of the effective stress.

Archard (7) notes that the preceding analysis may not be entirely accurate for conditions in which there are a large number of contacting asperities between surfaces. He indicates that for a set of asperities deforming elastically there are two different mechanisms that might occur.

1. As the load increases, the number of contacting asperities remains constant and the elastic deformation of each asperity increases. In this case the coefficient of friction is proportional to $N^{-1/3}$.

2. As the load increases, the number of contacting asperities increases proportionally and the deformation of each asperity remains essentially constant. In this case the coefficient of friction is independent of N .

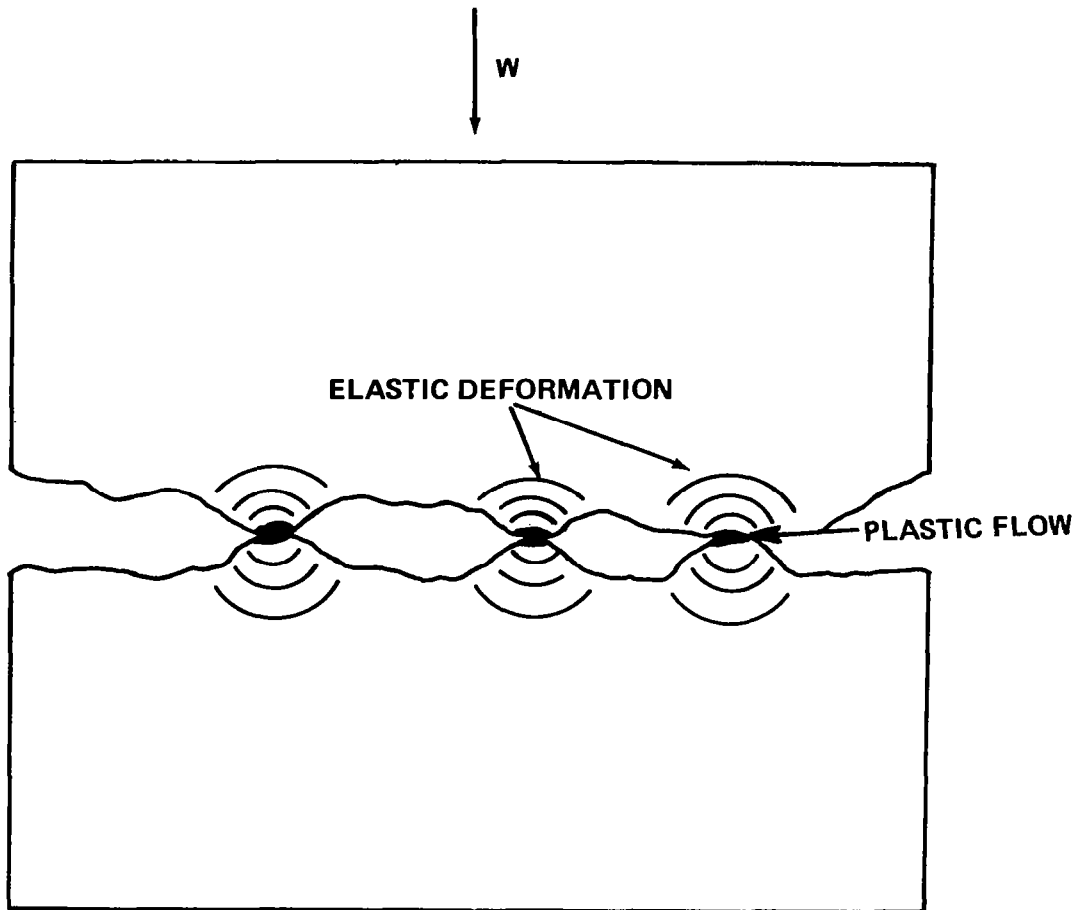
The important point to note is that Archard's analysis demonstrates that elastic materials may deform with constant coefficients of friction. He also points out that in most actual situations the behavior of the coefficient of friction will lie somewhere between the two extremes cited above. Mitchell (71) points out that experimental data exist which both support the minus one-third power dependence of the coefficient of friction on the normal load and contradict it. This seems reasonable in light of the preceding discussion.

One other factor complicates this discussion. In reality, in most soils, there is probably a mixture of plastic and elastic contacts. For instance, Bromwell (15) suggests that contacts between pure quartz particles are both elastic and plastic, with the percentage of plastic contacts increasing with increasing confining pressure.

A criticism that is sometimes brought against the adhesion theory of friction is that if friction is caused by interparticle adhesion, why is it that when two surfaces are pushed together they usually do not exhibit a strong permanent adhesion after the stress is released. Bowden and Tabor (14) answer this criticism by noting that in order to measure an adhesion between materials the normal load must first be removed. As the normal load is released, elastic stresses in the vicinity of the junctions will be released, causing a slight local straining and a change in shape of the interfacial contour. This will be true regardless of whether the junction is elastic or plastic. The junctions, being fairly brittle from the amount of working they have undergone, will break at very small levels of tensile strain. This means that as the normal load is removed, the junctions are pulled apart one by one so that practically no junctions are left when the adhesion measurements are made. This mechanism is illustrated in Figure 16.

The adhesion theory of friction is generally considered as being valid for describing the component of friction resisting the sliding of one macroscopically smooth surface over another (108). However, it is important to point out that in many systems there may exist other components to the total frictional resistance, including the elastic deformation of mutually interfering surface protrusions, brittle fracture and/or ploughing of surface protrusions, dilatancy, particle rearrangement and particle rolling.

The two theories that have just been presented, the theory of rate processes and the adhesion theory of friction, are seen as being useful in understanding the fundamental behavior of soils. These theories will be used in subsequent section of this report to develop a hypothesis for fine-grained, cohesive soil behavior.



FROM BOWDEN AND TABOR (14)

Figure 16. Sketch showing plastic deformation at the points of real contact. At these regions junctions are formed. The surrounding regions are deformed elastically so that when the load is removed these elastic stresses are released, and the junctions are broken.

5. Bjerrum's Model. Bjerrum (11) presented a mechanistic picture of shearing resistance in clays in which the differences between cohesive resistance and frictional resistance were explained in terms of the type of interparticle contact with which each component of strength was associated. Bjerrum maintained that there were two different types of contacts in cohesive soils, these being:

a. "At the contact points where the transmitted stresses are relatively large, the film of adsorbed water surrounding the particles has been squeezed away and a mineral to mineral contact area is established."

b. "At the contact points which carry a relatively small load, the stresses in the contact area are not large enough to displace the film of adsorbed water and there will be no mineral to mineral contact."

At the mineral to mineral contacts, resistance to shear displacements is developed through friction. The frictional resistance is attributed to the shear strength developed by "atomic bonding" in the area of contact. The number of bonds is thought to be proportional to the normal stress being transmitted through the contact. Deformation of the contacts due to normal loads is assumed to be elastic. As a consequence, if the stress transmitted at the contact is increased, the area of contact is increased, as is the number of bonds and hence the shear strength. On the other hand, if the stress being transmitted at a contact is reduced, the shearing resistance that can be developed at that contact will also be reduced, since the contact will rebound elastically. Such contacts show no cohesive or tensile strength.

Where the contact stresses are relatively small, the contacts are not mineral to mineral, but instead consist of merged films of "semi-rigid water." The area of these merged water films is thought to be proportional to the normal stress being transmitted through the contact. The maximum shear force which can be applied to such a contact is proportional to the area of contact and thus to the normal pressure transmitted through the contact points. The shearing resistance is assumed to develop through the adsorbed water.

The semi-rigid water contacts are thought to be plastic in nature. Therefore, if the normal force being transmitted through one of these contacts is reduced, the contact area will remain unchanged, as will the resistance to sliding. Bjerrum labels this component of strength the effective cohesion and notes that this cohesive contribution to strength increases with increased consolidation pressure, which is in agreement with the findings of Hvorslev (29). He also notes that the proposed shear strength model is in agreement with Hvorslev's failure criterion for remolded clays, which is given by the equation,

$$S = X\sigma_c' + \sigma' \tan\phi_e \quad (25)$$

where S is the shearing resistance, σ' is the effective normal stress acting on the shear plane, σ_c' is an equivalent consolidation pressure⁴, ϕ' is the angle of internal friction, and X is a constant.

Bjerrum attributed the time dependency exhibited by most clays to the behavior of the cohesive contact points. The time-dependent deformations were thought to be caused by the viscous flow of the adsorbed, structured water layer. This viscous flow is associated with the thermal vibrations of adsorbed water atoms. The rate of viscous flow, or creep,

⁴The equivalent consolidation pressure is defined as the consolidation pressure a normally consolidated clay would have to be subjected to in order to reach a water content equal to that of the clay being considered.

will depend on the shear stress being applied and on the thickness of the zone of semi-rigid water. The thickness of the semi-rigid water layer is reflected macroscopically through the plasticity of the soil. For a given soil, each shear stress level is associated with a certain rate of shear straining of the water layer. This straining would occur at a constant rate if it weren't for the fact that as the strains accumulate the shear stresses are transmitted to the relatively immobile frictional contacts. If the shear load given up by the cohesive contacts exceeds the capacity of the frictional contacts, creep rupture will occur. If, instead, the shear load given up by the cohesive contacts is smaller than the available friction, the creep deformation will eventually come to a halt.

Ramanatha Iyer (90) formulated a mechanism for the behavior of Drammen plastic clay, based largely on Bjerrum's ideas. Ramanatha Iyer agreed with Bjerrum's two contact concepts. He attributed frictional resistance to sliding between grains in mineral-to-mineral contact at asperities. Cohesion was attributed to the rupture of contacts at the adsorbed water phase. He agreed with Bjerrum that in clay deposits that have been subjected to stress for a long period of time, the effective stresses are gradually transferred from the cohesive contacts to the frictional contacts. His model departs from Bjerrum's, however, when he notes that with time the cohesive contacts may become rigid and lose their viscous nature. The adsorbed water at these contacts is thought to "approximate the behavior of ice," and the increased shearing resistance that these contacts develop with time is manifested through the preconsolidation effect found in many soils (9).

Ramanatha Iyer supports the work of Schmertmann and Osterberg (96) which indicates that upon load application cohesion is mobilized at much lower strains than is friction. Thus on application of a shear stress the cohesive contacts are stressed first. With time these contacts harden. If the applied shear stress is increased after some period of time, the soil exhibits stiff stress-strain properties until the strength of the cohesive bonds is exceeded, at which point frictional resistance is mobilized, but only after relatively large strains. The value of shear stress at which this structural breakdown has been termed the critical shear stress, and behavior of this type has been observed in a fairly large number of structurally sensitive normally consolidated clays (9).

Mitchell (71) pointed out that although Bjerrum's hypothesis of mineral-to-mineral adhesion at frictional contacts and merged water films at cohesive contacts appears to account for a number of observations, it does not agree with several of the findings of the rate process theory studies, particularly the finding that the activation energies measured in dry soils are similar to those measured in wet soils and that they are high enough to support the breaking of strong bonds during shear. Mitchell's results imply that wet soils have the same type of contacts as dry soils and these contacts are solid to solid in character.

Pusch (83) notes that although some of the shear tests run on clays at low effective stress levels (for example, see Larsson, 1977) do not show a cohesion intercept on a Mohr-Coulomb strength plot, this does not mean that cohesive forces do not exist within these soils. As evidence of cohesive forces in illitic clays, Pusch notes that clay-water pastes may show considerable yield stresses. He also points to thixotropic strength regain as further evidence. In two Swedish quick clays that he studied, Pusch found that freshly remolded specimens of the soil contained a large number of dispersed, very small particles. Within 30 days after remolding, a large number of particle aggregates had formed. Within 120 days after remolding, the undrained shear strength, as measured by the Swedish fall cone test, had increased from zero to $.042 \text{ kg/cm}^2$. This strength increase was attributed to the formation of "a continuous network of aggregates linked together by groups of small particles." These time-dependent changes require the presence of a cohesive force between soil particles.

In the presence of these cohesive forces, a sizeable cohesion intercept might be expected on a Mohr-Coulomb strength plot. According to Pusch, however, this is not necessarily the case. In order to understand the absence of a cohesion intercept the microstructural changes during shear must be considered. Figure 17 shows the microstructure of a marine illitic clay. The drawing in the left half of the figure illustrates the fabric of the undisturbed clay, while in the right half the fabric after the soil has been subjected to a small amount of shear straining is shown.

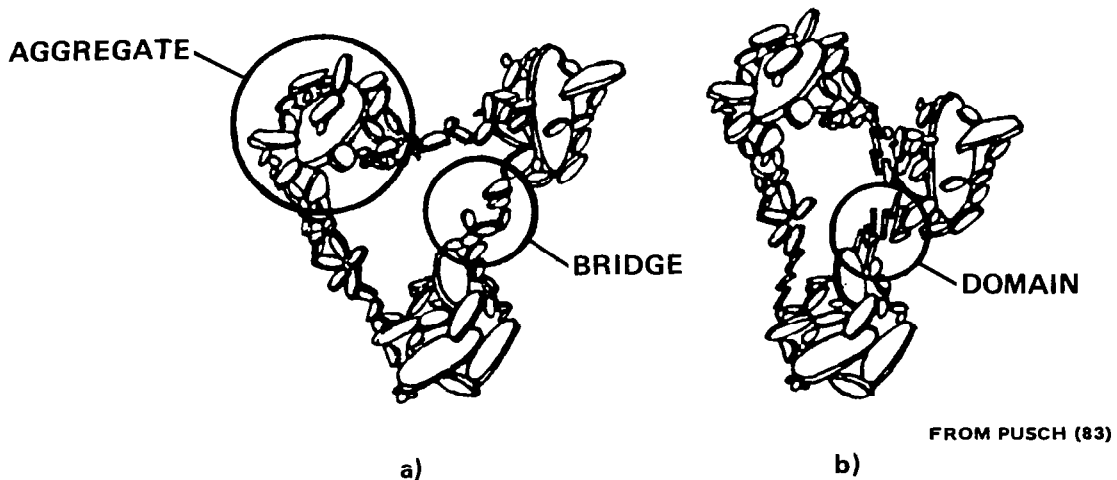


Figure 17. Illustration of initial fabric of an illitic marine clay, (a) in the undisturbed state, and (b) during shear. Note the formation of domains at the weak particle links in (b).

The undisturbed fabric is characterized by relatively strong, relatively dense aggregates, which are connected to each other by bridges or links of small particles. The links are thought to be the weakest points within the structure. When a deviator stress is applied to a soil with a fabric like that shown in Figure 17, some of the links break down. The particles within these links reorient themselves to form local groups of parallel particles called domains.

Pusch hypothesizes that the face-to-face orientation of particles within the domains causes large repulsive factors between overlapping diffuse double layers and increased particle spacing associated with the flow of water into the region between particles due to the large osmotic gradient. This results in a drop in bulk shear strength to a very low value, especially at low effective stresses where the contribution to shear strength made by dilatancy and friction are very low. Thus, although considerable cohesive forces are active within the soil, the structural breakdown which takes place during shear eliminates these forces, temporarily, at the weakest links in the soil structure, resulting in a very small or nonexistent cohesion intercept on a Mohr-Coulomb strength plot.

Experimental Studies of True Cohesion and True Tensile Strength in Non-Cemented, Fine-Grained, Cohesive Soils

There have been very few studies in which attempts have been made to measure the true cohesive strength and true tensile strength of fine-grained, cohesive soils. Of the studies that have been carried out, most have been concerned with the behavior of cemented sensitive soils. Measurements of true cohesion in soils in which cementation effects are only of minor significance are extremely rare. One of the most comprehensive studies of cohesive soil behavior at very low effective stress levels is the one performed by Ladd and Kinner (45).

Ladd and Kinner hypothesize that cohesion will exist between two clay particles in the absence of applied effective stresses if there exists a net internal attractive force between the particles; i.e., if at any given interparticle spacing, the sum of the short-and long-range attractive forces exceeds the sum of the short-and long-range repulsive forces. The net energy of attraction varies as a function of interparticle spacing, as hypothesized by Figure 18, and as a function of many of the physico-chemical variables discussed earlier.

Figure 18 illustrates several features of Ladd and Kinner's hypothesis for interparticle cohesion. For instance, it can be seen that each of the different interparticle forces has a range of interparticle distances over which it may contribute significantly to the force balance. Figure 18 also illustrates the concept of potential wells (points 1 and 3 in the figure), which represent states of stable equilibrium in which there is a net attractive force between the two particles. Based on Figure 18 Ladd and Kinner contend that there is no one true cohesion. Instead, they view true cohesion as being a complex function of particle spacing and system chemistry.

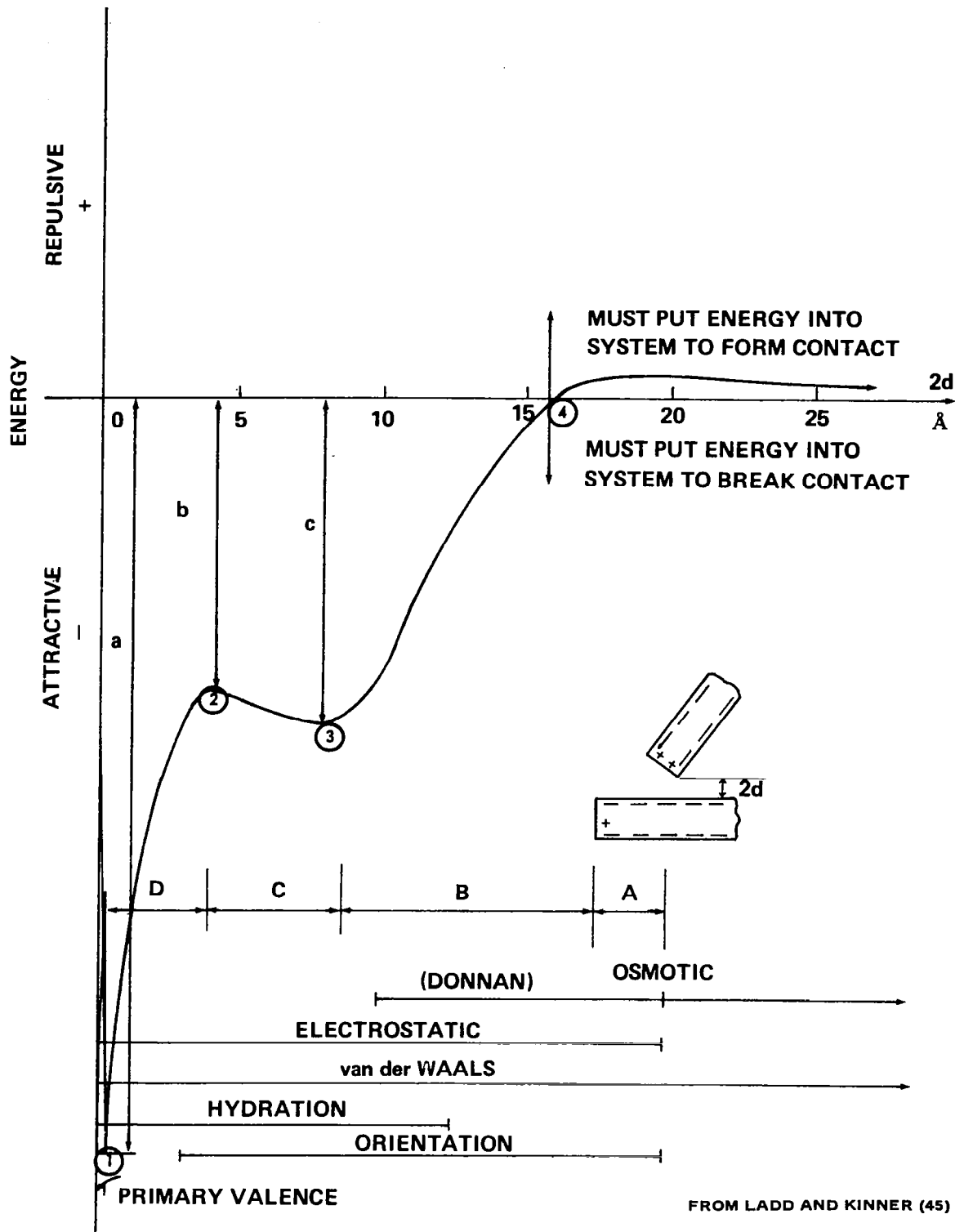


Figure 18. Hypothetical relationship of energy versus interparticle spacing.

If the hypothesis for cohesion presented by Ladd and Kinner is correct, it should be expected that the measured cohesion varies in a predictable way when measured under different environmental and compositional conditions. Among the factors which should affect the measured cohesion are pore fluid salt concentration, valency of the double-layer ions, pore fluid dielectric constant and pH.

One way in which the cohesion of a clay could be affected would be by increasing or decreasing the interparticle repulsive stresses. For instance the long- and short-range osmotic repulsions between particles may be increased by decreasing the pore fluid electrolyte concentration, by replacing the double-layer cations with other cations of lower valency and by increasing the dielectric constant of the pore fluid. As these factors are varied and the repulsive forces between particles increased, a decrease in the measured cohesion would be expected.

Similarly, in a soil such as kaolinite, in which the edge effects play an important role, it would be expected that the cohesion would be sensitive to the pore fluid pH as well as to the addition of dispersants and salts. The cohesive strength of kaolinite should decrease with an increase in pore fluid pH, since an increase in pH leads to a decrease or elimination of the positive edge charge found on kaolinite particles in low pH environments. The lower positive edge charge leads to smaller edge-to-face electrostatic attractions between the kaolinite particles, which should, in turn, reduce the cohesion. If a dispersant or a salt is added to a kaolinite that has positive edge charges, the dispersant's negatively charged radicals and the salt's anions would be expected to migrate to the particle edges, reducing their net positive charge and, consequently, reducing the electrostatic attraction between particles.

In order to test this predicted behavior and help verify the validity of their hypothesis Ladd and Kinner performed a series of drained unconfined compression tests on a variety of different soils. Unfortunately, with this type of test the actual cohesion cannot be measured because there will always be a small effective stress on the failure plane, as shown in Figure 19. However, as this figure also illustrates, this effective stress is small. If it is assumed that the Mohr-Coulomb failure envelope is a straight line right up to the shear stress ordinate and the angle of internal friction is of the order of 30° , the maximum difference between the true cohesion intercept and the strength measured in the unconfined compression test is less than 40 percent.

The soils used in Ladd and Kinner's study were Boston Blue Clay, Peerless Kaolinite and Vicksburg Buckshot Clay. Index properties for these three soils are given in Table 3. Mineralogical data for the soils are presented in Table 4.

Samples of the various soils were prepared by mixing together 8 to 10 kg of dry fractionated soil with water to form a slurry. The slurry was then placed in large consolidometers by a special technique which ensured saturation, and then consolidated one dimensionally to a vertical

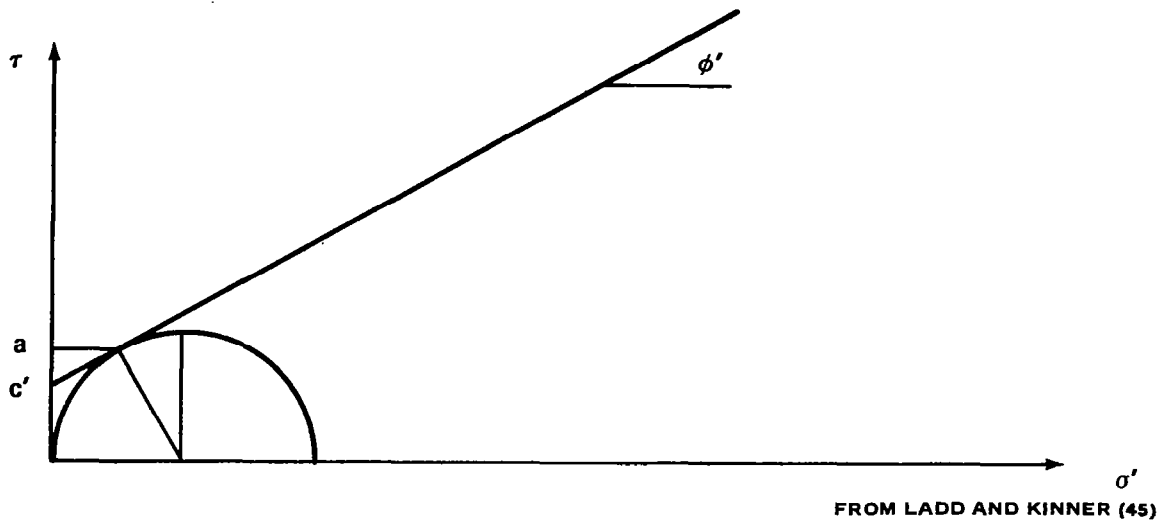


Figure 19. Mohr's circle for a drained unconfined compression test. c' represents the true cohesion of the soil being tested.

TABLE 3. INDEX PROPERTIES OF SOILS

FROM LADD AND KINNER (45)

Soil Type	Batch	Liquid Limit	Plasticity Index	Activity
Boston Blue Clay	A ¹	31.5	10.5	0.29
	B ¹	42.7	18.8	0.36
Peerless Kaolinite	A, B, C ²	55.0	27.8	0.48
Vicksburg Buckshot Clay	A, B ³	63.5*	38.2*	—
	C ³	59.6*	35.8*	0.76

1. Tested with 16 g/l NaCl pore fluid
2. Tested with distilled water and HCl mixed to pH = 4.5
3. Tested with distilled water

* Atterberg's limits determined by different persons. All batches from same source.

TABLE 4. MINERALOGICAL DATA

FROM LADD AND KINNER (45)

Boston Blue Clay (from Mitchell 1956)

~15 - 20%	Quartz
~5%	Chlorite
~30 - 45%	Illite
~1.5 - 3%	Iron Oxides
<1%	Organic Matter

Peerless Kaolinite (information obtained from Dr. R. T. Martin, Massachusetts Institute of Technology)

95+%	Kaolinite
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Vicksburg Buckshot Clay (Ladd, 1961)

~40%	Quartz and Feldspar
~25%	Montmorillonite
~25%	Illite
~1%	Organic Matter
~2%	Iron Oxides

pressure of either 1.0 or 1.5 kg/cm². Samples could then be trimmed from the large consolidated blocks as needed. Each sample would then be further consolidated to its maximum isotropic consolidation pressure, rebounded and placed in a soaking solution designed to give the desired pore fluid composition. During testing the sample was kept submerged in its soaking solution. No membranes or filter paper drains were used during the test, and a loading system was devised which made piston friction negligible.

The chemical environment during these tests was controlled by soaking the samples in a variety of pore fluids. Samples of Boston Blue Clay and Vicksburg Buckshot Clay were soaked in solutions having differing salt concentrations, varying from distilled water to a one molar concentration of CaCl₂. In order to investigate the effect of particle edge charge on the cohesion of Peerless Kaolinite samples were soaked in solutions of different pH. In addition, several tests were performed on the Peerless Kaolinite in which the pore fluid salt concentration was varied or a pore fluid other than water was used. Stress history was controlled by altering the isotropic consolidation pressure at which each sample was consolidated.

The results of Ladd and Kinner's tests are presented in Figure 20 through Figure 23. The tests results for each of the three different soils tested will be discussed separately.

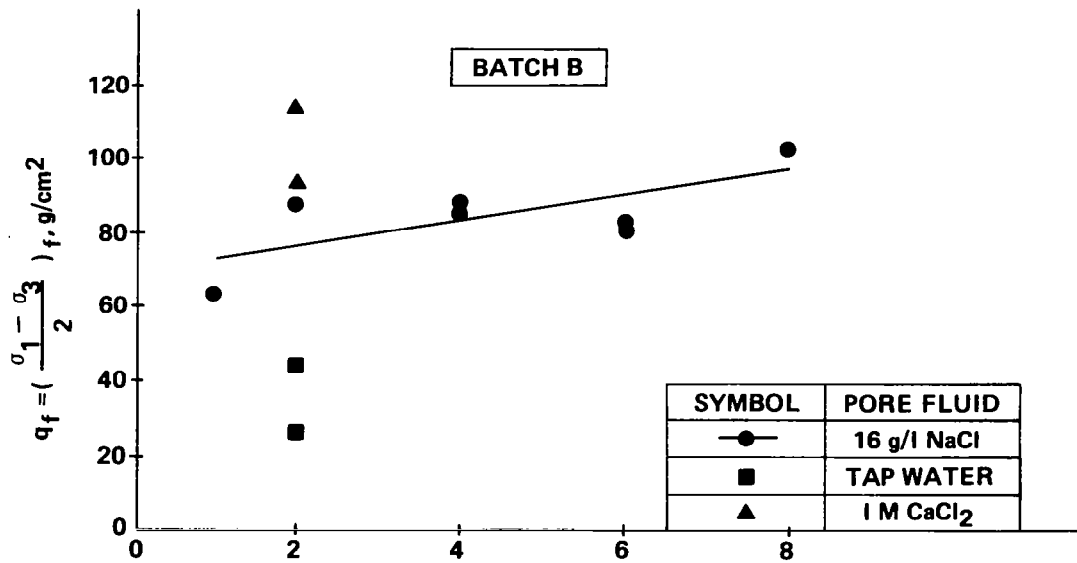
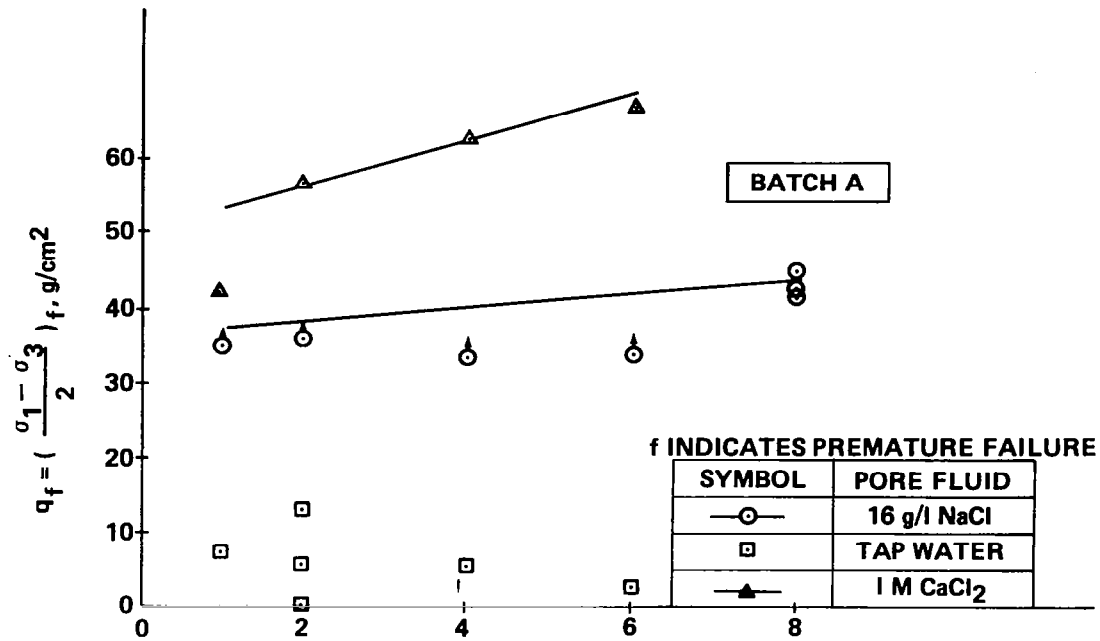
Boston Blue Clay. Figure 20 presents the results obtained from the tests on Boston Blue Clay. It can be seen that the samples soaked in 1 M CaCl_2 solution exhibited the highest strengths, while the samples soaked in tap water were the weakest. The strengths of the samples soaked in 16 g/liter NaCl (≈ 3 M) solution were intermediate. Moderate increases in strength were observed with increasing maximum past isotropic consolidation pressure (σ_{cm}) for all samples except those soaked in tap water. Not shown by Figure 20 is the fact that for the samples soaked at 16 g/liter NaCl the strain at failure increased with increasing σ_{cm} and, thus, increasing overconsolidation ratio (OCR).

The results of these tests indicate that the measured cohesion of Boston Blue Clay is sensitive to changes in the pore fluid electrolyte concentration and the valency of the double-layer cations. They also imply, therefore, that diffuse double-layer forces play a role in determining the cohesive strength of this material. This can be seen by noting that the highest strengths were measured for the samples with the most depressed double layers, i.e., the sample with the highest pore fluid electrolyte concentration and the highest valency of the cations in the double layer.

Ladd and Kinner attribute the increased strength of samples at higher maximum past consolidation pressures to closer particle spacings and/or more particle contacts. The behavior of the strain at failure in the 16 g/liter NaCl samples was seen as being anomalous. Ladd (43) notes that conventional drained triaxial compression tests on clays show that the strain at failure decreases with increasing OCR. Ladd and Kinner were not able to develop a suitable explanation for this behavior.

Vicksburg Buckshot Clay. Figure 21 presents the results obtained from the tests on the Vicksburg Buckshot Clay. All of the test results, except one, indicate a trend of increasing cohesion with decreasing extent of the double layer. The one test result that is not consistent is thought to be due to a tipping of the loading apparatus or eccentricity in the sample loading.

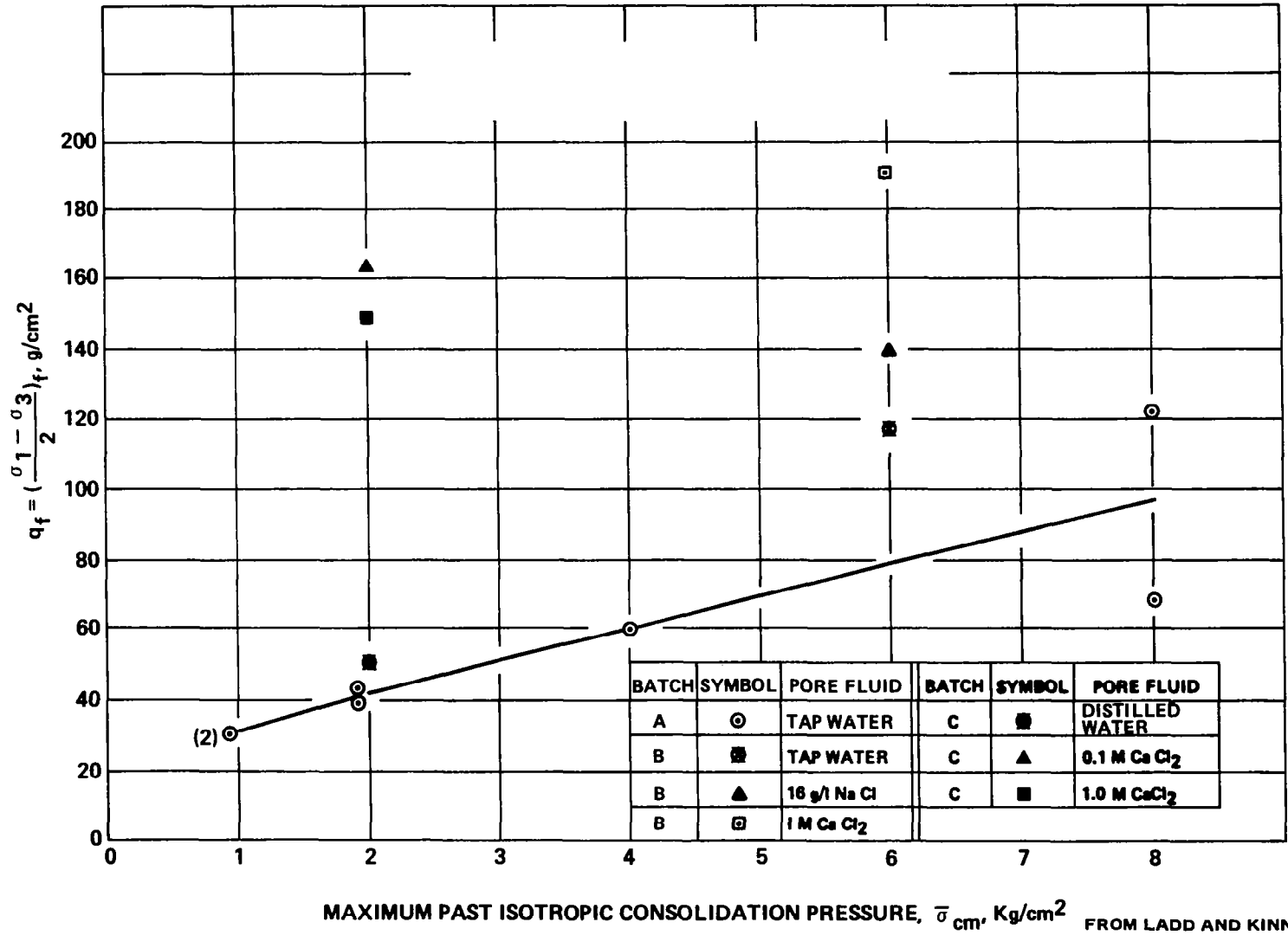
Ladd and Kinner further note that the extent of the diffuse double layer, as measured by the pore fluid electrolyte concentration, has a much greater effect on the cohesion intercept of Vicksburg Buckshot Clay than it does on the undrained strength of the same soil at higher effective stresses. The pore fluid electrolyte concentration had only a 10 to 20 percent effect on the undrained shear strength of Vicksburg Buckshot Clay at effective stress levels of the order of several kilograms per square centimeter. This compares to a 192 percent increase in sample strength registered by the 1 M CaCl_2 sample over that of the distilled water sample at a σ_{cm} of 2 kg/cm^2 . The 10 to 20 percent increase at higher stress



MAXIMUM PAST ISOTROPIC CONSOLIDATION PRESSURE, $\bar{\sigma}_{cm}, \text{ Kg/cm}^2$

FROM LADD AND KINNER (45)

Figure 20. Boston Blue Clay. Drained unconfined shear strength.



MAXIMUM PAST ISOTROPIC CONSOLIDATION PRESSURE, $\bar{\sigma}_{cm}$, Kg/cm² FROM LADD AND KINNER (45)

Figure 21. Vicksburg Buckshot Clay – Batches A-C. Drained unconfined shear strength.

levels is more in line with a number of other studies which indicate that at stress levels normally encountered in engineering practice the diffuse double-layer repulsions have only a minor influence on the measured shear strength parameters.

Peerless Kaolinite. Figures 22 and 23 present the results obtained from the tests on Peerless Kaolinite. As noted previously, Ladd and Kinner suspected that edge charge effects would play a comparatively important role in determining the cohesion of kaolinite. They expected to note a decrease in strength when the pore fluid pH was increased due to the decrease in electrostatic attractions. Studies have shown that the edge charge on a kaolinite particle tends to go from positive to neutral in a slightly alkaline environment and from neutral to negative as the pore fluid becomes highly basic.

The test results indicated that the effects of pH were not always as dramatic as anticipated or even in the predicted direction. As Figure 22 illustrates, samples soaked in a pH 7.0 pore fluid were found to be as strong as samples soaked in a pH 4.5 environment. More surprisingly, the pH 9.5 samples showed little or no drop off in strength. The results presented in Figure 23 are in somewhat better agreement with the expected trends than those found in Figure 22. Significant reductions in cohesion were experienced by the pH 9.5 samples. However, the pH 7.0 samples were still as strong as the pH 4.5 samples.

The complete insensitivity of the results shown in Figure 22 to sample pH are thought to be due to the sample preparation method. The initial pore fluid pH of the samples shown in this figure was 4.5. Ladd and Kinner hypothesize that at this pH the solubility of the kaolinite particles is relatively high. The samples were stored for some time at this pH. During this time "particle deterioration and deposition of anhydrous aluminum contaminants" on the particle edges was thought to have occurred. These contaminants may have acted as cementing agents or, instead, masked the edge charges. The net result was an insensitivity to later changes in pH. In order to avoid this problem with the samples which are shown in Figure 23 the initial batches were prepared with more neutral solutions.

Based on the test results shown in Figure 23, Ladd and Kinner drew the following conclusion with respect to the effects of pH on the cohesive strength of kaolinite:

- a. "The plus to minus edge to face electrostatic attraction, upon which the cohesion of kaolinite is highly dependent, is reasonably insensitive to environment provided the pH is neutral or moderately acidic, i.e., between pH = 4.5 and 7.0."
- b. "Under certain conditions in a basic environment, this insensitivity breaks down and the particle edges hydrolyze. Electrostatic attraction among flocculated particles is reduced and cohesion drops accordingly."

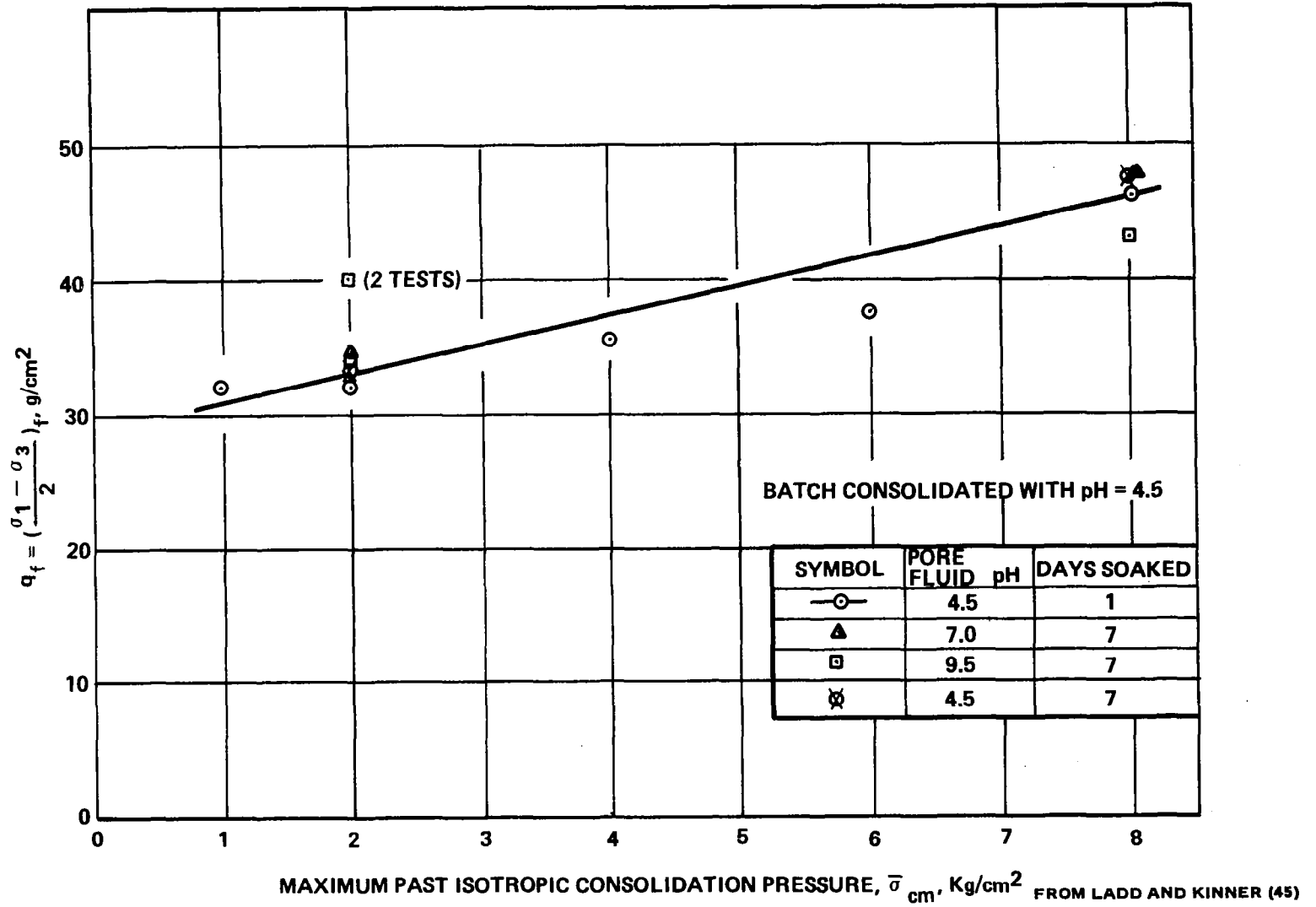


Figure 22. Peerless Kaolinite -- Batch A. Drained unconfined shear strength.

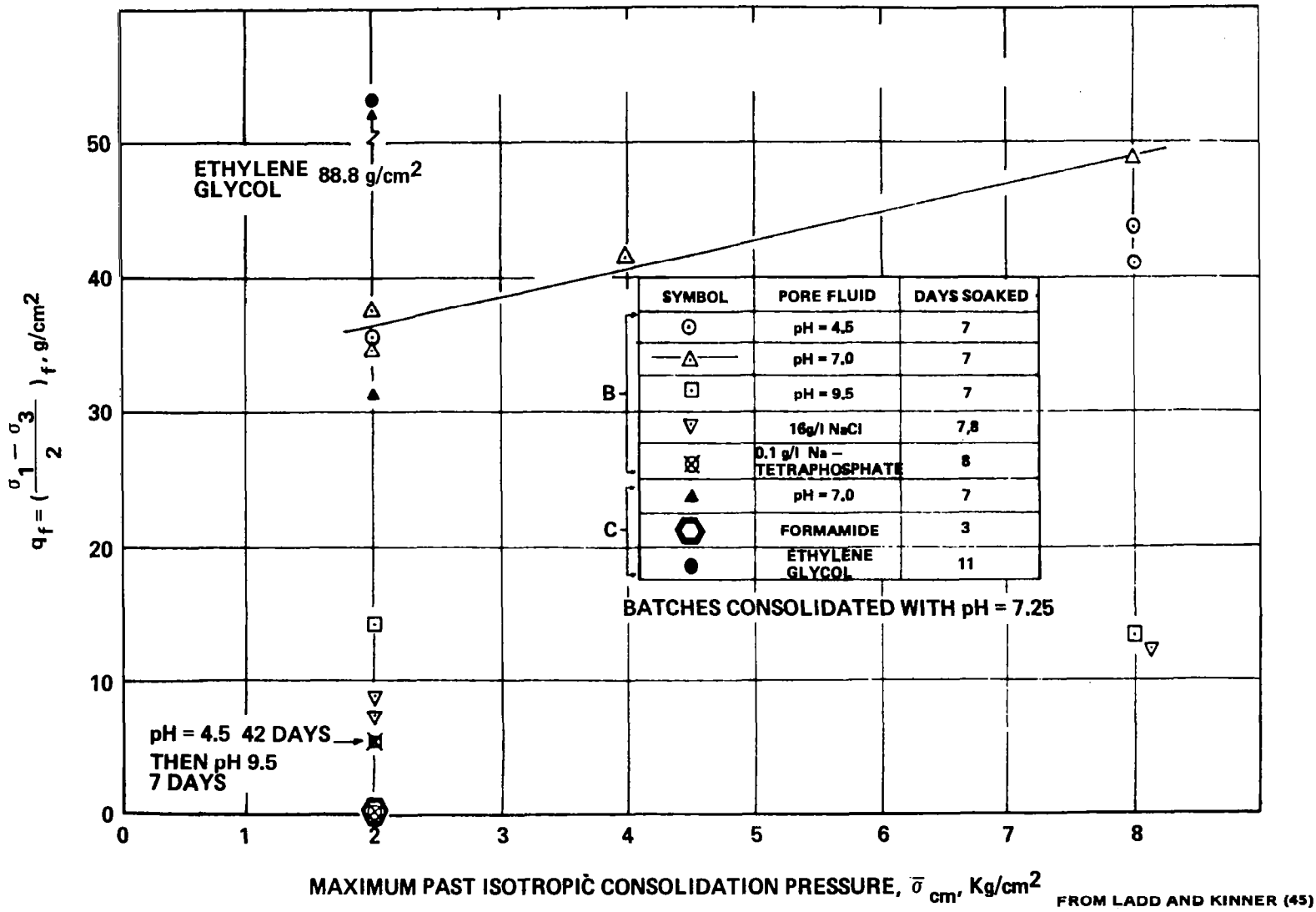


Figure 23. Peerless Kaolinite — batches B and C. Drained unconfined shear strength.

The effect of an increase in pore fluid salt concentration was to decrease the cohesion greatly. This behavior is thought to be due to the migration of some of the anions in the pore fluid to the particle edges, neutralizing any positive edge charge and thus diminishing the electrostatic attractions between particles. Additionally, the tests using NaCl in the pore fluid illustrate the relatively minor role of diffuse double-layer forces in influencing the cohesion of kaolinite. Increases in the pore fluid salt concentration decreased the extent of the diffuse double layer and thus decreased the long-range repulsive forces. If this reduction was significant, however, the loss of cohesion due to anion migration should have been offset, at least partially, by the gain in cohesion due to the depressed double layers. As Figure 23 illustrates, this clearly was not the case.

The results of Ladd and Kinner's study provide valuable insight into the nature of soil cohesion. They demonstrated clearly the dependence of the measured "cohesion" on several environmental and compositional factors, from which it can be concluded that "a clay does not possess an invariant 'true cohesion', but rather exhibits a cohesion which is a function of a variety of factors."

Sridharan and Venkatappa Rao (100) performed direct shear tests on a calcium kaolinite and a sodium montmorillonite in order to study the effect of pore fluid dielectric constant on the shear strength behavior of clays. Some of these tests were performed at normal effective stress levels as low as 0.15 kg/cm^2 . Although the effective stresses in these tests were larger than those used in the Ladd and Kinner study, they were still low enough to be largely influenced by the same mechanisms that are responsible for cohesion in soils.

Increasing the dielectric constant of the pore fluid should increase particle repulsions due to diffuse double-layer interactions and decrease the attractive forces associated with van der Waals interactions. The results presented by Sridharan and Venkatappa Rao show marked increases in shear strength in samples with lower dielectric constants. Water ($D = 80.4$ at 25° C) has the highest dielectric constant of the pore fluids used in the study. n-Hexane ($D = 1.89$ at 25° C) and carbon tetrachloride ($D = 2.28$ at 25° C) were two of the pore fluids with the lowest dielectric constants. In both the kaolinite and montmorillonite samples the n-hexane and carbon tetrachloride samples showed substantially higher shear strengths than the water samples. The strength changes associated with the kaolinite samples were much larger than those of the montmorillonite samples. The n-benzene and carbon tetrachloride kaolinite samples were found to be approximately 3 times stronger than those with water as the pore fluid. These results are in agreement with those of Ladd and Kinner (45), who found that the measured cohesion of Peerless Kaolinite approximately doubled when ethylene glycol ($D = 37$ at 25° C) was used as the pore fluid instead of water. Ladd and Kinner note, however, that variation of the dielectric constant may have influences on the clay-water system other than those described previously and, thus, these results should be interpreted cautiously.

In another study, Bishop and Garga (10) performed drained direct tension tests on samples of London Clay. Unfortunately, in this study the tests were performed on samples of undisturbed soil in which cementation or some other form of bonding was present. This bonding masked the true behavior of the material being tested. Still, the work of Bishop and Garga is instructive because it illustrates an experimental technique that has been used successfully to test clays at low effective stress levels. The drained direct tension tests performed by Bishop and Garga will be discussed in detail in a subsequent section of this report.

Experiment Studies of Friction in Soil Mechanics

Studies of the frictional characteristics of soil minerals are helpful in gaining an understanding of strength generation in soils. The frictional behavior of these materials is difficult to evaluate, however, because the results of such studies are highly dependent on the method of sample preparation and surface cleaning (13, 14, 15, 28). The results are also strongly influenced by such environmental factors as temperature, testing technique and strain rate. Studies by Bromwell (15) on the frictional behavior of quartz surfaces clearly illustrate the important role that test conditions have on measurements of friction angles. Depending on the roughness and cleanliness of the quartz surfaces and test conditions, Bromwell measured friction angles ranging between 8 and 45 degrees.

Horne and Deere (28) investigated the influence of surface moisture, surface roughness and sliding rate on the frictional resistance developed between surfaces of several soil-forming minerals. The investigation was carried out by means of direct shear tests in which three hemispherical buttons of a mineral were pulled across a plane surface of the same mineral. The tests were carried out on smooth, "clean" surfaces. "Clean" surfaces were defined by the authors as surfaces from which the grosser impurities had been removed. No attempt was made to make the surfaces chemically clean, and, therefore, the surfaces definitely contained impurities.

One of the most significant findings of the Horne and Deere study was that the frictional characteristics of the layered silicate minerals tested responded differently to changes in surface moisture condition than did the massive structured silicate minerals. They found that the frictional resistance between massive structured minerals increased with increasing surface moisture, whereas the frictional resistance decreased in the layered silicate minerals when the surface moisture content was increased. Thus, water was found to act as a lubricant when applied to the surfaces of layered silicate minerals and an antilubricant when applied to the surfaces of massive structured silicate minerals. These findings were also found to be applicable to the results of tests performed in air at various relative humidities. Higher frictional resistances were associated with higher relative humidities with the massive structured minerals, while the reverse relationship was found in the layered silicate minerals. Furthermore, the antilubricating action of water on the massive structured silicate minerals diminished rapidly with increasing surface roughness.

Horne and Deere attributed the antilubricating effect of water to the presence of a contaminant film that coats the mineral surfaces. This film reduces the frictional resistance developed between surfaces (13, 14). Disorientation of this film will tend to increase the frictional resistance between surfaces that have these contaminant coatings. Horne and Deere hypothesized that water, being highly polar, disrupted the adsorbed contaminant layer. To test this hypothesis they performed direct shear tests on quartz surfaces using a variety of polar and non-polar fluids. They found that highly polar fluids such as water, ethylene glycol and aniline had a much greater antilubricating effect than did non-polar fluids such as carbon tetrachloride.

When the layered silicate minerals were tested, it was found that the non-polar fluids were much less effective lubricants than was water. Ethylene glycol, which is highly polar, was found to have a relatively small lubricating effect on the surfaces of the layered silicate minerals, in apparent contradiction to the proposed hypothesis.

Horne and Deere suggest that the reason that fluids act as lubricants on the layered silicates and not on the massive structured silicates is due to the fact that when two contaminated surfaces of the layered silicate minerals are slid past one another, scratching of the surface occurs (14). This scratching exposes fresh cleavage planes since these minerals are easily separated along their basal planes. The freshly cleaved parts of the surfaces will exhibit increased attraction towards each other, increasing the frictional resistance, until the fresh surfaces are once again covered by an adsorbed layer. The presence of a fluid apparently expedites the adsorption of a surface film. This process of cleavage and adsorption would be a very complex process dependent on the nature of the cleaved surface, the fluid covering the mineral surfaces and the test conditions.

An interpretation slightly different than Horne and Deere's can be made by using the adhesion theory of friction and reference to Figure 15. When two layered silicate surfaces are slid past each other, some of the interacting asperities will inevitably cause some amount of shearing along the basal cleavage surfaces of these materials. The strength of any contact formed at these freshly cleaved surfaces will be dependent on the degree to which the contact has been covered by a contaminant film, as illustrated in Figure 15. In the presence of water or some other fluid, the freshly cleaved surfaces will be contaminated more quickly than would be the case if the surfaces were just exposed to the atmosphere. Hence, in the presence of a fluid the contacts will be weaker, causing the frictional resistance to be lower than it would be under atmospheric conditions.

Mitchell (71) provides a third possible explanation for the lubricating action of fluids on sheet minerals. He notes that in air adsorbed surface films are thin and any surface ions that might be present are not fully hydrated. During sliding this thin film is not easily disrupted. When the surfaces of these materials are wetted, the thickness of the surface film is increased and the surface ions are more fully hydrated. As a result, the mobility of this layer is increased, leading to a decrease in the frictional resistance that can be developed.

The effect of surface contaminants on the frictional behavior of quartz was investigated by Bromwell (15) through the use of ultra-high vacuums (up to 10^{-10} torr), elevated temperatures and special cleaning and handling techniques.

In order to obtain a stable clean surface free of any adsorbed contaminants Bromwell had to provide a clean ultra-high vacuum. In addition, elevated temperatures were generally needed to cause the spontaneous desorption of surface contaminants. For instance, the relationship between the adsorption energy of a gas, Q (cal/mole), the absolute temperature T , and the pressure of the gas, p (mm of Hg), in order to maintain one tenth of a molecular layer of gas on a solid surface can be approximated by (15),

$$Q = 5.1T (5 - \log_{10}p). \quad (26)$$

This equation shows that the energy required for adsorption increases proportionally as the temperature increases and logarithmically as the pressure decreases. Therefore, adsorption is least likely to occur under conditions of high temperature and low pressure.

Bromwell performed direct shear tests on square 1.5 inch wide samples of high-purity quartz crystals. Smooth surfaces were prepared by grinding with a No. 600 grit diamond wheel. Rough surfaces were obtained by grinding with a No. 220 grit diamond wheel. All specimens were cleaned with water and detergent. Some of the samples were also chemically cleaned. This involved a six-step procedure using chemical solvents and de-greasers.

The results of Bromwell's tests, shown in Figure 24 and Figure 25, clearly illustrated that the coefficient of friction of quartz is highly dependent on the condition of the surface of sliding. Clean surfaces led to increased mineral-to-mineral contact, increased adhesion and, thus, increased frictional resistance. Bromwell also found that chemically cleaned rough surfaces had lower frictional resistances than did chemically cleaned smooth surfaces. This behavior was attributed to the difficulty in obtaining clean, rough surfaces. Apparently, the cleaning procedure used in the study was not entirely effective in cleaning the rough surfaces.

Bowden and Hanwell (13) studied the frictional behavior of crystal surfaces in ultra-high vacuums. They found that the frictional resistance of diamond, magnesium oxide and sapphire increased by a factor of up to ten when it was possible to obtain clean surfaces under high vacuum conditions. They also noted that adsorption of even a few molecular layers of gas would decrease the frictional resistance to close to a material's atmosphere value.

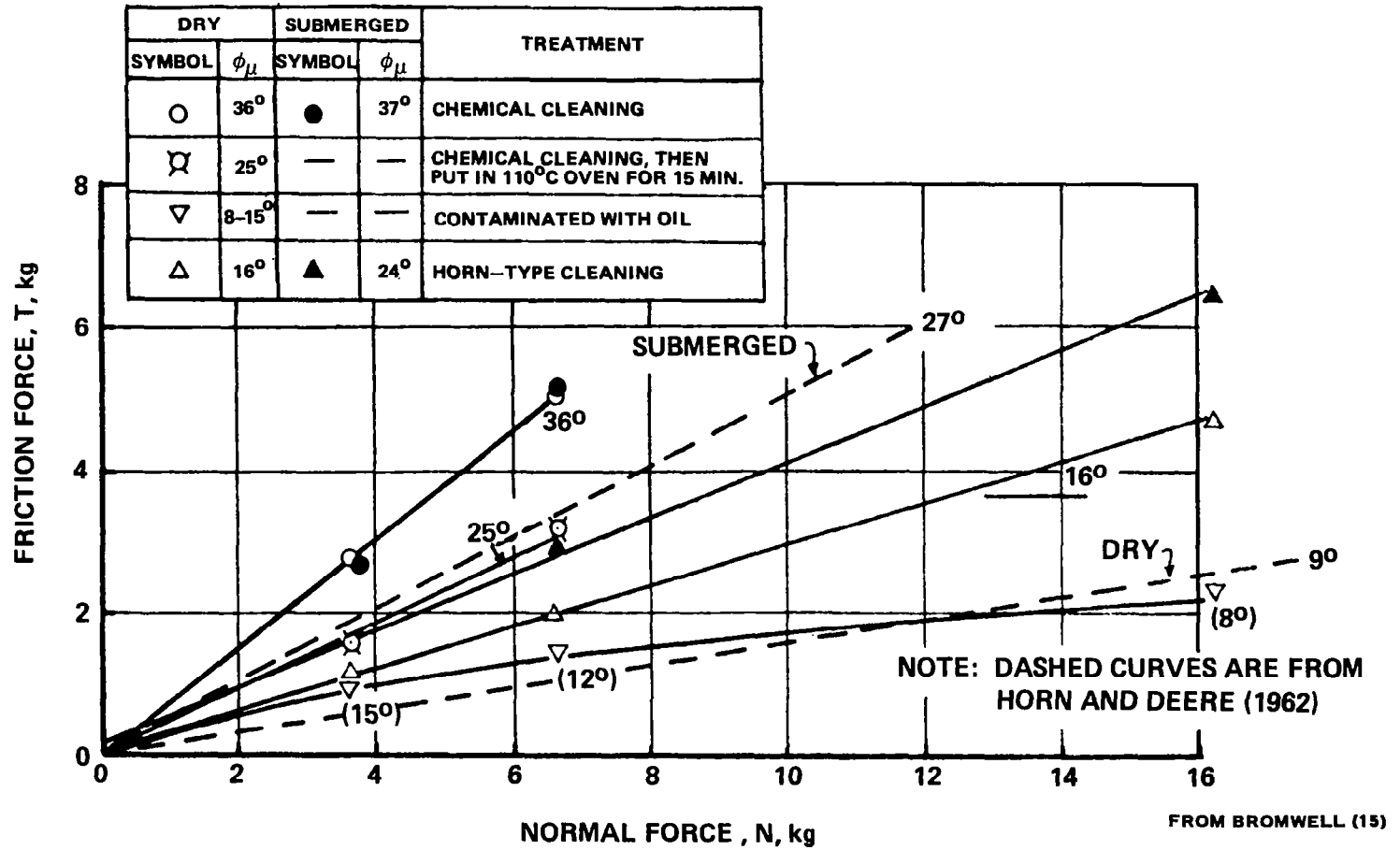


Figure 24. Results of atmospheric friction tests on smooth quartz surface.

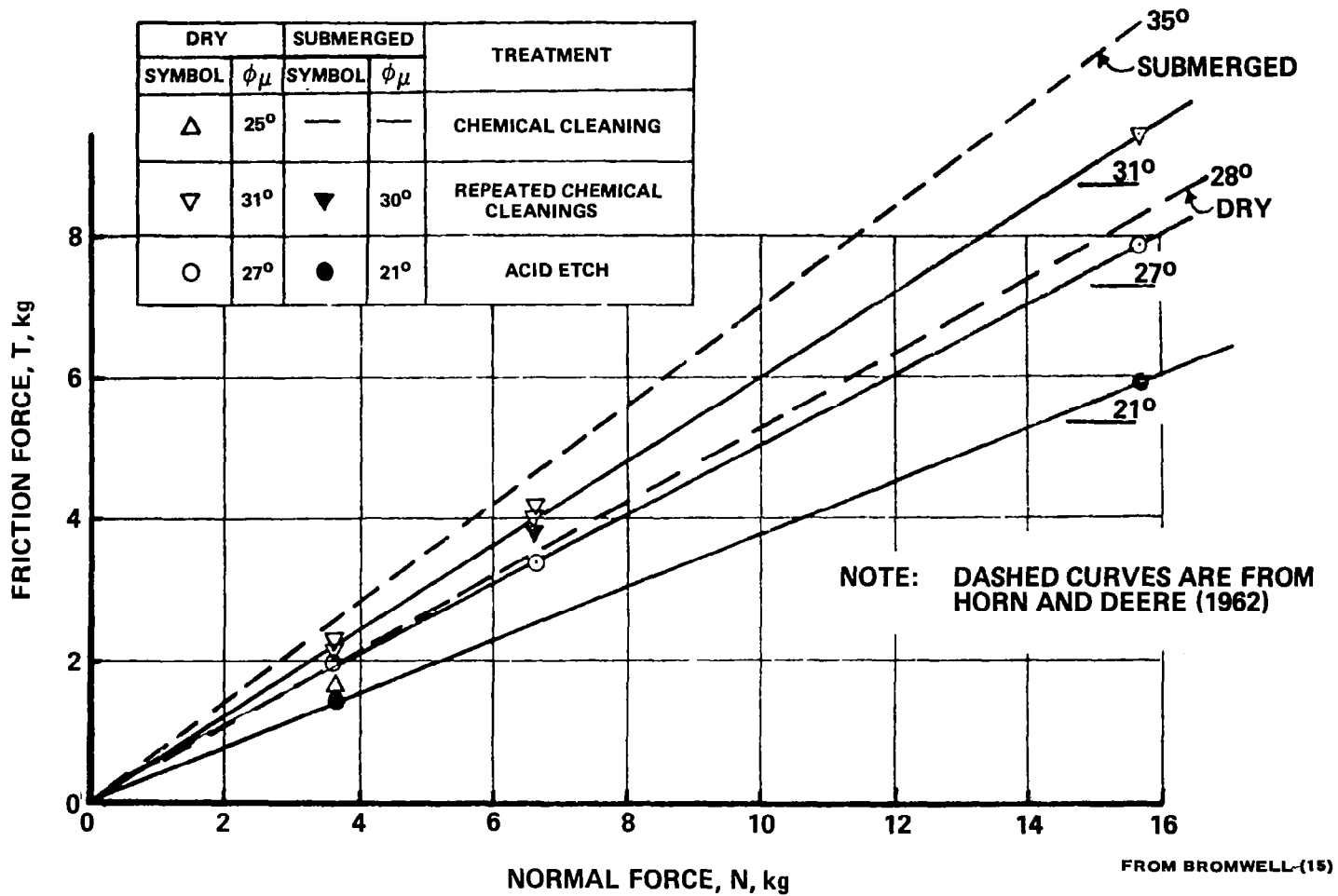


Figure 25. Results of atmospheric friction tests on rough quartz surfaces.

Yam and Kenny (108) emphasized that while the adhesion theory of friction is generally valid for smooth, clean surfaces, in most systems there will be contributions to friction other than adhesion at asperities. These other contributions include elastic deformations of surface protrusions, and dilatancy. They point out that for rough surfaces these additional contributions can both add significantly to the frictional resistance and cause the coefficient of friction to decrease with increasing normal load.

There are several pertinent conclusions that can be drawn from the study of the frictional behavior of soil and rock-forming minerals that are important to the shearing resistance of fine-grained soils. These studies have shown, for instance, that there is no one true coefficient of friction for a given mineral. Instead, there exists a range of possible values, the exact value in any given situation being dependent on a number of compositional and environmental factors. This is entirely analogous to Ladd and Kinner's (45) findings on the measured cohesion in clay soils. Thus, while the role of adsorbed water in the development of friction between two clay mineral surfaces is not entirely clear, we should fully expect the friction that develops between two particles of clay due to a given interparticle force to be dependent to at least some degree on factors such as the valence of adsorbed cations, the pore fluid electrolyte concentration and the dielectric constant of the pore fluid.

Needless to say, measuring the frictional resistance developed between two clay particles is an all but impossible task due to the small size of the particles. As will be seen, the bulk frictional properties of clay soils can be measured, but the results don't give explicit, unambiguous information on the friction developed between two individual particles.

Due to the fact that the structure of the clay-forming minerals is very similar to that of the layered silicates studied by Horne and Deere (28) similarities in behavior should be expected. Thus, it would be expected that water acts as a lubricant between clay particles and reduces the frictional resistance that could be developed by two clay particles sliding past each other. The friction angles measured for the layered silicates are probably applicable to the clay minerals (71). Support for this comes from the fact that the measured residual friction angles of several pure clay minerals obtained by Kenny (34) are similar to those obtained for muscovite, biotite, phlogopite and chlorite by Horne and Deere.

Several researchers have investigated the friction angles developed in bulk samples of clay minerals. Probably the most extensive of these studies was the one carried out by Olson and his co-workers (64, 80). Olson (80) attributed strength generation in clay soils to physical and chemical interactions between particles. Physical interactions are "controlled largely by the size, shape, packing, and physical properties of the individual grains and the friction between them." Clearly, interparticle friction in the Bowden-Tabor sense is one component of the physical interaction. Chemical interactions include particle interactions due to diffuse double layers, van der Waals forces and ionic forces. Physical

interactions predominate in soils in the sand and silt range and in some clays. In these soils gravitational forces due to the weight of the soil far exceed forces due to the system chemistry. Chemical interactions may become important for some clay soils under certain conditions, such as during initial fabric formation, during consolidation and during shear at very low stress levels.

In order to better understand the conditions under which physical and chemical interactions are of importance in clay soils, Olson and his co-workers ran a large number of drained and undrained triaxial tests on samples of kaolinite, illite and montmorillonite. These tests were performed on homoionic (sodium and calcium) samples prepared at varying pore water salt concentrations. It was reasoned that if chemical interactions were important in the development of shearing resistance in a certain soil, variation of the factors that affect chemical interactions should also have an effect on the measured shear strength.

The properties of the three soils tested in Olson's investigations are presented in Table 5. A summary to the triaxial tests performed is given in Table 6. Presented below are the findings of these investigations. The results of the tests on each of the three different clays tested will be presented separately.

Kaolinite. Figure 26 compares the effective stress shear strength envelopes of calcium and sodium kaolinite at a pH of 7 and a pore fluid salt concentration of .01 N. The results clearly indicate that under these environmental conditions the effect of cation valence is minimal. This result combined with the relatively high friction angle ($\phi' = 28^\circ$) led Olson to the conclusion that diffuse double-layer repulsive forces have little influence on the development of shearing resistance in kaolinite under these conditions. In addition, no difference in failure envelope was found between samples of normally consolidated and overconsolidated kaolinite.

Bailey (8) has pointed out that at pH levels of about 7 or less the edge charge on a kaolinite particle is positive. Under these conditions flocculated soil fabrics are likely. Olson hypothesized that the lack of any double-layer effect in kaolinite at a pH of 7 may be due to the predominance of the edge-to-face and edge-to-edge particle contacts. These types of particle arrangements would tend to minimize the repulsive forces due to diffuse double-layer interactions. To investigate this possibility a series of tests were performed on a Na-kaolinite with a pore fluid salt concentration of .001 N and a pH of 9.5, a chemistry that should have produced a dispersed structure in the kaolinite and maximum opportunity for diffuse double-layer interactions. Tests were also performed at pH values of 5 and 7. Significantly, when the soils were consolidated to the same effective stress, differences in void ratios existed for the samples at the different pH's. This was a clear indication of differences in the fabrics and internal forces acting in the various samples. Still, as seen in Figure 27, no significant differences existed in the failure envelopes for any of the normally consolidated samples. Overconsolidated samples at a pH of 5 showed only a slightly higher effective stress friction angle.

TABLE 5. PROPERTIES OF CLAYS

Mineral (1)	Supplier, Source Trade Name (2)	Liquid Limit in percent (3)	Plastic Limit in percent (4)	Specific Gravity (5)	Surface Area in square meters per gram (6)	Cation Exchange Capacity milli- equivalents per 100 grams (7)	Fraction Finer than 0.002 millimeter (8)	Estimated Ratio of Diameter to Thickness (9)
Kaolinite	Minerals and Chemicals Philipp Corp., Klondyke clay, McIntyre, Georgia	40 to 50	27 to 31	2.65	14	2.2	47	2-5
Illite	Fithian Illinois	80 to 87	30 to 35	2.80	90	28	100	10-50
Montmorillonite	American Colloid Company, Wyoming, Volclay	190 to 1160	31 to 47	2.65 to 2.80	500 to 700	100	97	150-500

FROM OLSON (80)

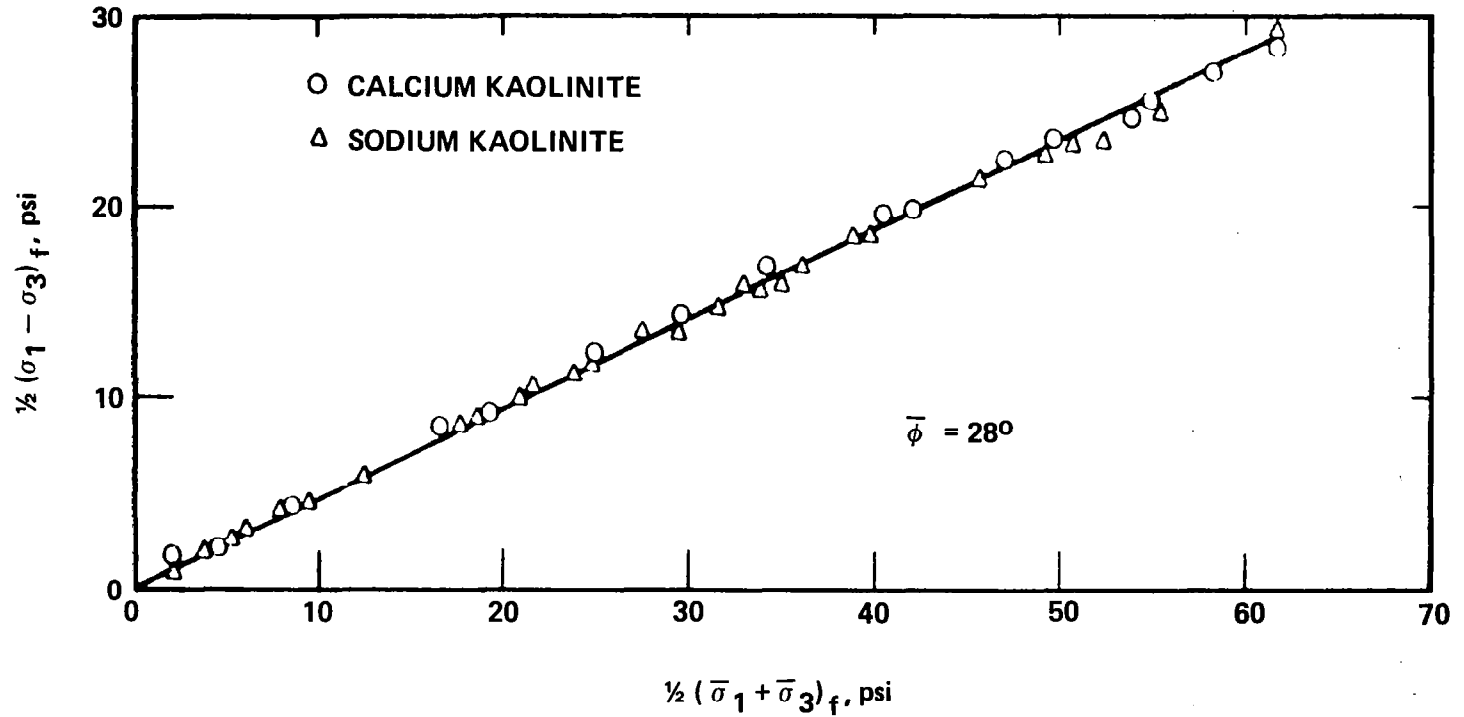
TABLE 6. SUMMARY OF TRIAXIAL COMPRESSION TESTS

Mineral (1)	Cation (2)	pH (3)	Electrolyte concentration, N (4)	Test type (5)	Number of tests (6)
Kaolinite	Ca ⁺⁺	7	0.01	\bar{R}	16
				S	19
	Na ⁺	5	0.001	\bar{R}	16
				S	17
				\bar{R}	101
Illite	Ca ⁺⁺	7	0.01	\bar{R}	14
			0.001	\bar{R}	21
			1	\bar{R}	3
			0.1	\bar{R}	3
			0.01	\bar{R}	4
Montmorillonite	Na ⁺	7	0.001	\bar{R}	3
			0.0003	\bar{R}	3
			0.01	\bar{R}	3
			0.1	\bar{R}	33
			0.01	\bar{R}	29
			0.007	S	12
				\bar{R}	17
Kaolinite	Ca ⁺⁺	7	0.001	\bar{R}	4
			0.01	\bar{R}	13
			0.1	\bar{R}	4
	Na ⁺	7	0.01	S	4
			0.001	\bar{R}	3
			0.01	\bar{R}	4
			0.1	\bar{R}	5

FROM OLSON (80)

From these results Olson concluded that physical interactions were primarily responsible for the development of shearing resistance in kaolinite. Chemical variables influenced the original structure of the specimens and thus the volume change tendencies and undrained shear strength, but not the effective stress shear strength.

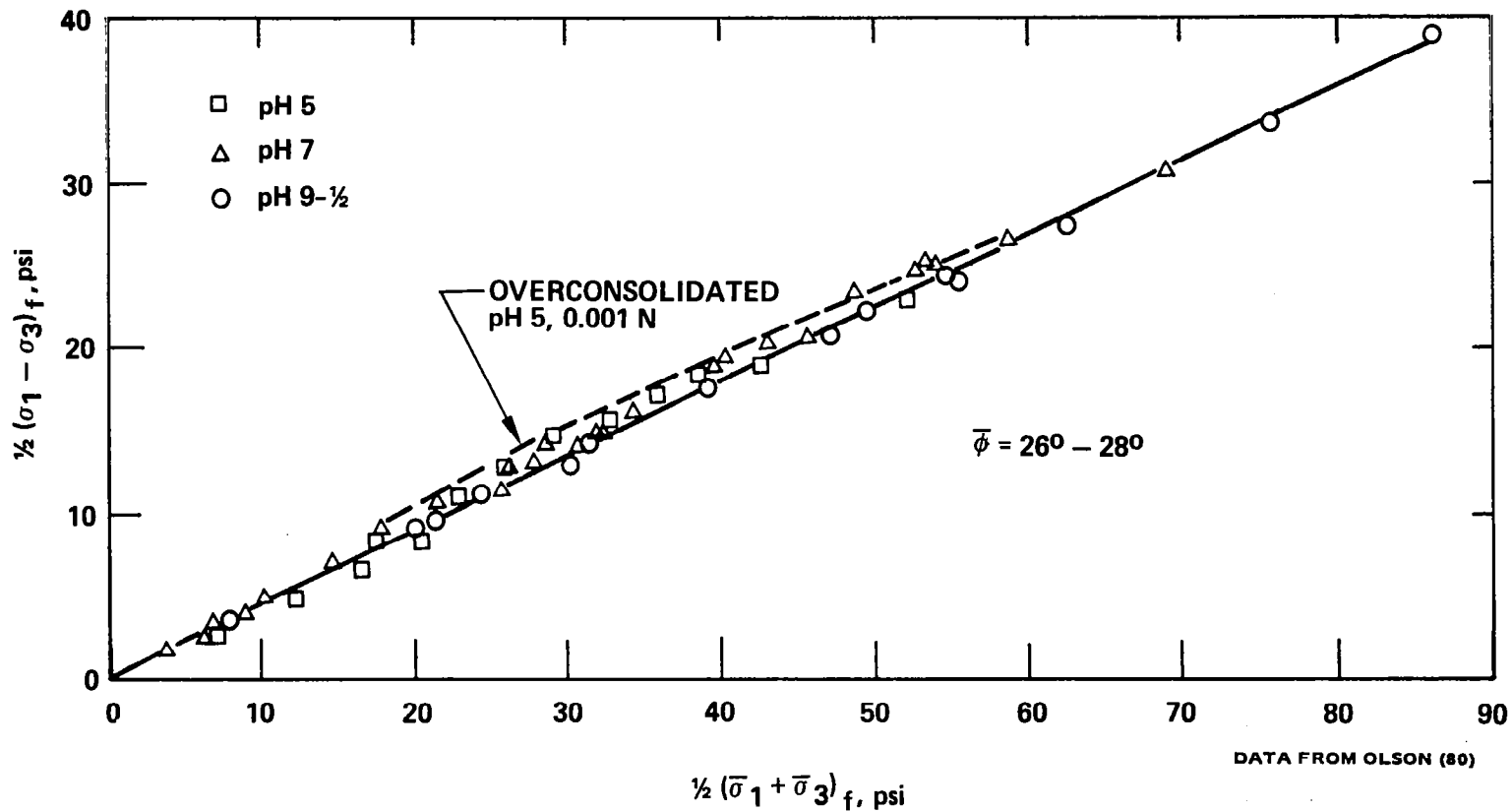
Illite. The results of the tests on illite are shown in Figure 28, 29, and 30. Based on the results shown in Figures 28 and 29, it can be concluded that physical interactions are primarily responsible for shear strength generation in calcium illites. This is based on the observations that large changes in pore fluid salt concentrations had practically no effect on the position of the effective stress failure envelope, nor did



VARIATION OF MAXIMUM SHEAR STRESS WITH AVERAGE EFFECTIVE NORMAL STRESS FOR SEDIMENTED SAMPLES OF KAOLINITE, pH 7

DATA FROM OLSON (80)

Figure 26. Failure defined as point of stress path tangency.



VARIATION OF MAXIMUM SHEAR STRESS WITH AVERAGE EFFECTIVE NORMAL STRESS FOR SEDIMENTED SAMPLES OF SODIUM KAOLINITE

Figure 27. Failure defined as point of stress path tangency.

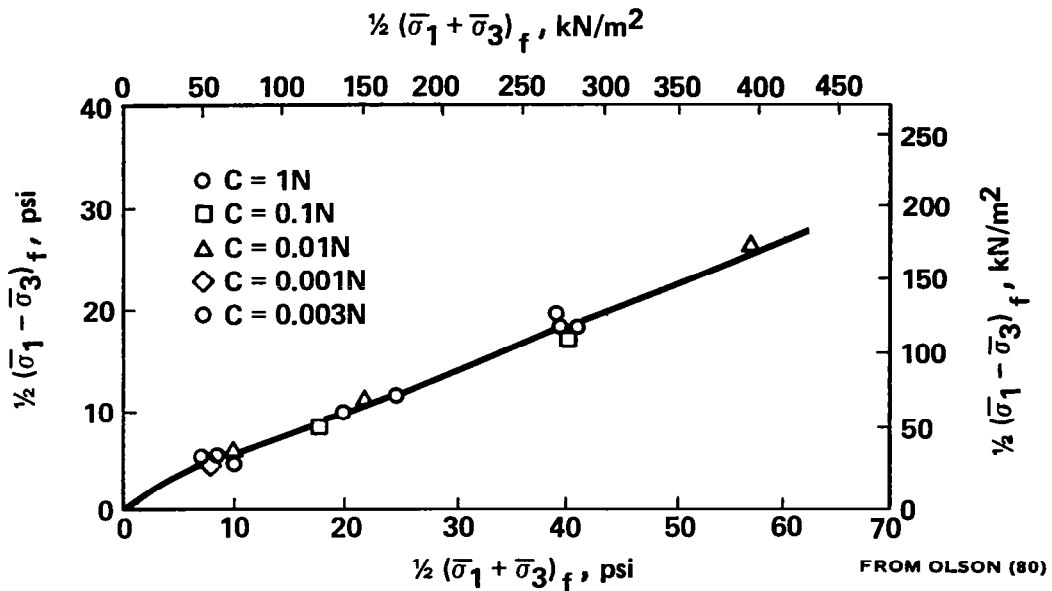


Figure 28. Failure envelope for \bar{R} triaxial compression tests on calcium illite at pH = 7 and range in pore water electrolyte concentration (failure was defined as point of stress path tangency).

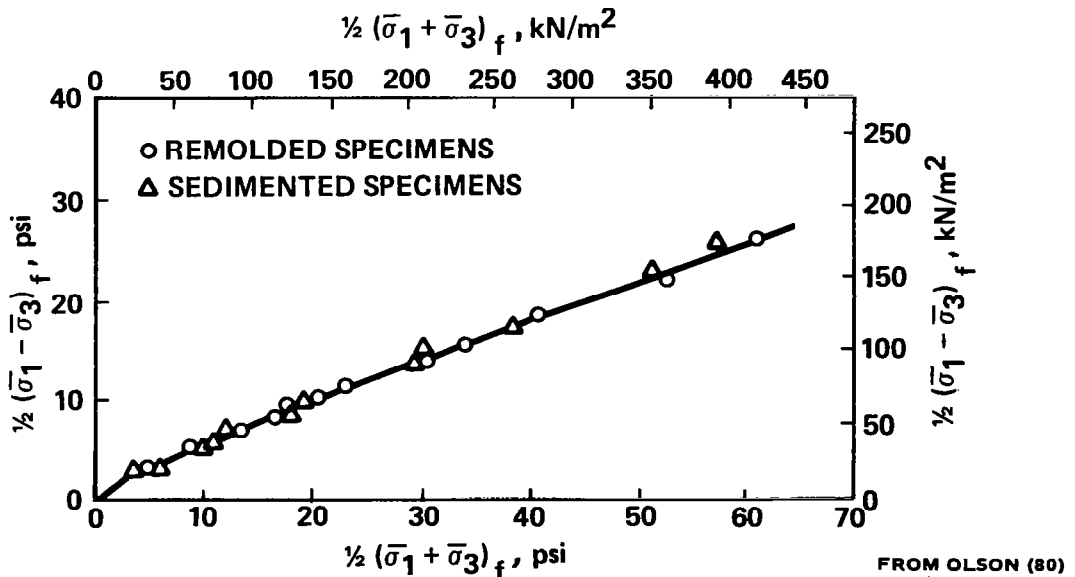


Figure 29. Failure envelope for remolded and sedimented specimens of 0.01 N calcium illite (failure was defined as point of stress-path tangency).

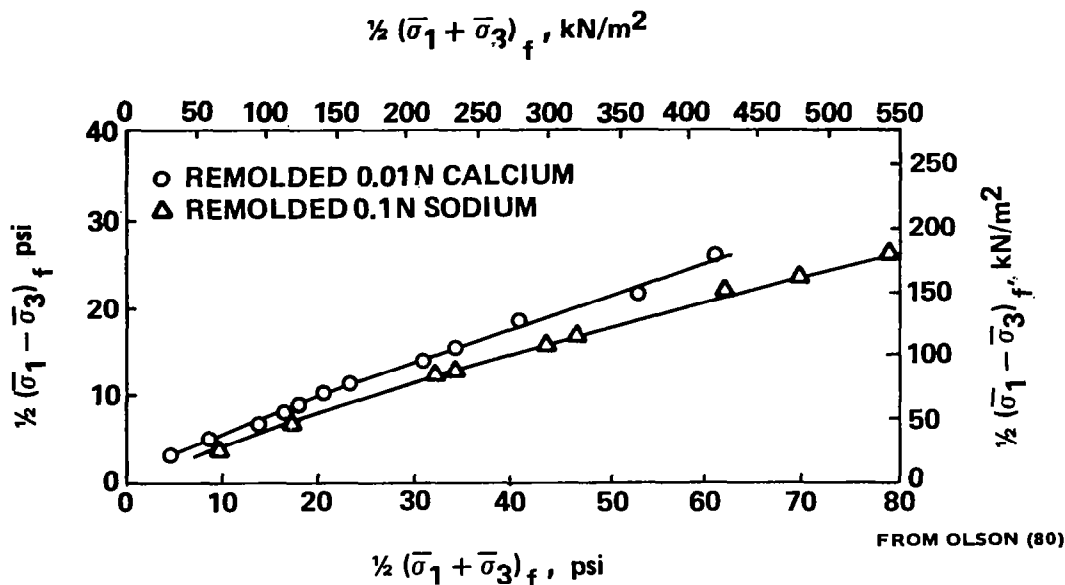


Figure 30. Failure envelopes for remolded specimens of calcium and sodium illite (failure was defined at point of stress-path tangency).

a change from an essentially flocculated fabric (sedimented specimens) to an essentially dispersed fabric (remolded specimens). Apparently, double-layer interactions play only a small role in the generation of shear strength in this soil.

Figure 30 illustrates the results from tests on both a calcium illite and a sodium illite. Clearly, the strengths of the sodium illite specimens were lower than those of the calcium illite specimens. Olson also found that the effective stress failure envelope for sodium illite samples at a pore fluid electrolyte concentration of .007 N was about 15% lower than that for sodium illite samples at pore fluid electrolyte concentrations of .1 N. Thus, it appears that chemical interactions, particularly diffuse double-layer interactions, play a role in determining the effective stress shear strength envelope of sodium illite.

Montmorillonite. The results of tests on both calcium and sodium montmorillonite are shown in Figure 31. The shear strength envelope for the calcium montmorillonite is seen to be relatively independent of pore fluid electrolyte concentration. Overconsolidated samples of calcium montmorillonite showed only slightly higher strengths than their normally consolidated counterparts.

The small effect of diffuse double layers has been attributed by Mesri and Olson (64) to the formation of domains⁵ in calcium montmorillonite. Alymore and Quirk (4) indicate that in this soil domains form progressively during sedimentation and consolidation. This process seems

⁵Domains are defined as submicroscopic regions within which the clay particles are in parallel array.

to be of an irreversible nature. Once domains are formed, there apparently exists a potential attractive energy barrier that keeps the basal spacing between calcium montmorillonite plates at less than 19\AA . Mesri and Olson suggest that the domains act as equidimensional particles and that physical interactions are much more probable than in sodium montmorillonite. However, at relatively low effective stresses the slope of the failure envelope for calcium montmorillonite was found to be about 15 degrees, while at higher consolidation pressures the slope was reduced to only 10 degrees. This decrease was attributed by the authors to greater parallelism of the clay plates at higher consolidation pressures, which in turn suggests at least a limited role for diffuse double-layer forces.

Figure 31 also illustrates the results of the tests on sodium montmorillonite. Two points are immediately apparent. First, the slope of the failure envelope for sodium montmorillonite is very small, decreasing from 4 to 0 degrees in the range of consolidation pressures studied by Mesri and Olson. Secondly, contrary to expectations, the failure envelope for sodium montmorillonite showed no dependence on the pore fluid electrolyte concentration.

The low shear strengths exhibited by the sodium montmorillonite can be understood in terms of its structure (64, 71, 80). It is thought that the large osmotic gradients set up between the unit layers of this soil cause interlayer water adsorption and separation of the soil into flakes only 10\AA thick. Olson (80) sees these flakes as being so flexible that essentially all interactions must be of the face-to-face type. The large surface areas of these flakes lead to "low intergranular stresses and a strong probability that the particles are always separated by at least one layer of adsorbed water if not a full diffuse double layer." If contacts are through adsorbed water layers, small changes in normal stress should cause a change in interparticle spacings, but little change in the soil's ability to resist shear, since the shearing resistance is still largely seated in the adsorbed water layer. Thus, changes in pore fluid salt concentration would cause some change in the normal stress acting between particles but little change in shearing resistance, as observed.

From these studies, Olson (80) concluded that in most situations physical interactions are the dominant mechanisms responsible for strength generation in clay soils. Exceptions to this are seen in the behavior of sodium illite and sodium montmorillonite. Strength generated through physical interactions will be largely dependent on particle size and geometry, which is in agreement with the adhesion theory of friction, once the role of surface contaminants is taken into account. Larger, thicker particles will be stronger than smaller flaky particles. Based on this reasoning, kaolinite should be stronger than illite which should be stronger than montmorillonite. The results of the investigations carried out by Olson and his co-workers support this view.

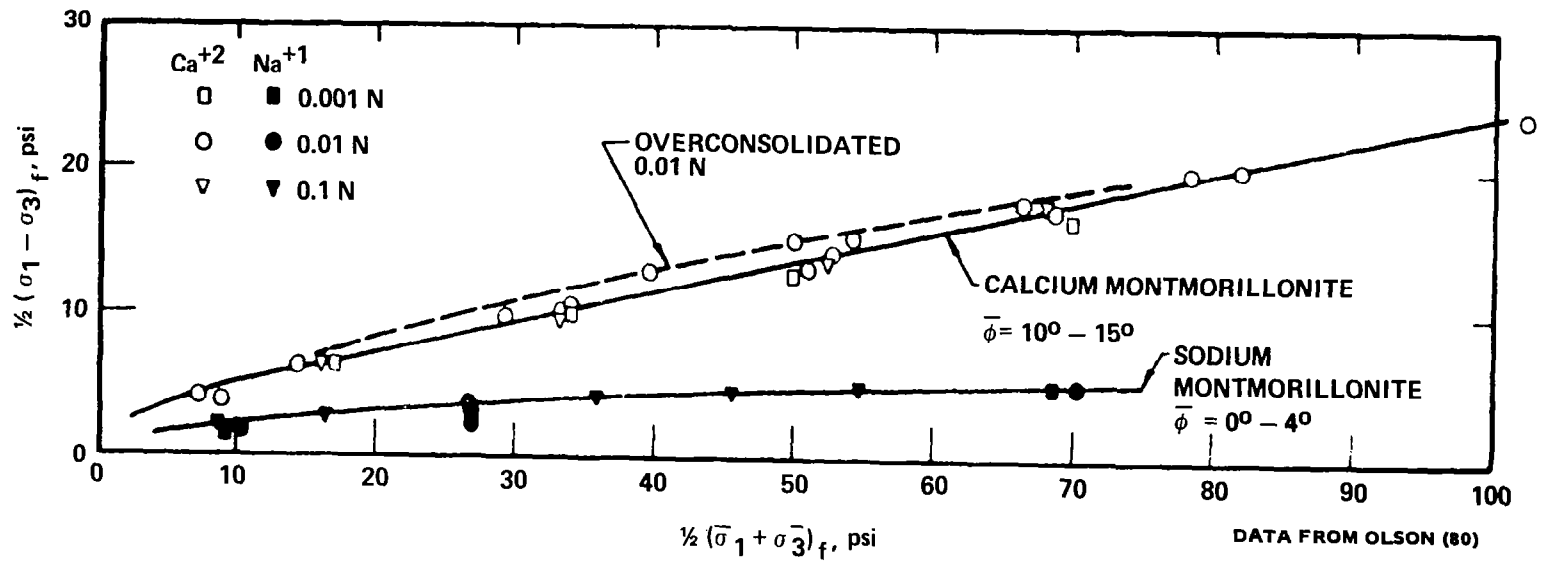


Figure 31. Failure defined as point of stress path tangency.

It should be pointed out these findings are based for the most part on tests performed at consolidation stresses greater than those of most interest in the present study. As was pointed out earlier, at very low stress levels it is likely that chemical interactions become of increased importance relative to physical interactions. Ladd and Kinner's (45) study seems to support this contention.

Mesri (63) presented a slightly different interpretation of the shear strength data on kaolinite, illite and montmorillonite presented by Olson (80). Shear strength, in Mesri's view, is developed solely through frictional resistance at interparticle contacts. An interparticle contact exists at points of approach of adjacent particles of less than 10\AA . Bonding at these contacts can take place through hydrogen bonding, cation linkage and primary valence bonding. The shear strength is a function of the number of interparticle contacts. Long-range interparticle forces are seen as influencing particle size, shape, aggregation tendencies, and arrangement during sedimentation and consolidation. They influence particle rearrangement during shear, but they are not seen as contributing to the development of shearing resistance at interparticle contacts.

The insensitivity of the shear strength of kaolinite to pH, adsorbed cation and pore fluid electrolyte concentration is seen by Mesri as reflecting an internal structure of relatively large, stiff, randomly oriented particles. Edge-to-face contacts predominate, leading to a large number of short range contacts. Whereas Olson attributes the higher strength of calcium illite over that of sodium illite to the larger diffuse double-layer repulsions in sodium illite, Mesri suggests instead that the higher strength of calcium illite is due to some degree of domain formation which leads to larger, stiffer particles and more short-range contacts. In montmorillonite, domain formation occurs in calcium montmorillonite, leading to increased particle size and a fair number of short-range contacts. In sodium montmorillonite the large double-layer repulsive forces and the flaky nature of the particles are seen as leading to almost no short-range interparticle contacts, and therefore the strength of sodium montmorillonite is very small. In addition, increased consolidation pressure in montmorillonite does not substantially increase short-range interaction between particles which is manifested through the relatively small strength gains in montmorillonites with increased consolidation pressures.

Experimental Studies of Shear Strength Generation in Cemented Sensitive Soils

Most of the studies involving testing of undisturbed cohesive soils at very low effective stress levels or in tension have been carried out on Canadian or Scandinavian cemented sensitive clays. Studies of this nature include the work of Bjerrum and Wu (12), Crawford (19), Conlon (18), Kenny Moun and Beere (35), Mitchell (74), Loisel, Massiera and Sainani (53), Sangrey (95), and Bjerrum (11).

One of the early studies of a cemented sensitive clay was the one by Bjerrum and Wu (12) on the Lilla Edet Clay of Sweden. From the results of drained and undrained triaxial tests the authors used Hvorslev's (29) procedure to calculate values for true cohesion. They found that in the Lilla Edet Clay the relationship between true cohesion and consolidation pressure consisted of two parts, depending on whether the samples were consolidated to pressures greater than or less than the preconsolidation pressure (as measured in a conventional triaxial test). At pressures below the preconsolidation pressure relatively large values of true cohesion were measured, while at pressures which exceeded the preconsolidation pressure the true cohesion was linearly proportional to the consolidation pressure. It was also found that there was a significant reduction in true cohesion in the vicinity of the preconsolidation pressure. Bjerrum and Wu concluded that the observed behavior was attributable to the presence of rigid cementation bonds which remained essentially intact at pressures below the preconsolidation pressure but broke down at pressures that exceeded this critical value.

Crawford (19) performed drained unconfined compression tests on high-quality undisturbed samples of Leda clay. This type of test was carried out with the idea that the effective stresses on the failure plane would be so small that the measured shear strength would be due in the most part to cohesion rather than to friction. The unconfined compressive strengths measured by Crawford were quite large, ranging between 1 and 2 kg/cm². In addition, the results of consolidated undrained and drained triaxial tests performed on Leda Clay indicated a behavior similar to that of the Lilla Edet Clay, with the critical consolidation pressure for the Leda Clay being about 4 kg/cm². Crawford concluded that the large strength exhibited by the Leda Clay at low effective stresses is due to the pressure of cementation bonds and other "intrinsic stresses," and that at stresses less than the preconsolidation pressure, these bonds control failure and put the applicability of the Mohr-Coulomb failure theory in doubt.

Although it is clear that clays such as the Lilla Edet and the Leda are cemented, it is not entirely clear what the cementing agents are or what the precise cementing mechanism is (95). Calcium carbonate is one cementing agent that has been reported frequently, and Bjerrum (11) suggests that this material exists as a uniform smear on the surfaces of the mineral particles.

Kenney et al. (35) used EDTA (ethylenediaminetetraacetic acid) to strip a clay from Labrador, Canada, of calcium carbonate, gypsum and iron compounds. The removal of these cementing agents caused significant property changes in the soil. Loiselle et al. (53) also used EDTA treatments to study cementation in a cemented Canadian clay. They attributed cementation to amorphous silica and alumina as well as ferrous oxide precipitates and calcium carbonate. In addition to these chemicals, Sangrey (95) cites amorphous manganese oxides and organic compounds as other possible cementing agents.

The cementation bonds formed during the geologic history of these Canadian and Scandinavian clays remain essentially intact during unloading. This results in the maintenance of intergranular stresses, elastic strain energy and shear strength during the unloading process (11). The inhibition of swelling during unloading can be attributed to the tensile strength of these bonds.

When subjected to normal and shear stresses, these soils do not respond in the same way as noncemented soils. As an example of this difference in behavior, Sangrey (95) noted that the typical consolidation curve for a sensitive cemented clay plots as a straight line on a void ratio-pressure diagram as long as the cementation bonds are not broken. As soon as the bonds are ruptured, however, the plot is no longer linear. Instead it shows the type of nonlinear behavior typical for noncemented soils. Sangrey also notes that another unique feature of these soils is the independence of the void ratio on the effective principal stress ratio during consolidation. The void ratio during consolidation is a function only of the mean normal effective stress. These unique properties are illustrated for Labrador clay in Figure 32.

Cemented sensitive clays also exhibit low strains to failure during triaxial and direct shear tests. At stresses well below the measured preconsolidation pressure, yield usually occurs at axial strains of 1% or less. Under these conditions, stress-strain curves tend to be nearly linear. The yield loci of these soils at low stress levels is governed by the strength of the cementation bonds. Only after the mean normal effective stress exceeds the level required to break the cementation bonds does the yield locus follow the Mohr-Coulomb failure envelope.

Based on much experimental data, and particularly the work of Conlon (18) and Sangrey (95), a hypothetical yield curve can be described for these cemented materials. This curve is shown in Figure 33. Point a in the figure corresponds to the tensile strength of the material and is typically quite small. At extremely low values of mean normal stress the yield locus increases linearly with increases in mean normal stress. This behavior corresponds to line segment abc in Figure 33. Sangrey attributes this linear behavior to fissuring in the soil. The mechanism associated with this type of failure involves the dilation of small blocks of cemented material, of the order of 1 to 10 mm in size. Dilation is made possible by natural fissuring and the very low confining stresses in the region where this behavior occurs. Apparently at higher confining stresses, the energy required for dilation is larger than the energy required to break the cementation bonds. Along the portion of the curve cd the yield locus is controlled by the rupture of cementation bonds. Since the strengths of these bonds are essentially independent of the mean normal effective stress the yield locus is either horizontal or inclined at a small upward slope.

Between points e and d the yield locus is situated below the Mohr-Coulomb failure envelope. If a soil element is situated on line segment ed, it will undergo a structural breakdown and large strains will ensue.

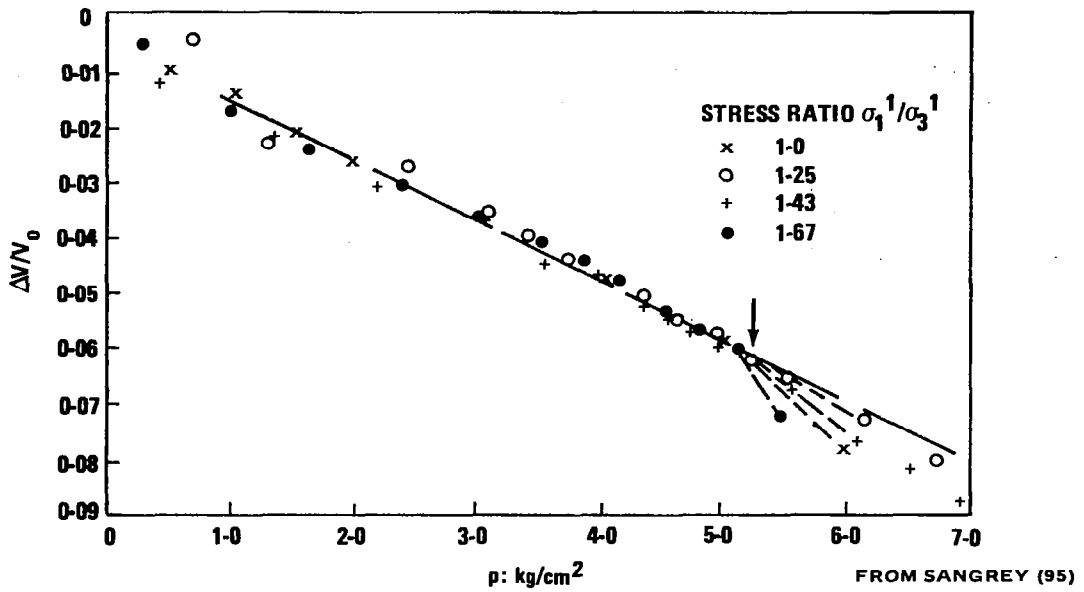


Figure 32. Volumetric compressibility of Labrador Clay under a range of stress ratios.

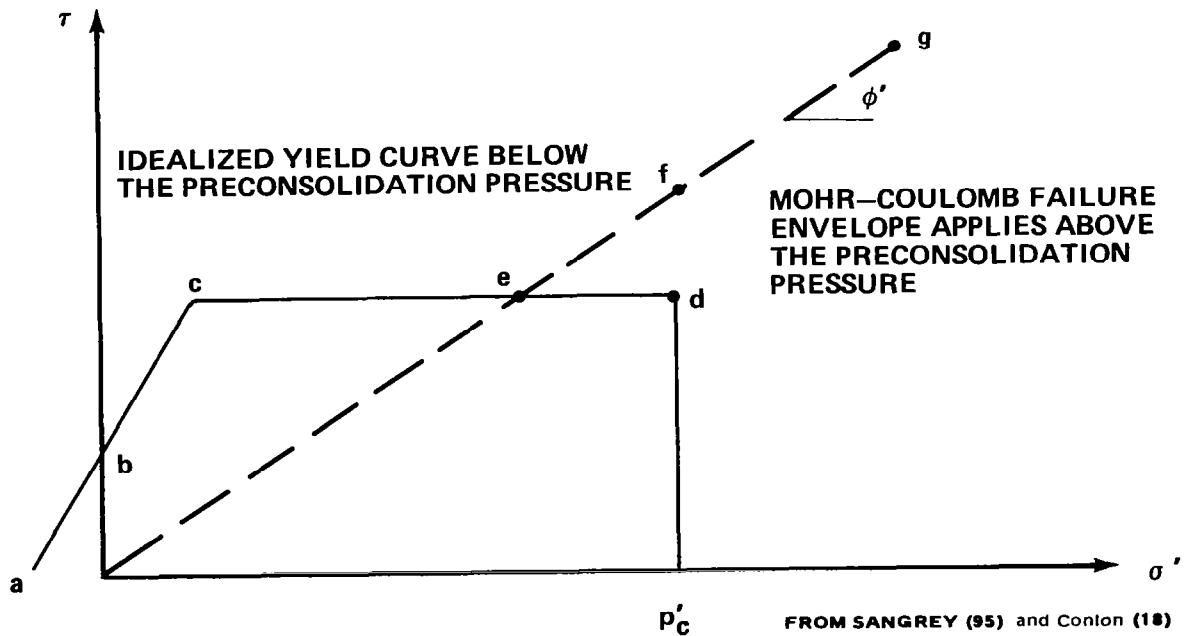


Figure 33. Hypothetical yield curve for cemented sensitive clays.

If the pore pressures that are generated during the structural breakdown are not too large, the soil element will eventually be able to sustain any increment in load that does not cause the soil to reach the Mohr-Coulomb failure envelope; however, this ability to carry additional load would only occur after large strains developed.

At consolidation pressures in excess of the preconsolidation stress and when samples are remolded, breakdown of the cementation bonds occurs and the yield locus follows a straight line defined by the classical soil strength parameter ϕ' , the angle of internal friction.

Review of the studies of cemented sensitive soils has shown that relatively large values of cohesion and tensile strength may be measured in these soils. This cohesion or tensile strength is typically quite a bit larger than would be expected in an uncemented soil. In tests on relatively large samples at very low confining stresses strengths much smaller than the cemented soil strength may sometimes be measured due to dilation and movement of small blocks of intact material along very fine fissures. The techniques used to measure the strengths of these materials at very low stress levels and in tension may prove useful to the present study. Most notable among these techniques are drained, unconfined compression tests, triaxial extension tests and drained direct tension tests.

IV. COMPREHENSIVE HYPOTHESIS FOR STRENGTH GENERATION IN FINE-GRAINED COHESIVE SOILS

In the preceding chapters of this report literature germane to an understanding of the behavior of fine-grained cohesive soils has been reviewed. The findings of these different studies can now be used to develop a comprehensive hypothesis for the strength behavior of cohesive soils. This model will be used as a framework through which the salient features of cohesive soil behavior can be better understood. It will also serve as a guide for the evaluation of potential Spacelab experiments. The hypothesis presented here is drawn, in part, from the work of Mitchell, Singh and Campanella (73) and Matsui et al. (62).

It is hypothesized that shearing resistance in soils is generated at interparticle contacts. The strength-generating mechanism involves some form of interparticle bonding. The results of creep and consolidation studies that have employed rate process theory (6, 17, 60, 72, 73) suggest that interparticle contacts are essentially mineral-to-mineral and shearing resistance is developed through primary valence bonding at the contacts. Results from scanning electron microscope studies (61) and acoustic emissions testing (62) also seem to support the mineral-to-mineral contact viewpoint. Still, some researchers (11, 24, 82) support the view that bonding is through adsorbed water layers and that at least some of the shearing resistance developed in a soil is due to hydrogen bonding between adsorbed water layers.

Mitchell et al. (73) and Mitchell (71) have summarized the findings from the application of rate process theory which support mineral-to-mineral bonding. These include (1) experimental activation energies in the range of 30 to 45 kcal/mole, which is in the range of energies associated with primary valence bonding, (2) activation energies for sands and clays and for wet and dry soils are similar, and (3) the activation energies are independent of consolidation pressure and water content. Pusch (86), however, suggested that one or two molecular layers of water are adsorbed by the soil particles so tenaciously that drying at 100°C will not remove them and therefore the bonding mechanism in the dry soil would be expected to be similar to that of a wet soil. The results presented by Bowden and Hanwell (13) and Bromwell (15) clearly illustrate the effort required to remove adsorbed films from the surfaces of many materials.

Matsui, Ito and Abe (61) presented evidence for at least some mineral-to-mineral contact in a kaolinite soil. They used a scanning electron microscope to observe tracks formed on the surface of one group of kaolinite particles that had been sheared across another group of kaolinite particles. The microscope used had a resolving capability of 100Å. It also had a low acceleration voltage which allowed the clay particles to be observed without the need for a conductive metal coating. The clay particles were pasted as thinly as possible on small circular metal plates.

Before shearing, the surfaces were observed by microscope to confirm that no tracks existed on the particles. Two similar metal plates were then placed face-to-face under water and sheared under a normal stress of $.5 \text{ kg/cm}^2$. The surfaces were observed after shear, and scratches were found on the particle surfaces. The dimensions of the tracks varied between 1.0 and 4.2 microns in length and .1 to .6 microns in breadth.

Matsui et al. (62) suggest that further evidence of mineral-to-mineral contacts in cohesive soils is provided by the results of acoustic emissions tests in clays. Acoustic emissions are sounds generated within many materials when they deform under stress. The generation of emissions during deformations are taken by the authors as evidence in support of solid-to-solid contact, because deformation through adsorbed water layers would not be expected to generate any "noise".

It has been pointed out, however, that ice emits an acoustic response when deformed, and since adsorbed water is known to have a structure different than that of normal water (but definitely not that of ice), it might be expected that deformation of the adsorbed water layer cause an acoustical response. At best then, the acoustic emission response of cohesive soils suggests some sort of solid-to-solid contact, with either direct mineral-to-mineral contact or contact through adsorbed water layers that behave as a solid.

Pusch (82) suggests that a strongly attached thin film of adsorbed water is primarily responsible for the development of shearing resistance in clays. He notes that the pressure required to remove one monomolecular layer of water from a clay may be as about 4000 atmospheres. Lambe and Whitman (48) suggest that contact stresses at adjoining asperities may reach several thousand atmospheres, however Pusch disputes this. Based on his studies of illitic marine clays, he characterizes the primary mode of deformation in these soils as basal plane slip at interparticle domains. Due to the large surface areas of these materials along their basal planes, Pusch feels that interparticle pressures within the domains should be significantly less than several thousand atmospheres, and these smaller interparticle pressures are not sufficiently large to remove the last molecular layers of water. Particle spacings are determined so as to establish equilibrium between long- and short-range attractive and repulsive forces. In an illitic clay at an effective stress of one atmosphere Pusch calculated the average interparticle spacing between particles to be between 50 and 75Å. It must be remembered, however, even on smooth mica flakes, surface asperities larger than 100Å are possible. Thus, it is not clear if Pusch's reasoning for adsorbed water contacts is entirely acceptable.

Resendiz (92), based on his studies using rate process theory, concluded that "a well defined layer of solid-like water with low yield stress surrounds every clay particle". Based on this yield stress he calculated an area of real contact between clay particles of $.18 \text{ cm}^2/\text{cm}^2$ for a test he performed at 150 psi. However, as pointed out by Scott and Ko (97), this contact area is extremely high, so high in fact that it casts doubt on the interpretation of rate process theory employed by Resendiz.

Clearly, the nature of interparticle contacts remains embroiled in some amount of controversy. The evidence presented seems to support mineral-to-mineral contacts more strongly than adsorbed water contacts, although this is not indisputable. What seems likely is that in reality there are both types of contacts and that under certain conditions one or the other type will predominate. Thus, adsorbed water contacts will predominate in clay-water-slurries (6, 60) and possibly in a soil such as a sodium montmorillonite. Mineral-to-mineral contacts will probably predominate in sands and in most kaolinitic and illitic clays. Even in these latter soils adsorbed water or some other contaminant film might make a contribution to shearing resistance in a manner similar to that illustrated in Figure 15.

The view that both mineral-to-mineral contacts and adsorbed water contacts are possible under some conditions but that mineral-to-mineral contacts predominate in most circumstances seems to be consistent with the activation energies measured by various researchers. Matsui and Ito (60) and Andersland and Douglas (6) have concluded that the flow mechanism in dilute clay suspensions is similar to that of a Newtonian fluid. Activation energies measured under these conditions were similar to those for the viscous flow of water (4 to 5 kcal/mole). In these dilute clay suspensions no mineral-to-mineral contact is likely. Matsui and Ito have suggested that as the water content of a clay suspension is decreased through consolidation, a transition region is reached in which both solid-to-solid and adsorbed water contacts contribute to the shearing resistance. In the transition region, as the water content is decreased the percentage of solid-to-solid contacts increases. This is reflected in a variation in the measured activation energy. It was found that in the transition region the activation energy increased with decreasing water content. At some water content solid-to-solid contacts presumably predominate and further decreases in water content caused no change in the measured activation energy.

In the development of the soil behavior model herein, it is assumed that the dominant contact type is mineral-to-mineral. Contact takes place at adjacent asperities which are pushed together with enough force to disrupt the adsorbed water layer.

Particle bonding is assumed to take place at interparticle contacts. The surfaces of silicate minerals are made up of oxygen and silicon atoms and hydroxyl ions. Bonding develops between these different atoms and ions. Due to the similarity of the polar water molecule and the surfaces of the silicate minerals, some water may take part in the bonding (Trollope, 1964), forming a solid structure in conjunction with the mineral surfaces at the asperities. Assuming that a water molecule can take part in forming a bond at an interparticle contact is not the same as assuming that bonding is through adsorbed water layers. It is assumed that contacts can have variable strengths due to the presence of a variable number of bonds. Each bond is assumed to have the same strength.

Shear displacements at interparticle contacts take place through the movement of flow units over energy barriers, as discussed previously in the section of the report on rate process theory. The flow units consist of atoms. This is consistent with Rosenqvist's concept that creep movements result from the diffusion of oxygen atoms in and around interparticle contacts. The energy barriers, which restrict the movement of the flow units, are caused by the interparticle bonds. As pointed out earlier, shearing stresses tend to reduce the height of the energy barriers in the direction of the shear stress and increase the height of the barriers in the opposite direction. The equation governing the rate of shear strain due to a certain deviator stress and temperature has been given as

$$\dot{\epsilon} = X \frac{KT}{h} \exp \left[\frac{-\Delta F}{RT} \right] \exp \left[\frac{\sigma_D \lambda}{4SKT} \right] \quad (13)$$

The terms in this equation have been defined previously. The importance of the term S, the number of bonds, and X, a factor which is an unknown function of time, structure, and the component of displacement due to the surmounting of an energy barrier by a single flow unit, will be discussed here.

Methods for obtaining the number of bonds, S, experimentally have been discussed previously. Typical results from these experiments are given in Figures 13 and 14. These figures illustrate the finding that for normally consolidated clays the number of bonds is proportional to the effective consolidation pressure. When a plot is made of the shear strength of these materials versus the number of bonds, it is seen that the shear strength is proportional to the number of bonds. In fact, Matsui and Ito (60) present the results from a number of studies on normally consolidated and overconsolidated clays, sands and clay pastes which indicate that the shear strength of any soil is directly proportional to the number of bonds. Their findings are summarized in Figure 34. Mitchell et al.'s (73) test results show that both the effective stress and strength are proportional to the number of bonds, which provides a basis for the Mohr-Coulomb effective stress failure criterion.

These findings are entirely compatible with the adhesion theory of friction, which states that the frictional resistance is directly proportional to the area of real contact which, in turn, is directly proportional to the applied normal stress. Based on this model a mechanistic picture of what takes place as two clay particles approach each other can be developed. As the two particles are pressed together due to interparticle normal forces, the area of real contact at the interparticle contact will grow approximately proportionally to the increase in the normal force. Variations from an exact proportionality may exist if the contact is elastic, but as Archard's (7) analysis showed, this variation should be small. Since the surface structure of the clay minerals is reasonably uniform with

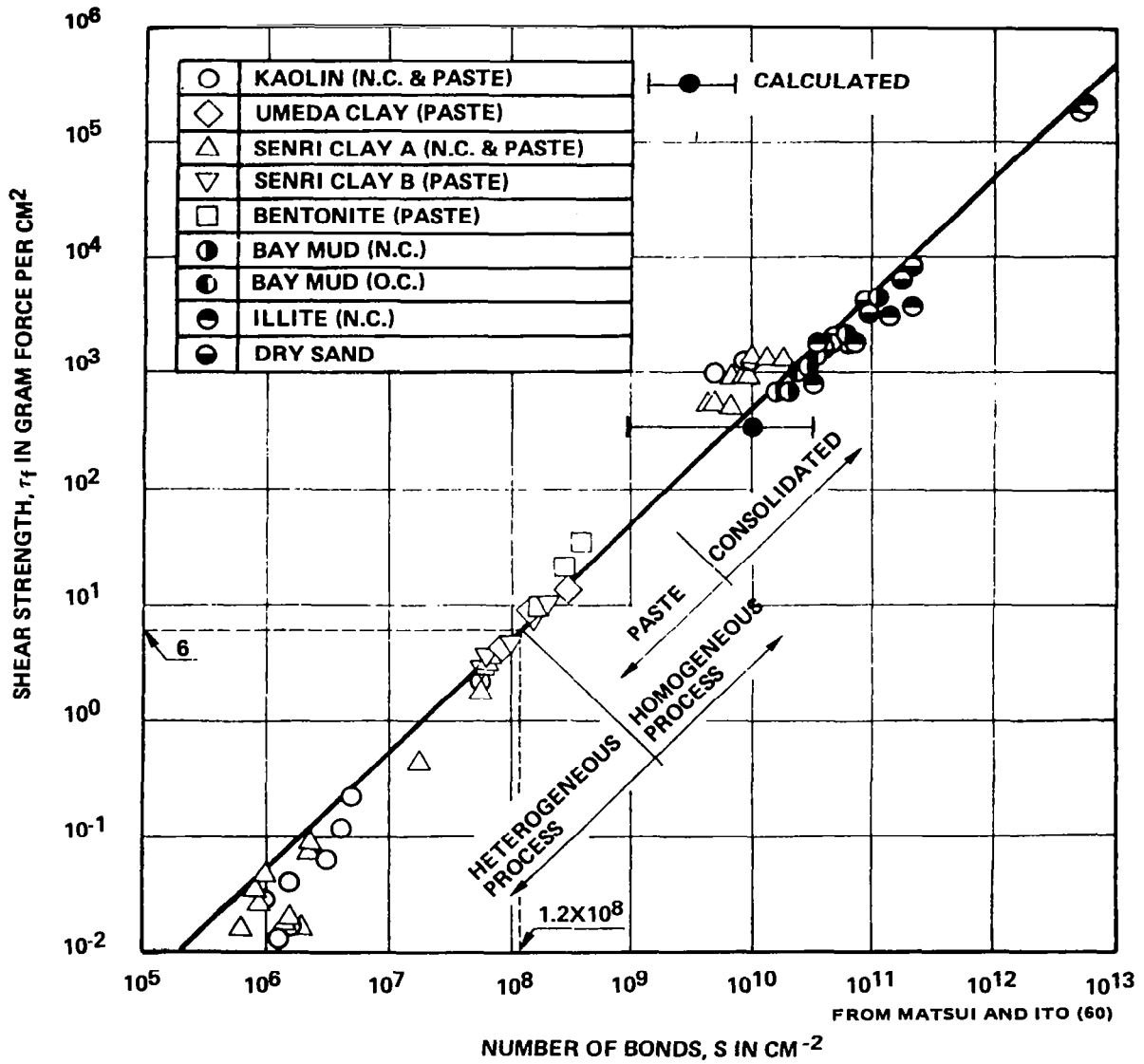


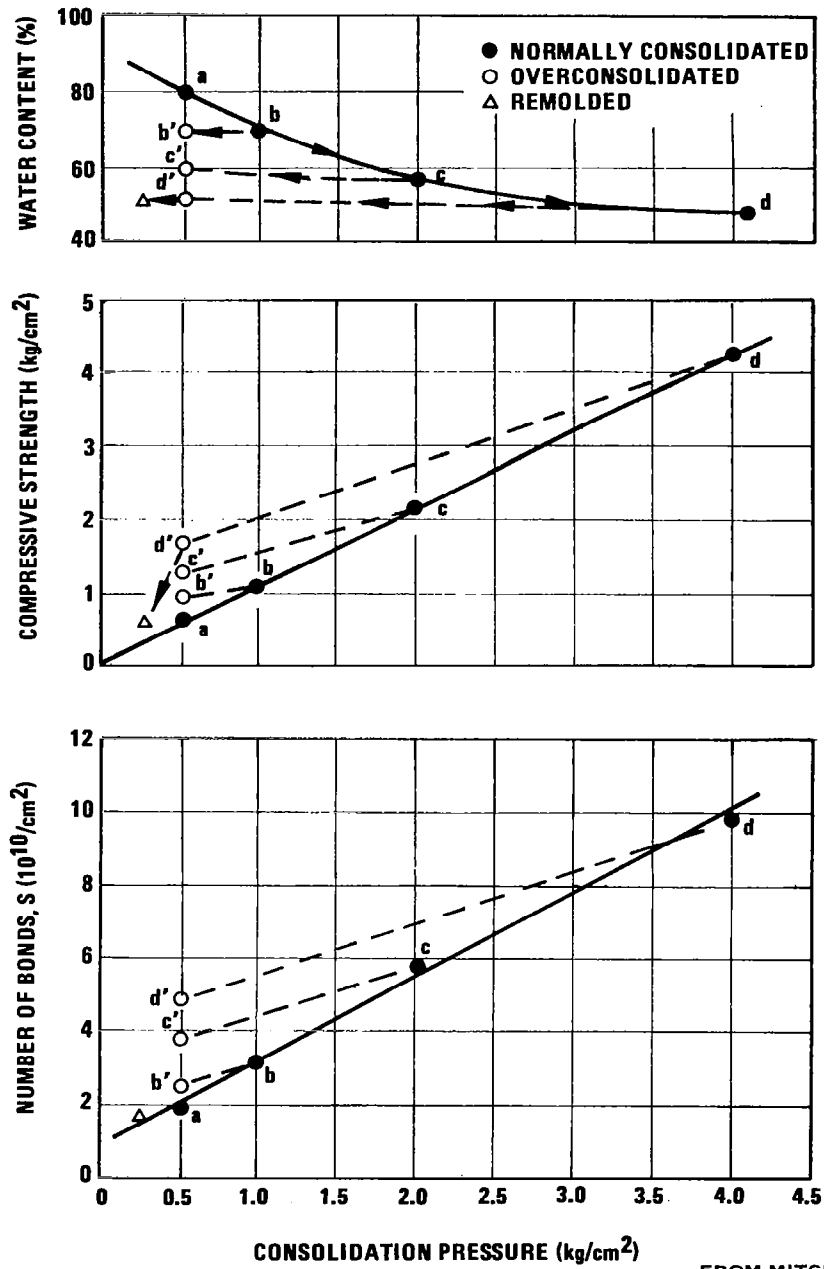
Figure 34. Relationship between shear strength and number of bonds.

lateral extent, the number of bonds per unit contact area should be about constant, as should the bond strength. Thus, as the real area of contact grows, the strength of the contact should grow proportionally. Interparticle normal forces can be caused by applied stresses, long- and short-range attractive and repulsive forces and kinematic constraints. That the same proportionality exists between the shear strength and the number of bonds for a number of different soils is attributed to the similarity in surface structure and bond type in the various soils. Differences in the effective stress strength envelopes for different soils can be thought of as being due to differences in the number of bonds formed at any given value of effective stress.

The effect of overconsolidation on soils can be understood in terms of the effect of overconsolidation on the number of bonds. Figure 35 illustrates the results of tests on San Francisco Bay Mud. It can be seen that at a given effective stress the strength and number of bonds of the overconsolidated samples are larger than the strength and number of bonds of the normally consolidated sample. The sample with the largest overconsolidation ratio exhibits the largest strength and the greatest number of bonds. Apparently, overconsolidation causes some amount of rebound, due to the release of elastic stresses and interparticle repulsive forces. The rebound causes some of the bonds to rupture, but not as many as would be required to give only as many remaining as would exist for a normally consolidated soil at the same effective stress.

Recently Matsui et al. (62) presented evidence that the number of bonds is not constant, but increases with increasing shear strains at small values of strain. At strains greater than 1 to 2 percent they found the number of bonds to be approximately constant. In addition, they found that at a given shear stress, the shear force acting on each bond is mobilized with increasing shear strain. These relationships are illustrated in Figure 36 and Figure 37. The fact that the bond resistances are only fully developed after considerable shear straining might explain, in part, the time dependence of strain rate during creep. Strain rates immediately after load application are relatively large, because only a portion of the bonds that will be stressed at failure will be stressed initially and because the bonds that are active require significant straining before they can fully mobilize their bond strengths. After some amount of straining the strain rate decreases due to an increased number of operative bonds and an increase in the resistance to shear mobilized by each bond. In equation 13 no time dependence is found for strain rate. In addition, X is assumed to be constant. Matsui et al.'s findings may suggest, however, that X is a function of shear strain (structure). As the shear strain increases, the number of bonds and the mobilized strength of the bonds would increase and so would X . In this way the time dependence of strain rate could be accounted for.

Pusch (84, 85, 87) has presented a stochastic rate process model to describe the deformation of soils. In Pusch's model there exist different types of interparticle contacts, each of which exhibits a different energy barrier height. Thus, there is a spectrum of energy barrier



FROM MITCHELL (71)

Figure 35. Consolidation pressure, strength, and bond numbers for San Francisco Bay Mud.

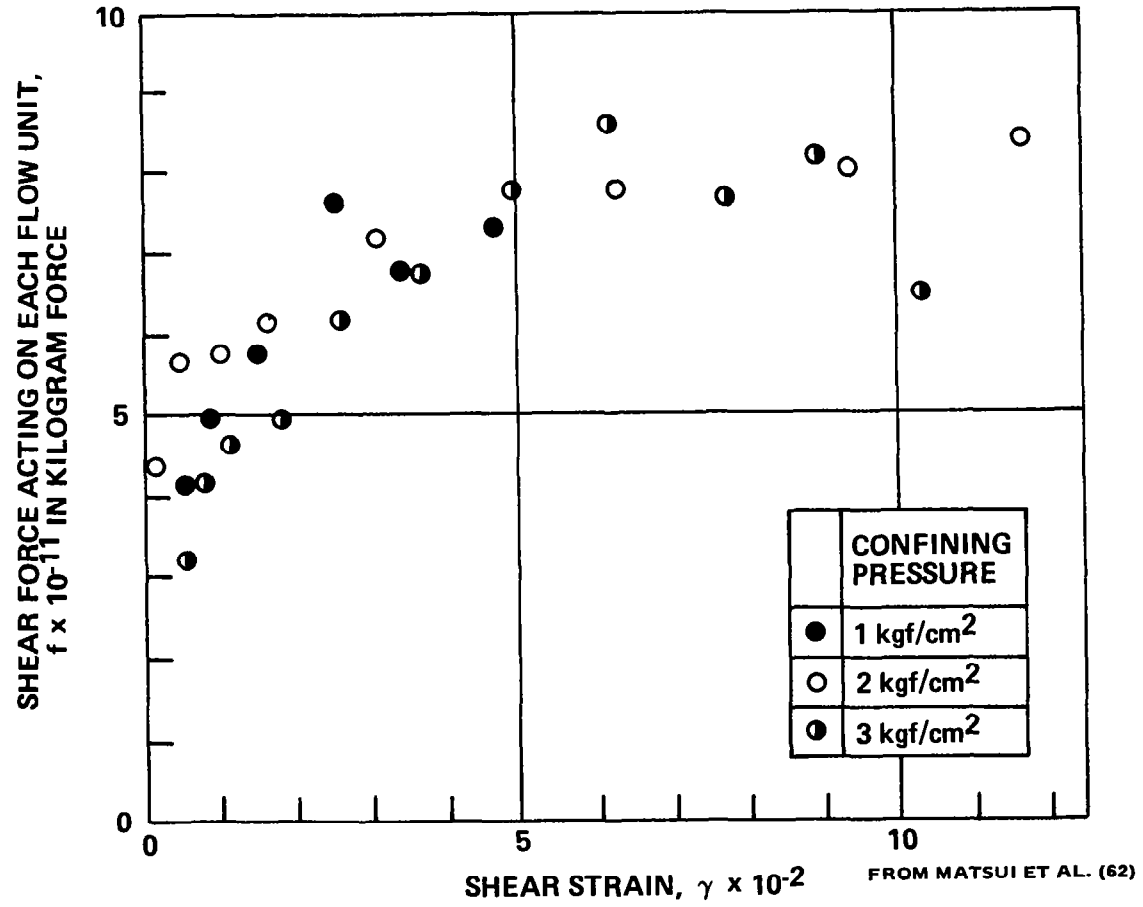


Figure 36. Increase in shear force per flow unit with increasing shear strain.

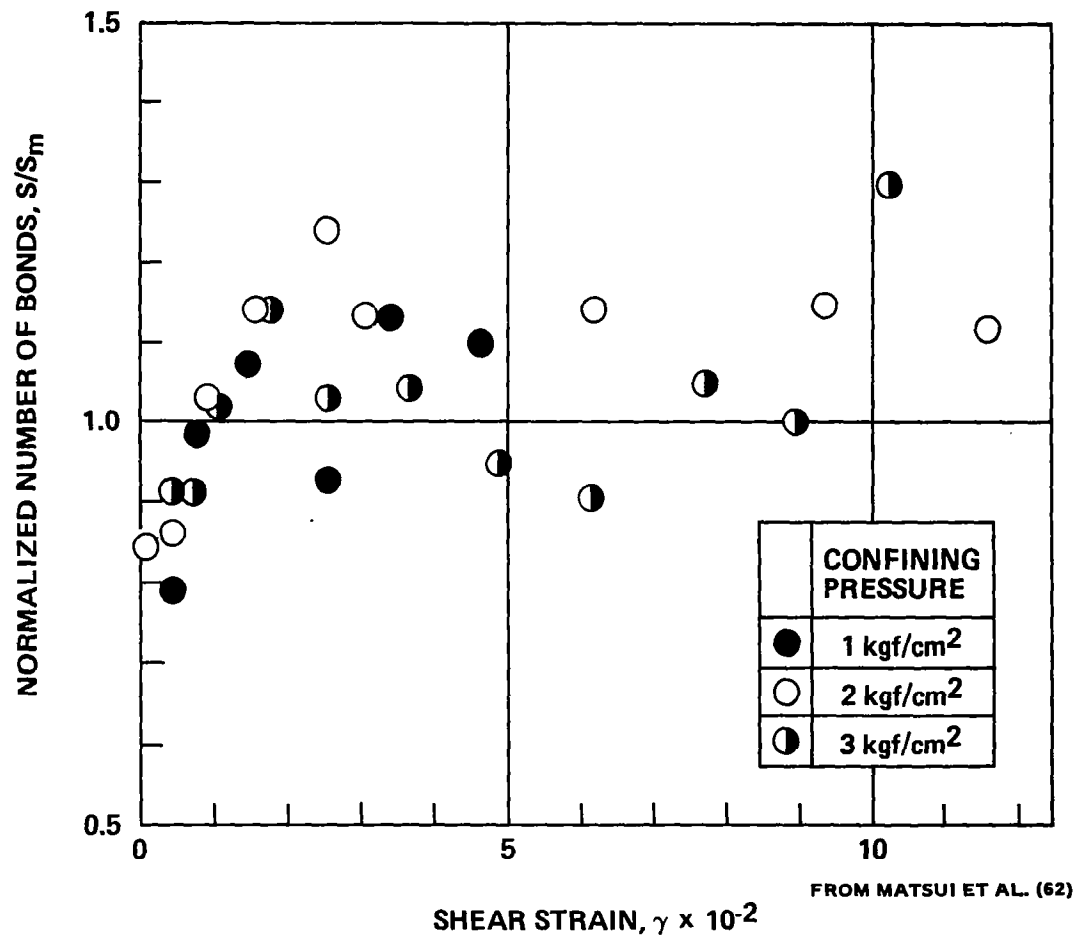


Figure 37. Increase in number of bonds with increasing shear strain.

heights dependent on the number of each different type of contact. This spectrum evolves with increasing creep duration, with the spectrum shifting towards an increased number of high energy barriers with increasing time. The increased number of high energy barriers leads to a decrease in strain rate with time.

The stochastic approach seems to be a fruitful way to describe the evolution of creep. However, there seems little evidence in support of the assumption about the distribution of energy barrier heights made by Pusch. The more usual approach taken to explain the variable contact strength is that there are a variable number of bonds at the contact and this number depends on the force transmitted at the contact. This approach seems more reasonable in light of the surface structure of the clay minerals.

It has been pointed out that the area of real contact at an interparticle contact is directly proportional to the normal force acting between particles. There exist several contributions to this normal force (47, 62). These include the applied mean total stress, the applied deviator stress, pressures in the pore air and pore water, normal forces arising from dilatancy and previous stress history, electrostatic and electrodynamic attractive forces, repulsive forces due to double-layer interactions, surface and ion hydrations and Born repulsions, and water suction in partially saturated soils.

At stress levels normally encountered in engineering practice, the largest contributions to the interparticle forces are due to the applied stresses, pore pressures, dilatancy and stress history. At very low stresses, however, the other contributions to the interparticle forces may become important. If this is the case, the classical effective stress equation,

$$\sigma = \sigma' + u_w \quad , \quad (27)$$

may not be entirely appropriate, and one of the revised forms of the effective stress law (25, 47, 62, 100) may be more appropriate. As an example, the revised effective stress law presented by Mitchell (71) is given below.

$$\sigma = \sigma'_i + u_w + R - A \quad . \quad (28)$$

In the above equations, σ is the applied total stress, σ'_i is the true intergranular stress, u_w is the pore water pressure, and R and A are the electrical repulsive and attractive stresses, respectively. Recently a detailed analysis of a number of the revised forms of the effective stress equation has been made by Matsui et al. (62).

Several of the contributions to interparticle normal forces may exist in the absence of externally applied stresses. These include contributions from dilatancy, previous stress history, electrostatic and electromagnetic attractions, short-range bonding as well as various repulsive forces. It is these forces that are responsible for interparticle bonding in the absence of externally applied stresses. Bonds that might exist in the absence of externally applied stresses are responsible for true cohesion and true tensile strength in soils.

Clearly, there is no single source of true cohesion in soils. Rather, cohesion depends on the interparticle normal forces that develop in response to the above mentioned factors (45). In addition, the cohesion a soil possesses is not constant but is instead a function of any variable that alters these factors. If the change in some variable (e.g., pore fluid electrolyte concentration) increases the attractive forces between particles, the true cohesion will be increased due to an increased number of bonds or, alternatively, if the change in a variable decreases the attractive forces between particles, the true cohesion will be decreased due to a decreased number of bonds.

In summary, the salient features of the comprehensive hypothesis for strength generation in cohesive soils presented herein include:

1. Shearing resistance in soils is developed at interparticle contacts.
2. In soils mineral-to-mineral and adsorbed water contacts are possible. Adsorbed water contacts probably predominate in clay-water suspensions, whereas mineral-to-mineral contacts probably predominate in most consolidated clays.
3. Bonding takes place at interparticle contacts. This bonding develops between the oxygen, silicon, and hydroxyl ions that make up the clay particle surfaces. Some adsorbed water may participate in this bonding.
4. The shear strength of cohesive soils is a macroscopic manifestation of the shear strength of interparticle bonds.
5. The shear strength per bond is approximately constant. However, the number of bonds per contact is variable and, therefore, the contact strength is variable.
6. Shear displacements at contacts involve the flow of atoms on adjacent clay particle surfaces. This flow process can be described using rate process theory.
7. Contact strength is proportional to the number of bonds comprising the contact. The number of bonds is proportional to the normal force transmitted at the contact.

8. These findings are consistent with the adhesion theory of friction which postulates that the frictional resistance of a contact is proportional to the real area of contact, which in turn is proportional to the normal force being transmitted at the contact.

9. At a given consolidation pressure an overconsolidated soil has more bonds than a normally consolidated soil and is thus stronger.

10. Bond strength is mobilized with strain.

11. There are several contributions to the normal force acting between two clay particles.

12. Normal forces between particles may exist in the absence of externally applied stresses. These forces account for true cohesion in cohesive soils.

13. There is no single true cohesion or true angle of internal friction in cohesive soils. These quantities are complex functions of many compositional and environmental variables.

V. RECOMMENDATIONS FOR IN-SPACE AND TERRESTRIAL LABORATORY TESTING

Introduction

The review of fine-grained cohesive soil behavior, with emphasis on conditions of low effective stress has clearly pointed out the need for further research in several different areas. Questions still remain concerning the nature of the clay-water-electrolyte system. The results of previous research, while answering many questions, still leaves several ambiguities and in some instances the results of different studies appear to be contradictory. As an example of a research area in which these sorts of problems have occurred, we need only look at studies concerned with the nature of adsorbed water or studies designed to investigate the nature of interparticle contacts.

Specific questions concerning the behavior of cohesive soils that need resolution include:

1. Is there a true cohesion generated by interparticle forces in the absence of interparticle cementation? What interparticle forces are primarily responsible for this cohesive? What factors affect its magnitude?
2. Do cohesive soils exhibit a true tensile strength?
3. What is the shape of the failure envelope in the region of the stress origin?
4. What type of stress-strain-time response do cohesive soils exhibit in the region of the stress origin?
5. What is the exact role of surface films and adsorbed water layers in the generation and mobilization of shearing resistance?
6. Under what conditions are interparticle contacts mineral to mineral? When are they through adsorbed water films?
7. Is the fundamental hypothesis that has been proposed previously well supported experimentally?

In our opinion, an experimental research program incorporating both terrestrial laboratory testing and in-space testing in the zero-gravity, high vacuum environment of the NASA Spacelab could provide information that would contribute significantly to an understanding of cohesive soil behavior.

In our opinion, the testing procedures and apparatus used aboard Spacelab should be of the simplest types possible. The use of very complex experiments in the in-space environment should be reserved until the time when the ability to perform relatively simple experiments in space has been well established. In-space experiments should be designed to take full advantage of qualitative descriptions, observations, and comparisons. In light of these considerations, the person who performs the in-space experiments should be familiar with cohesive soil behavior in the terrestrial environment.

It is proposed that the current study be used to investigate the following items:

1. The nature of true cohesion.
2. The nature of true tensile strength.
3. The effect of the earth's gravitational field on the intrinsic mechanical properties of fine-grained cohesive soils.

The first two test recommendations involve only relatively simple testing procedures and apparatus. A study of the intrinsic mechanical properties of cohesive soils will require fairly sophisticated testing procedures and apparatus. These tests have been recommended, in spite of their complexity, based on the availability of a true triaxial testing apparatus specially designed for the Spacelab environment (104). Investigations into (1) the properties of the adsorbed water that surrounds clay particles, (2) the development of friction between mineral surfaces, and (3) the possible existence of true cohesion in normally consolidated soils have not been recommended as part of this study. These investigations would require sophisticated testing techniques and it is thought that they are best reserved until the ability to perform relatively simple tests in space is well established.

Measurement of True Cohesion

Justification. Direct measurement of soil cohesion in the absence of normal stress is not possible on earth due to the presence of gravitational body forces. The unique environment of the NASA Spacelab offers the ideal conditions in which to investigate the nature of true cohesion. The zero gravity conditions of space will allow, for the first time, testing of fine-grained cohesive soils at the zero effective stress level. Only at zero effective stress can true cohesion be unambiguously determined. Extrapolation of true cohesion from tests that are not performed at zero effective stress is highly uncertain due to uncertainty in the shape of the failure envelope in the vicinity of the stress origin.

It is suggested that true cohesion measurements be made on at least two different soils, possibly an illite and a kaolinite. In addition, it is recommended that the role of physico-chemical variables and stress history in the development of true cohesion be investigated. Significant physico-chemical variables include type of pore fluid, type of adsorbed cation, pore fluid electrolyte concentration and pH. The primary stress history variable is the maximum past consolidation pressure.

The results of tests designed to measure true cohesion will be of both practical and scientific interest. On the practical side, finite element analyses of deformation-induced cracking in earth fill dams by Lefebvre and Duncan (51), Eisenstein et al. (22) and Strohm and Johnson (101) have illustrated the importance of cohesive soil behavior at low stress levels and in the tensile stress region in attempting to predict potential zones of cracking in earth dams. Similarly, studies of hydraulic fracturing in earth dams (32, 79) have been concerned with the behavior of cohesive soils under these same conditions.

An increased understanding of the conditions of failure at low effective stress levels and in the tensile region will be useful to researchers in the field of the constitutive modeling of cohesive soil behavior. One of the chief concerns of those who study constitutive modeling is defining the shape of the yield criteria. Very little data presently exist on the shape of the yield loci in the region of very low confining stresses. The proposed Spacelab experiments will provide useful data with which the yield loci may be better defined. Data obtained from the experiments will also be useful in developing plastic potentials and work hardening rules.

From a scientific point of view there are several benefits to be gained through the proposed experimental investigation. These tests will provide the first unequivocal evidence for the existence or absence of a true cohesion. It is our opinion, based on the material reviewed for this project, that true cohesion can exist in cohesive soils in the absence of cementation. However, this point of view is not universally shared. The proposed tests will do much to clarify these differences of opinion.

A second area of scientific interest which will be able to be explored in the proposed investigation is the development of true cohesion with strain. Schmertmann and Osterberg (96) have concluded that the cohesion component of strength develops to its maximum at strains much smaller than those needed to mobilize frictional resistance. In their tests they observed that the cohesion quickly rose to a maximum and then decreased to a significantly smaller value. They termed this effect the "peaking effect". However, as pointed out by Mitchell (71), due to possible curvature in the effective stress failure envelopes and different fabrics in the normally consolidated and overconsolidated samples used in Schmertmann's and Osterberg's analysis, the correctness of their interpretation is uncertain. The NASA Spacelab measurements of true cohesion and the strains or displacements required to develop it should be free from these types of interpretation problems.

Finally, the true cohesion measurements will provide much insight into the role of interparticle forces in the development of shearing resistance at low effective stress levels. Through variation of the type of adsorbed cation, pore fluid electrolyte concentration, pH and other physico-chemical variables, the importance of diffuse double-layer forces, edge charge as well as some of the other interparticle forces will be determinable, at least qualitatively.

In addition to the zero effective stress experiments to be performed in space, it is recommended that a similar set of tests be performed on earth at a very low value of effective stress. Tests similar to the drained unconfined compression tests performed by Ladd and Kinner (45) would be appropriate for this set of experiments. The results of the terrestrially performed tests could be compared with the results of the tests performed in space. Comparison of the test results will clarify the importance of small effective stress levels when trying to ascertain the magnitude of true cohesion.

Perhaps it will be found that drained unconfined compression tests on earth give good approximations to the true cohesion measured in space. Perhaps it will be found that the two sets of tests give entirely different results and that terrestrially based tests do not provide good approximations to the true cohesion. The two sets of tests, the in-space true cohesion tests and the terrestrially based drained unconfined compression tests will provide information on the curvature of the Mohr-Coulomb failure envelope in the region of the stress origin. For comparative purposes it is also recommended that a limited number of drained unconfined compression tests be performed in-space. These tests could be compared directly to the terrestrially performed tests to obtain information on the role of body forces in determining the behavior of cohesive soils at low effective stress levels.

It is suggested that the same soils and the same environmental conditions be used for both the in-space tests and the terrestrially based tests. In this way direct comparison of the two sets of test results will be possible, and the effects of small effective stresses on the measured values of true cohesion will be most easily ascertained.

Method of testing. The only types of test that can maintain zero effective stress on the failure plane at failure are the direct shear test and the torsional or ring shear test. No configuration of the conventional triaxial test can meet this objective. The direct shear test is being recommended for the study of true cohesion in-space because of its relative simplicity. The advantages of the direct shear apparatus include:

1. In a zero gravity environment tests can be performed with zero effective stress on the failure plane at failure.
2. Relatively rapid drainage can be achieved through the use of thin samples. This leads to shorter testing times, a definite advantage in the Spacelab environment.
3. Good accuracy can be obtained at low effective stress levels since there is no need for filter paper drains or rubber membranes.
4. The tests can be run to large displacements. Thus a peak cohesion and a residual cohesion can be measured.

5. The test uses K_0 consolidation and simulates plane strain conditions.

6. Apparatus is relatively simple.

There are several disadvantages associated with the direct shear test. These include:

1. Non-uniform stresses and strains within the sample. The non-uniform strain field precludes the measurement of stress-strain data. Instead, stress-shear displacement measurements must be made.

2. Stress concentrations near the ends of the direct shear test specimens cause the ends of the samples to reach peak stress before the middle.

3. Some friction develops in the loading system due to interaction of the moving parts.

The last disadvantage is thought to be the most serious at small loads. In space, however, the weights of the moving parts will essentially be zero, and the friction developed between the moving parts will be greatly reduced. With careful machining of parts and proper calibration of the system, it is thought that machine friction can be kept within tolerable limits.

Sample Preparation. In order to minimize the amount of time required aboard the NASA Spacelab it is suggested that all samples be initially prepared on earth. Initial preparation would involve the mixing of dry fractionated soil with water to form a slurry. The slurry would then be consolidated one dimensionally to a pressure below the maximum consolidation pressure to which the soil would eventually be subjected. Care must be taken during the early stages of sample preparation to ensure that the soil remains saturated. After the initial consolidation, the soil would be stored until needed for testing.

Some of the stored samples would be used for the terrestrial tests, and some would be taken into space. In the case of the samples to be tested terrestrially, the stored samples would be taken and trimmed as needed. Following trimming, each sample would be consolidated one dimensionally in a triaxial cell to its maximum consolidation pressure. It would then be rebounded, weighed, measured and placed in a soaking solution. The soaking solution should be so designed as to give each sample the desired pore fluid chemistry. The time required for each sample to obtain the same pore fluid chemistry as the soaking solution will have to be determined as part of the NASA Spacelab study. As a guide, Ladd and Kinner (45) soaked the triaxial compression samples used in their studies for between 1 and 49 days.

The samples that are brought into space for testing would also need to be trimmed and placed into a mold for one-dimensional consolidation to their maximum consolidation pressures. After consolidation, the

samples would have to be rebounded, soaked in the appropriate soaking solution, placed in the direct shear apparatus and tested. Soaking times for the direct shear test samples should be substantially less than the soaking times needed for the drained unconfined compression test samples due to the fact that much thinner samples can be used in the direct shear tests. Both types of tests should be designed so that the soil samples can remain submerged in their appropriate soaking solutions during testing.

Equipment and Instrumentation Requirements. In that the aim of the Phase 1 proposal is to develop preliminary concepts for Spacelab experiments, only very general equipment requirements will be outlined here.

In order to make measurements of true cohesion in space a direct shear apparatus will be required. Direct shear testing machines are available from most manufacturers of soil mechanics laboratory equipment. The unique nature of the NASA Spacelab tests may require substantial modifications to any standard direct shear apparatus, or it may even require the design and machining of an apparatus specially suited for the Spacelab testing program.

The direct shear testing machine used in this study must be suitable for applying small loads accurately, maintaining constant shear rates, and be machined to tolerances that provide a minimum of friction. The top half of the shear box must be designed to move in such a way that there is no tendency to tip or rotate with respect to the bottom half of the shear box. In addition, the insides of the shear box must be designed so as to minimize the amount of friction that can develop between the soil sample and the walls of the shear box. One way to minimize this friction is to construct the inside of the shear box out of a thin layer of highly polished teflon.

The direct shear apparatus must be designed so as to allow the shear box to remain submerged for the duration of the test. Load cells fitted to the direct shear device should be capable of measuring horizontal shear stresses in the range of 0 to 200 grams/cm² accurately. It is desirable to measure both the horizontal and vertical deflections of the sample during the test. Horizontally and vertically mounted linear variable differential transformers (LVDT) would be suitable for making such measurements. A data acquisition system will be required to record the output from the load measuring and displacement measuring devices.

In order to consolidate the direct shear test samples to their maximum consolidation pressure a one-dimensional consolidometer will be needed for use aboard the NASA Spacelab. The consolidometer should be designed for use with soil samples that have the same size as can be accommodated by the direct shear apparatus. Provisions will also need to be made for storing, trimming, weighing and drying samples aboard the Spacelab. The pH of the soaking solution should be monitored using a pH meter, while the pore fluid electrolyte concentration can be determined by measuring the electrical conductivity of the soaking solution.

The drained unconfined compression tests could be carried out using a technique similar to the one used by Ladd and Kinner (45). They used a guided piston loading system to perform drained, stress-controlled tests, as shown in Figure 38. This technique was adopted because simpler techniques led to problems with eccentricity which developed as the sample deformed. Eccentricity tended to cause the samples to fail prematurely. For the in-space experiments a method for applying the vertical load that does not depend on the application of dead weight would need to be devised.

GUIDED PISTON LOADING SYSTEM

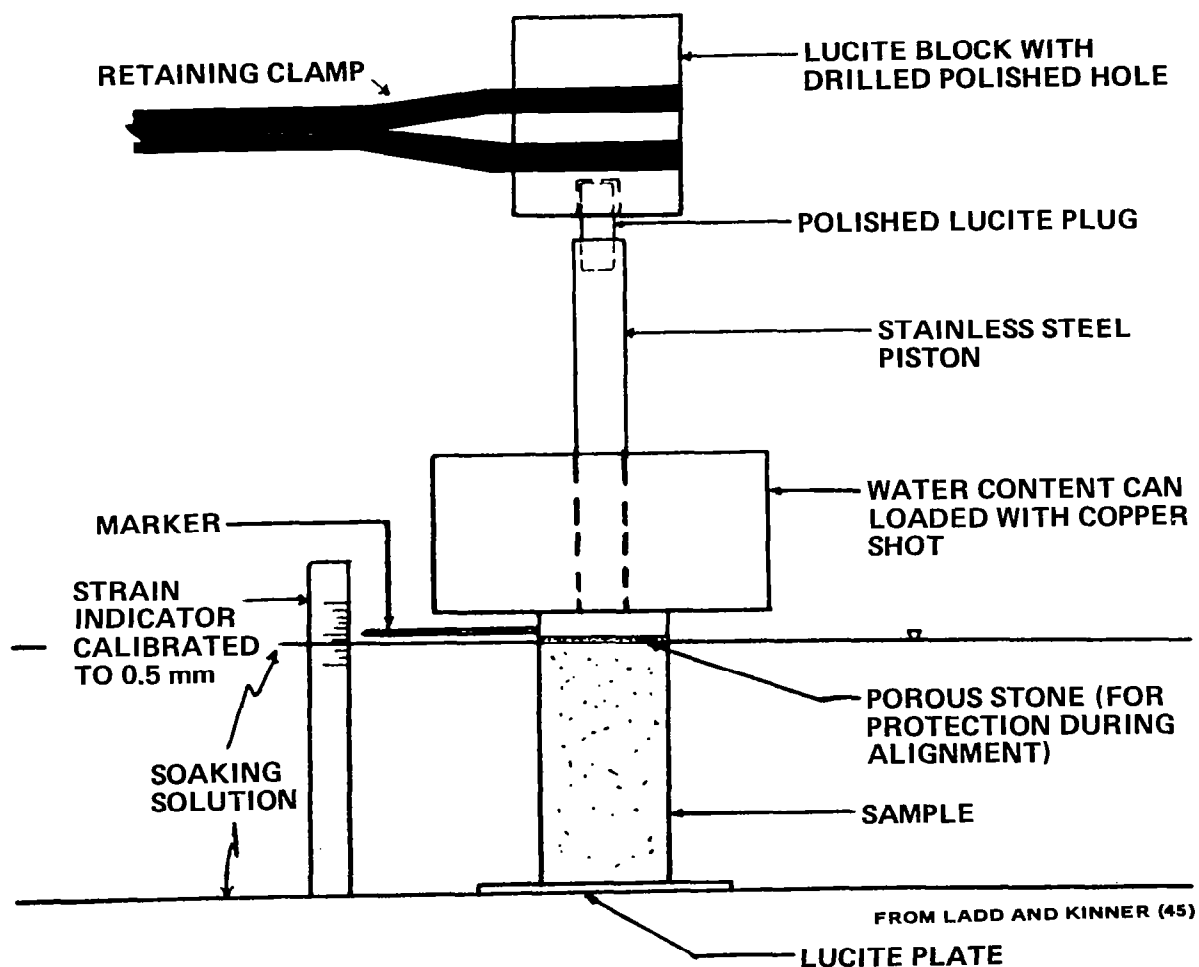


Figure 38. Drained unconfined compression test set-up.

In addition to an apparatus similar to that shown in Figure 38, both the terrestrial and in-space laboratories would also need to be equipped with a standard triaxial cell in which the soil specimens could be subjected to their maximum consolidation pressures, as well as the appurtenant equipment needed to apply and record consolidation pressures, volume changes and axial deformations. As with the Spacelab experiments, provisions will also be required in the terrestrial laboratory to store, trim, weigh and dry the soil samples as well as to control and measure the soaking solution pH and electrolyte concentration.

Test Variables. Test variables that are germane to an experimental study of true cohesion in cohesive soils includes:

1. The normal stress acting on the failure plane.
2. The shear stress acting on the failure plane.
3. The horizontal displacement of the sample.
4. The vertical displacement of the sample.
5. The strain rate at which the test is performed.
6. The maximum consolidation pressure.
7. The physico-chemical variables of the pore fluid.
 - a. The type of pore fluid.
 - b. The pore fluid electrolyte concentration.
 - c. The type of exchangeable cation.
 - d. The pore fluid pH.

Measurement of True Tensile Strength

Justification. There exists a wide variety of earth structures that may be subjected to tensile stresses and strains. As was pointed out previously, there have been several studies which have demonstrated the importance of the tensile strength properties of a soil comprising a dam in determining the zones of cracking which develop in that dam. Similarly, analyses of hydraulic fracturing problems have shown that a soil's tensile strength plays a role in determining the extent of fracturing. The tensile properties of soils are also important when attempting to determine the extent of tension cracks in slope stability studies, in determining cracking due to subsidence, and in the analysis of the fatigue behavior of pavements. Recently, some attention has been focused on the problem of cracking of clay liners surrounding low-level radioactive waste disposal sites.

In light of the many ways in which the tensile strength of soils plays a role in geotechnical engineering, there have been relatively few studies concerned with the tensile strength of cohesive soils. These studies, while useful in their limited applications, do not provide a great deal of insight into the development of true tensile strength in soils for the following three reasons:

1. Several of the studies (10, 18) have been concerned with the behavior of cemented soils. In these soils the strengths of the cementation bonds control the tensile behavior of the soil and very little is learned about the possible fundamental mechanisms that might contribute to a soil's true tensile strength.

2. Most of the studies (1, 2, 37, 38, 52, 78, 91) have been carried out on compacted, partially saturated soils. In these types of studies it has been difficult to separate the contribution that the true tensile strength makes to the measured tensile strength from the contribution made by stresses caused by water suction in the partially saturated soils.

3. Interpretation of the results of the various studies is complicated by the fact that different methods of testing were used in the different studies. Only the direct tension test measures tensile strength directly and requires no special assumptions about a soil's properties (3). Problems arise with the other test methods because each test imposes different stress states, sets up different stress gradients, induces failure on different planes, and often requires special assumptions about the material being tested. For instance, interpretation of the double punch test requires the assumption that the soil is perfectly plastic (23), whereas the indirect tension test and the hollow cylinder test require the assumption that the soil behaves as a linearly elastic material (3).

In our opinion the NASA Spacelab experimental program provides an excellent opportunity to increase our understanding of the tensile behavior of fine-grained cohesive soils. The results of these tests would be of use to researchers interested in earth dam cracking, hydraulic fracturing, slope stability, radioactive and hazardous waste containment, wave-induced submarine landslides (99), the fatigue behavior of pavements and the constitutive modeling of cohesive soils. In addition, the experimental program as envisioned will add significantly to our understanding of the physico-chemical behavior of cohesive soils.

Fortunately, the tensile testing of cohesive soils can be performed in terrestrial laboratories. The zero gravity environment of the NASA Spacelab is not a prerequisite for successful testing. These tests are an entirely appropriate contribution to the present study, however, because a knowledge of the tensile behavior of cohesive soils plays an integral part in the understanding of the behavior of these soils at very low effective stresses. From a practical point of view, the zones in an earth structure that are subjected to tensile stress and strain states get to these states along stress paths that take them through various low effective stress states. In order to be able to predict the behavior of these zones in the earth structure, the soil's behavior under all conditions of very low stress, both compressive and tensile, should be known.

For reasons to be discussed subsequently, we recommend that the direct tension test be used to measure the tensile strength of the test soils. The design proposed by Bishop and Garga (10) has been used successfully to measure the tensile strength of London Clay. Their apparatus is capable of defining a soil's failure envelope within the region of tensile stresses (with certain limitations), as seen in Figure 39. Because of this advantage it is suggested that the Bishop and Garga apparatus be used to measure tensile strengths.

The possibility exists that the true tensile strength is uniquely related to the true cohesion of a soil since many of the same strength mechanisms are active under both conditions. In order to test this hypothesis, it is suggested that the direct tension tests be performed under exactly the same environmental and compositional conditions as used in the direct shear tests designed to measure true cohesion and the drained unconfined compression tests. Together, these three sets of tests will provide comprehensive data on the strength behavior of the cohesive soils tested under conditions of very low compressive and tensile stresses. This data will be used in estimating the location of the soil's failure envelope, as illustrated in Figure 39. Possible relationships between true cohesion and true tensile strength could be investigated, as well as possible relationships between these two quantities and the drained unconfined compression test results. The effect that changes in the physico-chemical variables have on the shape and location of the failure envelope will be of practical interest, as well as providing engineers and scientists valuable information through which they can better understand fine-grained cohesive behavior.

Method of Testing. As noted previously, it is recommended that the direct tension test apparatus designed by Bishop and Garga (10) be used to determine the tensile strengths of the soils tested. The advantage of their apparatus is that no end clamps are needed to apply a tensile stress to the soil specimen. The technique employed by Bishop and Garga uses a sample in which the middle section of the sample has a reduced diameter. The specimen is placed in a triaxial cell and an all around pressure and an axial tensile force, T , are applied. As illustrated by Figure 40, the all around confining pressure and the axial tensile force produce a tensile stress which is given by the equation:

$$\sigma_T = \frac{T}{A_M} - (\sigma_A - \mu) \quad (28)$$

where

σ_T = tensile stress

σ_A = confining pressure

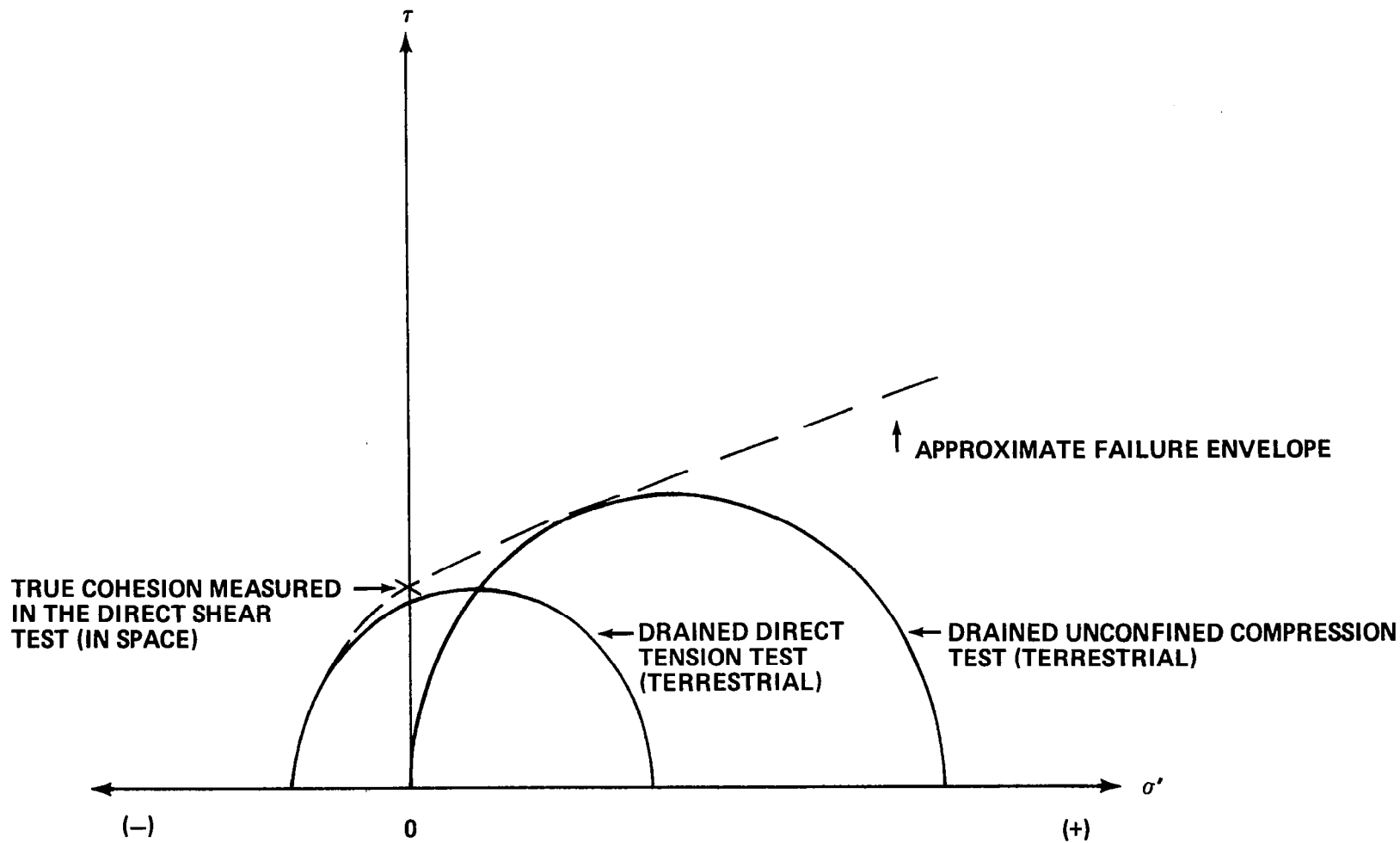


Figure 39. Example of the expected results from the low effective stress strength tests performed in-space and terrestrially.

μ = back pressure in the specimen

A_M = cross sectional area of the specimen at the middle.

Al-Hussaini and Townsend (10) indicate that the tensile strength of clays can be obtained with reasonable accuracy using Bishop and Garga's apparatus. The direct tension apparatus is preferable to most other techniques for determining true tensile strength because no special assumptions about the soil's properties are required in order to determine the tensile strength.

Sample Preparation. The preparation of samples for the direct tension tests will be nearly identical to that of the direct shear tests and the unconfined compression tests. The only modification to the preparation procedure will be in the final trimming of the samples. A specially prepared lathe will be needed to trim the direct tension test samples to their final shape.

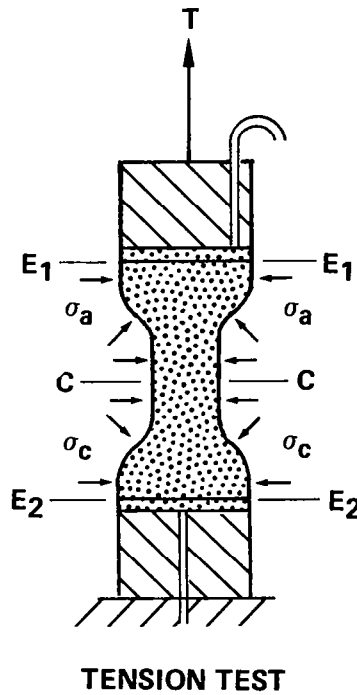


Figure 40. Direct tension test used by Bishop and Garga (40). The soil specimen is placed in a triaxial cell and the tensile stress is applied through a stainless steel ram clamped to the specimen top cap. By varying the sample geometry and the cell fluid pressure a tensile stress can be maintained across section C-C while a compressive stress is maintained across sections E₁ - E₁ and E₂ - E₂.

Equipment and Instrumentation Requirements. A standard triaxial cell, with perhaps minor modification can be used to run the direct tension test. In addition, standard laboratory equipment for measuring volume change, cell pressure and axial deformation and a system for recording the test measurements will be needed. Except for the specially designed lathe, exactly the same laboratory equipment can be used to prepare the test specimens as is used to prepare the drained unconfined compression test specimens. In addition, the same equipment can be used to monitor and control the various physico-chemical test variables as is used in the unconfined compression tests.

Test Variables. Test variables that are germane to the study of the true tensile strength of cohesive soils include:

1. Effective confining pressure acting on the sample.
2. Tensile stress acting on the sample.
3. Axial deformation of the sample.
4. Change in volume of the sample
5. Maximum past consolidation pressure.
6. Physico-chemical environment
 - a. Pore fluid electrolyte concentration
 - b. Type of pore fluid
 - c. Type of exchangeable cation
 - d. Pore fluid pH.

Effect of Gravity on the Stress-Strain Behavior of Cohesive Soils

Justification. It has been amply shown in the geotechnical literature that clay particles tend to become oriented in the direction normal to the direction of the major principal stress during one-dimensional consolidation (36, 39, 40, 55, 58, 59, 66, 76, 88). This preferred particle orientation causes an inherent anisotropy within one dimensionally consolidated clays. This anisotropy of the clay microfabric can lead to anisotropy in the clays' macroscopic engineering properties.

Mitchell (66), for instance, found that anisotropy of fabric leads to an anisotropic permeability. Fabric anisotropy may also lead to anisotropy of a soil's undrained shear strength. This is well illustrated by Figures 41 and 42. Ladd (44) points out that undrained strength anisotropy is controlled by pore pressure development during shear and the effective stress-strength parameters mobilized at failure. Several studies (20, 36, 76) have found that the effective stress-strength parameters are

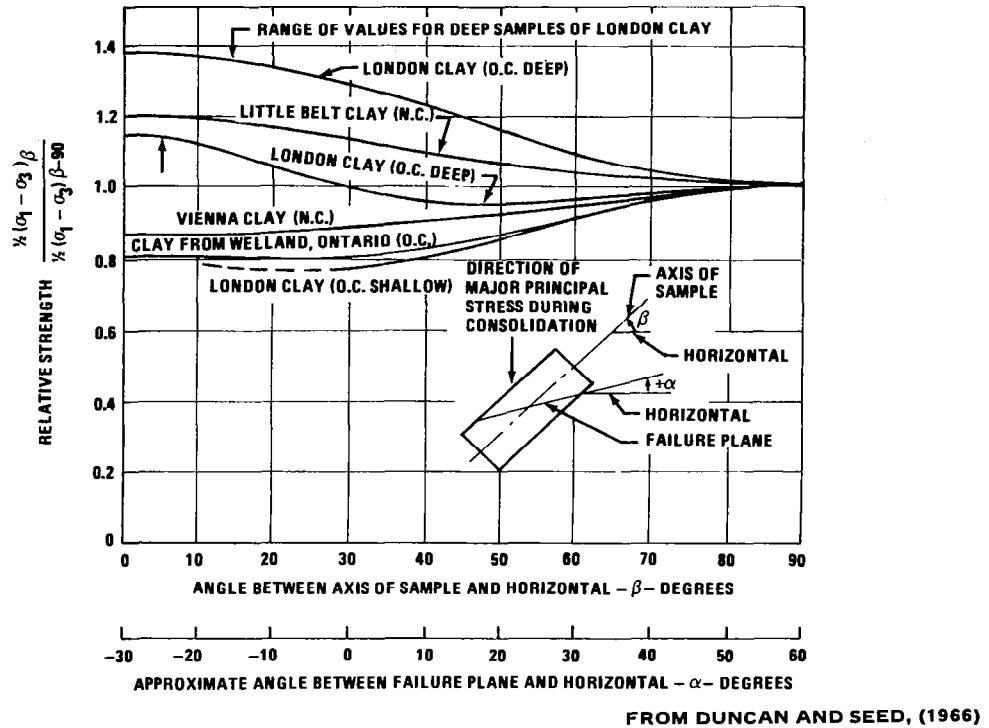


Figure 41. Summary of data concerning the variation of compressive strength with orientation of the failure plane.

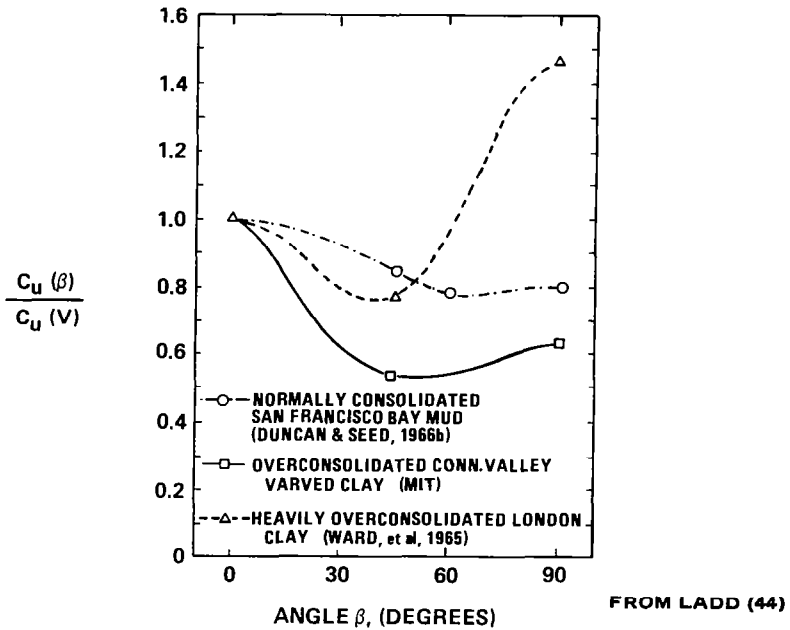


Figure 42. Undrained strength anisotropy of three clays as measured by UU triaxial compression tests.

independent of the orientation of the shear stress relative to that of the major principal consolidation stress. If this is the case, the differences in the pore pressures developed during shear, which is the result of fabric anisotropy.

In addition to strength and permeability anisotropy, anisotropic fabrics also lead to anisotropic deformation behavior. Both Mitchell (66) and Quigley and Thompson (88) have pointed out that the volumetric compressibility of a soil is dependent on the degree of preferred orientation. Ladd (44) has shown that initial secant moduli for Boston Blue Clay are 50 to 100 percent higher in plane strain compression tests than in plane strain extension tests, and many examples could be cited which show that the strain at failure in a given test is, in part, a function of the orientation of the applied stresses. Krizek et al. (40) found that anisotropically consolidated clays exhibited anisotropic creep responses as well as anisotropic undrained strength and deformation properties.

The preceding discussion has clearly illustrated that an anisotropic stress system during consolidation leads to anisotropic soil properties. On the other hand, isotropic consolidation stresses tend to produce random fabrics and isotropic engineering properties. In addition, limited data presented by Martin (58) seem to indicate that an isotropic consolidation stress, when applied to a soil with an anisotropic fabric, tends to cause little change in that fabric, at least for fabrics with relatively low degrees of parallel particle orientation. This last point is an important one in regards to the present study. Its implication is that on earth we can never prepare a truly isotropic, homogenous soil specimen due to the presence of gravitational body forces. In the process of sample preparation gravitational stresses will induce at least a small degree of preferred particle orientation.

There is considerable evidence that even in a slurry a small amount of particle rotation results due to gravitational body forces. Martin (58) found that a kaolinite slurry at a water content of 190 percent had an average particle inclination angle of about 33 degrees. A truly random fabric would theoretically have an average inclination angle of 45 degrees, while a fabric in which there was perfect parallel particle alignment would theoretically have an average inclination angle of 0 degrees. Martin also found that the average angle of inclination changed little after the slurry was placed in a special balloon and consolidated isotropically to a pressure of 1 kg/cm^2 . Martin's experiments were carried out on flocculated samples. It would be expected that the average particle inclination angle would be even lower in a dispersed sample.

Martin and Ladd (59) used X-ray diffraction techniques to estimate the average particle orientation angle in kaolinite prepared by different techniques and consolidated one dimensionally to a number of different confining pressures. Their results are shown in Figure 43. It can be seen that at a consolidation stress of only $.01 \text{ kg/cm}^2$ the average particle

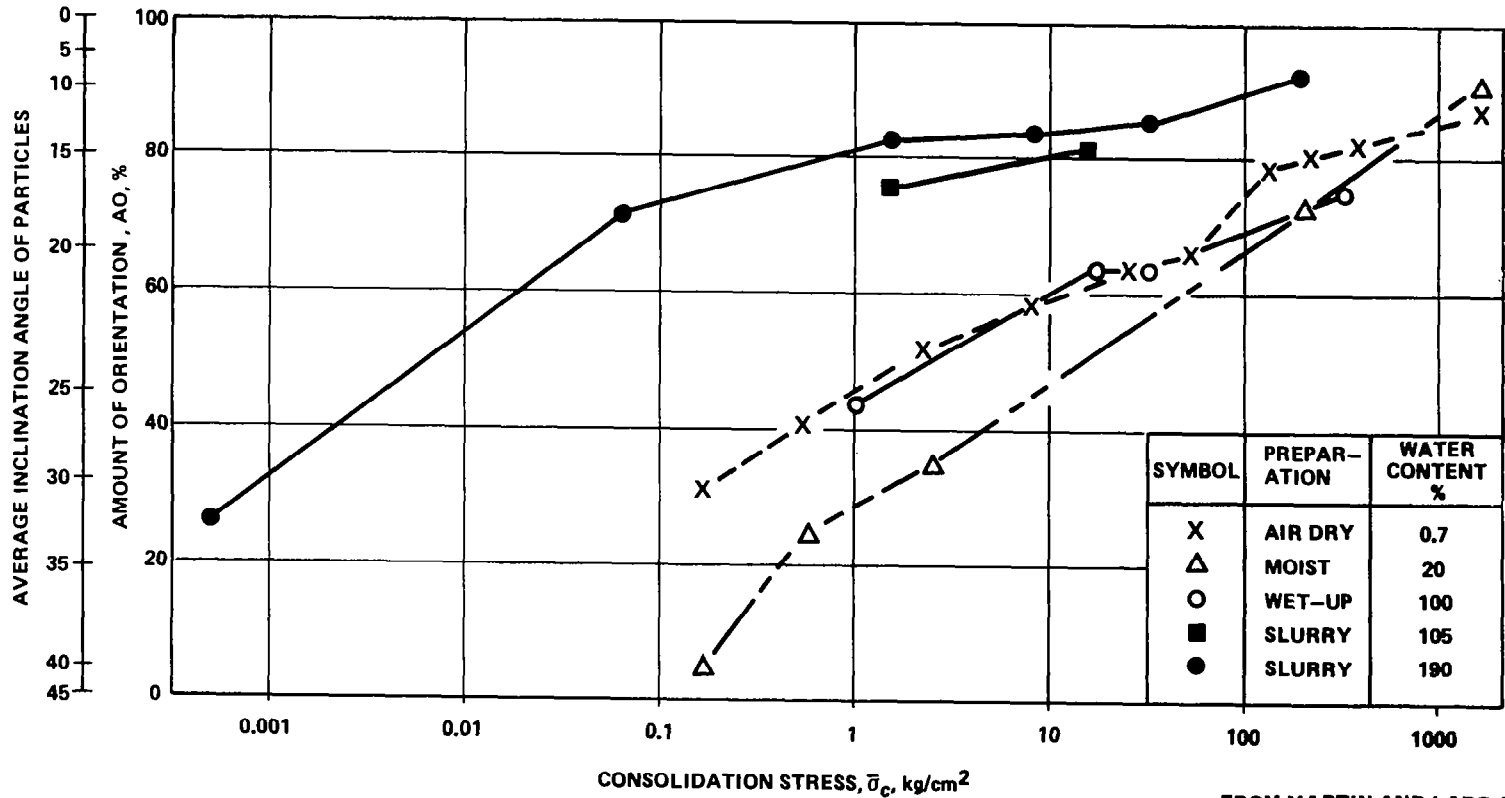


Figure 43. Particle orientation of kaolinite versus consolidation stress for various methods of sample preparation.

FROM MARTIN AND LADD (59)

orientation angle of kaolinite samples prepared from a slurry was measured to be only 22 degrees. This pressure corresponds to a height of soil of only about 2 or 3 inches. Undoubtedly, many cases must exist in which samples intended to exhibit isotropic behavior were subjected to one-dimensional consolidation pressures greater than $.01 \text{ kg/cm}^2$ during the sample preparation process.

Morgenstern and Tchalenko (76) used a petrographic microscope to study optical birefringence in a kaolinite. Many of their conclusions were similar, at least qualitatively, to those of Martin and Ladd. They found that even in the absence of applied consolidation stresses sedimented kaolinite showed a small amount of preferred particle orientation. They also found that at a consolidation pressure of $.1 \text{ kg/cm}^2$ "intense" preferred orientation had been developed. McConnachie (55) using scanning electron microscope techniques to study the consolidation of kaolinite also found that intense preferred orientation developed at a pressure of about $.1 \text{ kg/cm}^2$ and that the degree of orientation did not change much with further consolidation.

Thus, even in samples which would normally be considered isotropic there is at least a small degree of preferred particle orientation. The degree to which a soil will exhibit anisotropic engineering properties will depend on the degree of preferred particle orientation and the stress range over which the properties are being measured.

In soils in which the degree of preferred particle orientation is high, it is likely that there will exist a high degree of undrained strength and stress-strain anisotropy. If the degree of preferred particle orientation is relatively small and the applied consolidation stresses are relatively large, the degree of strength and stress-strain anisotropy will probably be very small. This conclusion is based on the observation that at normal working stress levels isotropically consolidated samples exhibit reasonably isotropic behavior. A statement similar to this cannot be made for the case of a relatively small degree of preferred particle orientation in a sample subjected to very low applied consolidation stresses. This is due to the fact that no data could be found in the geotechnical literature on strength or stress-strain anisotropy at very low effective stress levels.

It has been pointed out previously that at low effective stress levels, interparticle attractive and repulsive forces and microdilancy play their most important roles. Both of these strength-generating mechanisms are strongly dependent on soil fabric and interparticle spacing, and thus they are probably also strongly dependent on the degree of preferred particle orientation. Based on these observations it is possible that at low effective stress levels, even small degrees of preferred particle orientation, induced by gravitational body forces, may lead to anisotropic strength and stress-strain properties.

In addition to the problem of preparing a truly isotropic specimen within the gravitational field of the earth, there is a second problem imposed by terrestrial laboratory testing; namely, the inability to apply to a sample truly uniform stress and strain fields. As pointed out by Sture et al. (104) the objective in soils testing is often to subject an "elemental test specimen to orthogonally oriented actions and measure the ensuing responses which in turn can be reduced to stress and strain tensors". A key requirement in obtaining meaningful stress-strain data from the test results is that the stress and strain states be reasonably uniform through the entire sample. Nonuniformity within a test specimen can lead to the incorrect assessment of stress-strain properties and progressive failure. On earth, the presence of gravitational body forces causes all laboratory soil specimens to be subjected to a nonuniform stress state. In a typical 8.9 cm tall triaxial specimen the vertical stress at the bottom of the sample is approximately 18 grams/cm^2 greater than at the top of the sample. If larger samples are used, this stress difference will be proportionally larger.

At relatively large stress levels (several kg/cm^2) the applied stresses are so much larger than the gravitational body forces that the nonuniformities imposed by gravity can be neglected. At very low stress levels this may not be the case, particularly if the objective of the laboratory testing is to obtain very accurate stress-strain data on a soil in order to develop precise constitutive relationships. Small size samples will minimize the influence of gravity stresses; however "the instrumentation and methodologies used to monitor the specimen responses in small samples are often of questionable quality, and the accuracy with which uniform, homogenous, and controlled boundary conditions can be applied is often inadequate" (104). Thus, in tests performed at low stresses, if reasonably sized (8.9 cm or larger) specimens are used, the applied stresses may be only several times larger than the stress difference induced by gravitational body forces. This stress nonuniformity could mask the true stress-strain behavior of some soils at these low stress conditions.

The NASA Spacelab experimental program provides an opportunity to investigate, (1) The influence of sample homogeneity on the constitutive response of fine-grained cohesive soils at very low stress levels (0. to $.25 \text{ kg/cm}^2$), and (2) The influence of gravity-induced stress nonuniformities on the constitutive response of fine-grained cohesive soils. The true triaxial testing apparatus proposed for use aboard the NASA Spacelab by Sture et al. (104) appears ideally suited for implementing this investigation.

In order to carry out this investigation it is suggested that the true triaxial testing apparatus be used to subject a set of normally consolidated and overconsolidated soil samples to a series of different stress paths at confining pressures ranging between 0 and $.25 \text{ kg/cm}^2$. The experimental program would include both in-space and terrestrial soil sample preparation and in-space and terrestrial testing. As illustrated by Figure 44, samples prepared and tested in space should exhibit a truly isotropic fabric and will not be subjected to any gravity-induced stress non-uniformities during testing.

		SAMPLE PREPARATION ENVIRONMENT	
		TERRESTRIAL	SPACE
TEST ENVIRONMENT	TERRESTRIAL	ANISOTROPIC FABRIC GRAVITY INDUCED STRESS NONUNIFORMITIES	ISOTROPIC FABRIC GRAVITY INDUCED STRESS NONUNIFORMITIES
	SPACE	ANISOTROPIC FABRIC NO GRAVITY INDUCED STRESS NONUNIFORMITIES	ISOTROPIC FABRIC NO GRAVITY INDUCED STRESS NONUNIFORMITIES

Figure 44. In-space and terrestrial testing program proposed for investigating the role of gravity in influencing the stress-strain behavior of cohesive soils at low effective stress.

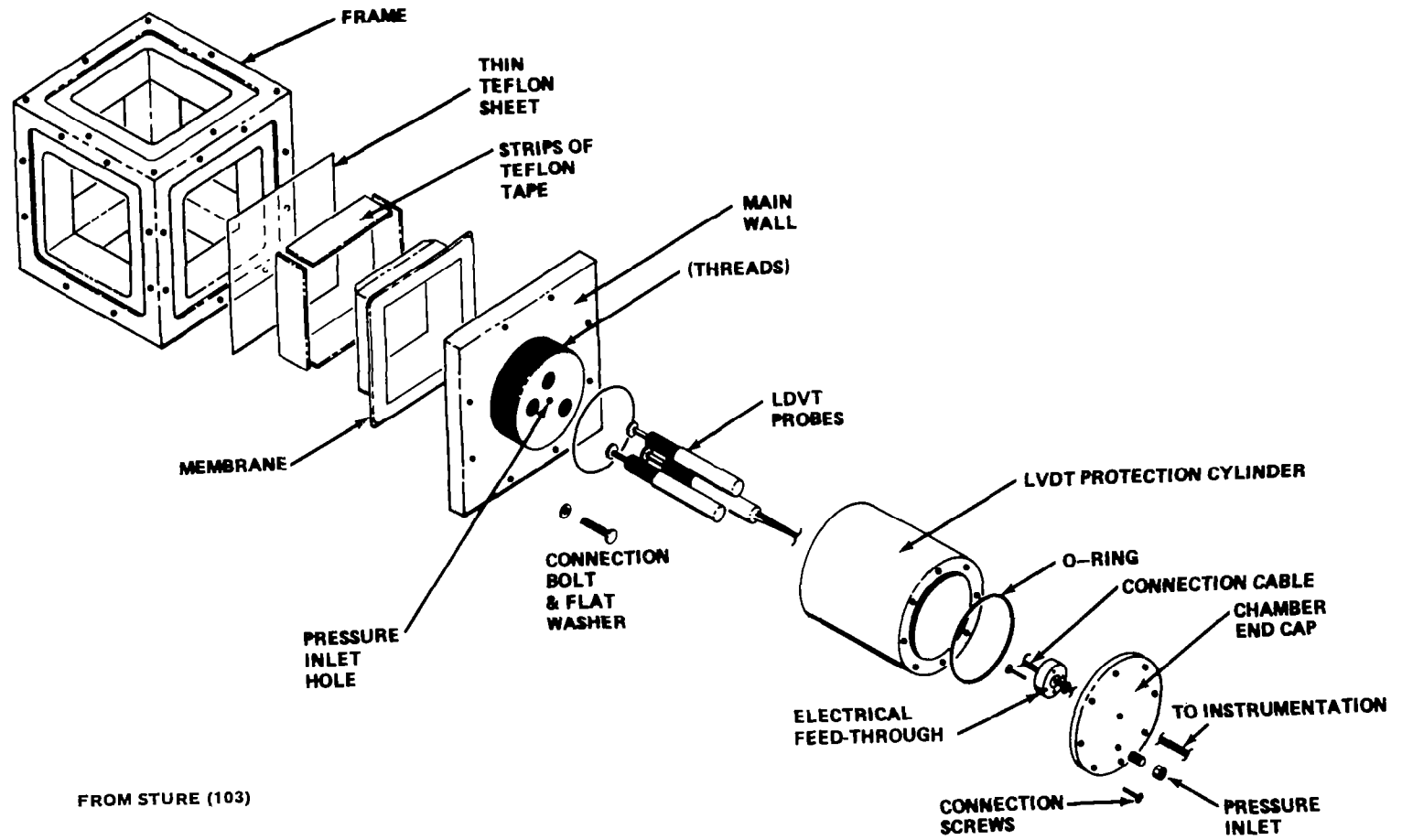
A second set of samples prepared in space would be brought back to earth and tested terrestrially. These samples would exhibit truly isotropic fabrics but would be tested in an environment in which they would be subjected to gravitational body forces. Comparison of the results of the tests performed in space and those performed terrestrially would provide a measure of the influence of gravitationally induced stress non-uniformities on cohesive soil behavior.

A third and fourth set of test samples would be prepared on earth. Both of these sets of samples, although consolidated isotropically, would still exhibit some amount of preferred particle orientation, as discussed previously. The third set of samples would be taken into space for testing while the fourth would be tested on earth. The third set of samples would not be subjected to any gravitational body forces. Therefore, any differences in the results of the third set of tests when compared to the first set of tests would presumably be due to the effects of preferred particle orientations in the third set of samples.

The results obtained from the tests on the fourth set of samples would be useful in assessing the combined effects of preferred particle orientation and gravity-induced stress nonhomogeneities.

Method of Testing. It is strongly recommended that the multiaxial cubical test device, proposed for use in the NASA Spacelab for the testing of particulate materials (104) be used to carry out the experimental program described herein. This apparatus, which is illustrated in Figures 45 and 46, applies pressure to the specimens by means of flexible membranes which contain a fluid under pressure. These pressures are controlled by three independent channels, which allow the application of three sets of independently controlled principal stresses on the six faces of the cubical specimen. This results in uniform stress distribution through the sample and the ability to follow relatively complicated stress paths. The flexible cushions allow nearly unrestrained deformations in the specimen. Both normal and shear distortions can be accommodated. Tests can be run under both drained and undrained conditions. Sture (103) has provided a detailed discussion of the multiaxial cubical test device and the instrumentation which it requires.

Sample Preparation. Provisions will be required for preparing isotropically consolidated samples both terrestrially and in space. One method of preparation, used by Martin (58) in his studies of kaolinite, calls for dumping a soil-pore fluid slurry into a large ballon lined with elastic strips. Consolidation is achieved by applying a vacuum to the ballon and slowly draining the excess pore water. Drainage in this case is essentially radial. Krizek, Edil and Ozaydin (41) used an isotropic slurry consolidometer in order to obtain isotropic samples. The isotropic consolidometer involves the use of double flexible rubber membranes which are filled with a slurry to form a sphere about 25 cm in diameter. This sphere is then floated in a liquid of slightly higher density within a pressure chamber which is loaded hydrostatically. Drainage is provided at two diametrically opposed points. After consolidation, a spherical soil



FROM STURE (103)

Figure 45. Exploded view showing one face assembly of multi-axial cubical test apparatus.

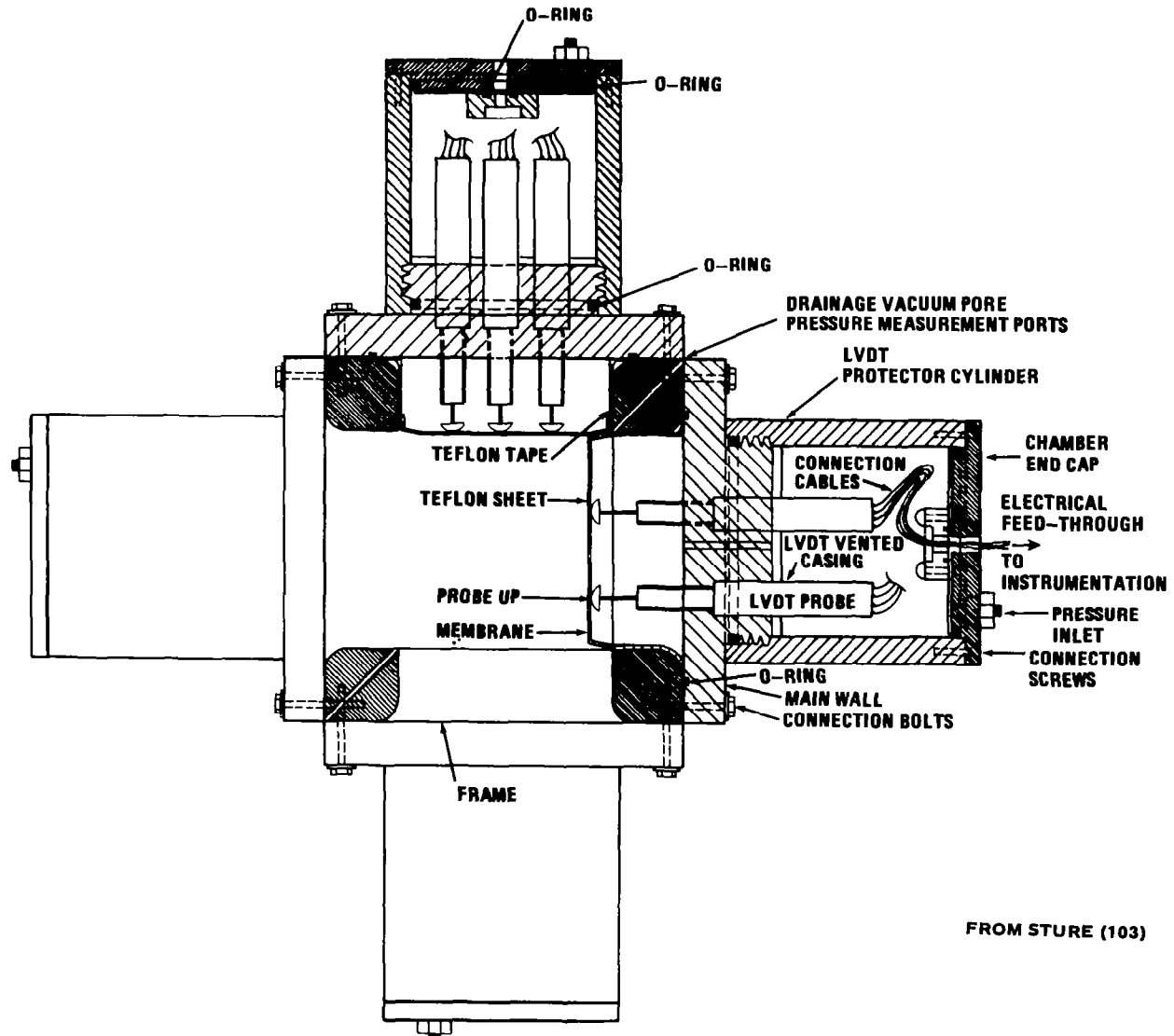


Figure 46. Cross-sectional view of assembled multiaxial cubical cell.

sample 15-20 cm in diameter is obtained. This technique could easily be adapted to large size triaxial cells using standard cylindrical membranes, radial drainage and with water as the cell fluid. It is possible that several isotropic slurry consolidometers may need to be provided aboard the NASA Spacelab in order to meet the time constraints imposed by the in-space testing. Soil trimming and handling equipment will also be required to trim the samples once consolidation is complete and to place the samples in the multiaxial cubical test apparatus after trimming.

Equipment and Instrumentation Requirements. The laboratory equipment and instrumentation requirements needed in order to carry out the multiaxial cubical soil tests have been discussed in detail by Sture (103) and Sture et al. (104). Reference should be made to these reports for further information.

Test Variables. Test variables that are germane to the study of the effect of gravity on the stress-strain behavior of cohesive soils will include:

1. Sample preparation environment (terrestrial/in-space)
2. Sample test environment
3. Soil type and properties
4. Physico-chemical environment
 - a. Pore fluid electrolyte concentration
 - b. Type of pore fluid
 - c. Type of exchangeable cation
 - d. Pore fluid pH
5. Maximum isotropic consolidation pressure
6. $\sigma'_1, \sigma'_2, \sigma'_3$, the principal effective stresses
7. $\epsilon_1, \epsilon_2, \epsilon_3$, the principal strains
8. μ , the pore water pressure
9. the boundary drainage conditions.

VI. SUMMARY AND CONCLUSIONS

This report contains a review of the nature and behavior of fine-grained cohesive soils. Emphasis has been on the fundamental mechanisms controlling strength generation in these soils, particularly under conditions of low confining stress. The role of interparticle forces in generating strength has been discussed, as has the nature of the adsorbed water layer that surrounds clay particles. The importance of interparticle contacts has been emphasized.

Based on the review it has been concluded that shear strength in soils is generated by bonding at interparticle contacts. Experimental evidence seems to support the idea that under most circumstances these contacts are mineral-to-mineral in nature. Based on activation energies measured by a number of researchers it has been concluded that the interparticle bonds are quite strong; in fact, about as strong as primary valence bonds. It has been found experimentally that the strength of a soil is proportional to the number of bonds. The hypothesis for strength generation presented in this report has been shown to be entirely compatible with the adhesion theory of friction and the theory of rate processes.

True cohesion, which is defined as the existence of a shear strength in the absence of any externally applied effective stresses, is believed to exist in some soils. True cohesion and true tensile strength can be attributed to interparticle attractive forces which may be present in the absence of any externally applied stresses. Contributions to these interparticle attractive forces may include electrostatic and electromagnetic attractions as well as interparticle bonds which may develop at contacts due to dilatancy and previous stress history. It has been pointed out that a soil has no one true cohesion value. Instead, the true cohesion is variable, changing in response to changes in the interparticle attractive and repulsive forces acting in the soil.

Experimental studies on soils at low confining stresses have been reviewed as have experimental studies designed to measure the frictional resistances developed between mineral surfaces. These studies have shown that both cohesion and friction are highly dependent on a large number of variables. True cohesion has been shown to be a function of soil type, pore fluid type, type of adsorbed cation, pore fluid electrolyte concentration and pH. The frictional resistance developed between mineral surfaces has been shown to be sensitive to the mineral type, surface moisture condition, type of surface moisture, presence of contaminant films and surface roughness.

Based on the review of theory and experiment it has been suggested that a NASA Spacelab experimental program can significantly contribute both to a basic understanding of the behavior of fine-grained cohesive soils, and to many practical aspects of geotechnical engineering. An experimental program has been recommended that involves both in-space and terrestrial laboratory testing. Three main series of experiments have been recommended. These are:

1. Experiments designed to measure true cohesion
2. Experiments designed to measure true tensile strength
3. Experiments designed to evaluate the influence of the earth's gravitational field on the constitutive behavior of fine-grained cohesive soils at low effective stress levels.

The results of these experiments should go a long way to increase engineers' and scientists' knowledge of the stress-strain-strength behavior of cohesive soils at low effective stress levels. They will also provide an increased understanding of interparticle forces, contacts, and bonding. The results will also provide geotechnical engineers with information that will be useful in the areas of terrestrial and submarine slope stability, pavement design, the control of cracking and hydraulic fracturing in earth dams and other earth structures, the control of cracking due to subsidence and differential settlements, as well as a variety of other problems.

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16. Abstract <p>This report describes a proposed experimental program to investigate fine-grained, cohesive soil deformation and strength behavior, especially under conditions of very low confining pressure. It is proposed that this program be carried out through a combination of different experiments performed in both terrestrially based soil mechanics laboratories and in space aboard the NASA Spacelab.</p> <p>Based on a comprehensive review of the literature pertaining to fine-grained, cohesive soil behavior, it has been concluded that the NASA Spacelab experimental program can add significantly to the understanding of the stress-strain-strength properties of cohesive soils. From a scientific viewpoint, the NASA Spacelab program will contribute to the scientific state-of-the-art in the areas of interparticle attractive and repulsive forces, the nature of true cohesion, and the nature of true tensile strength. The influence of gravity-induced fabric anisotropy and stress nonhomogeneities on the stress-strain behavior of cohesive soils can also be investigated.</p> <p>The results of the study will be directly applicable to many practical problems in geotechnical engineering. An increased understanding of the strength behavior of cohesive soils under conditions of low effective stress will be of use to engineers interested in the cracking and hydraulic fracturing of earth dams, slope stability, pavement design, subsidence problems and submarine landslides. In addition, engineers trying to develop constitutive models for cohesive soils will find the results of the NASA Spacelab study of use in attempting to construct yield surfaces for stress combinations in the vicinity of the stress origin.</p> <p>Three different sets of tests are proposed for the NASA Spacelab experimental program. The first set of tests is designed to measure the true cohesion of several different soils. These tests would be carried out in space through use of a specially prepared direct shear apparatus. As part of this first series of tests, it has also been recommended that a set of drained unconfined compression tests be performed terrestrially on the same soils as tested in space.</p> <p>The second set of tests involves using a form of the direct tension test to measure the true tensile strength of the same types of soils used in the first series of tests. The direct tension tests could be performed terrestrially. The combined results of the direct shear tests, direct tension tests, and unconfined compression tests can be used to construct approximate failure envelopes for the soils tested in the region of the stress origin. Relationships between true cohesion and true tensile strength can also be investigated. In addition, the role of physico-chemical variables should be studied.</p> <p>The third set of tests will make use of a multiaxial cubical or true triaxial test apparatus to investigate the influence of gravity-induced fabric anisotropy and stress nonhomogeneities on the stress-strain behavior of cohesive soils at low effective stress levels. These tests would involve both in-space and terrestrial laboratory testing.</p>			
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