

SHEAR STRENGTH AND CONSOLIDATION CHARACTERISTICS OF CALCIUM AND MAGNESIUM ILLITE

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ABSTRACT

In recent soil mechanics literature attempts have been made to explain the engineering properties of clays in terms of surface chemical theories, particularly the Gouy-Chapman theory. Experimental confirmation of these theoretical predictions have been restricted almost exclusively to studies performed on fine grain size fractions of montmorillonite. It seemed desirable to determine experimentally the effect of physico-chemical variables on the engineering properties of a more typical engineering material. The $< 2 \mu$ fraction of Fithian illite was selected.

A study was made of the effect of variations in the pore water electrolyte concentration on the engineering properties of homoionic Ca- and Mg-illite. Pore water electrolyte concentrations ranged from 1 N down to about 10^{-4} N. Engineering tests included Atterberg limits, one-dimensional consolidation tests, and effective stress triaxial tests.

The electrolyte concentration was found to have only a small effect on the Atterberg limits, the maximum limits occurring at electrolyte concentrations between 0.01 N and 0.1 N. For virgin consolidation the soil seemed strongest at about the same range in electrolyte concentrations but the effect was not large. Electrolyte concentration had almost no effect on the position of the rebound (swelling) curves. The most important effect of electrolyte concentration seemed to be its effect on the geometric arrangement of the particles for samples that were sedimented from dilute suspension. The geometric arrangement of particles seems to be a more significant variable than the osmotic repulsion between particles.

There still appears to be merit in the old mechanical approach where the compression characteristics are explained by elastic bending (reversible), slippage (partially reversible), and rupture (irreversible) of particles. Double layer phenomena seem to exert a much smaller influence on the engineering properties of nonexpanding lattice clay minerals than is commonly inferred in the literature.

INTRODUCTION

In 1958 a research program was started in the Civil Engineering Department of the University of Illinois to determine the relative effect of the various physico-chemical variables on those properties of cohesive soils that are of interest in soil mechanics and foundation engineering.

Mineralogy and grain size (specific surface) seemed to be of prime importance. Illite was selected as the mineral to receive first study because very few data are available on its engineering properties and because of its wide distribution in the glacial deposits of North America. Most previous studies have utilized the $< 0.2 \mu$ fraction to magnify the effect of the physico-chemical variables and to simplify certain experimental procedures. For an engineering study it seemed desirable to put these variables in more proper perspective by working with a coarser grain size fraction. The $< 2 \mu$ fraction of Ca- and Mg-illite was used. The concentration of the chloride in the pore water was controlled in the range from about 1 N down to about 10^{-4} N.

The engineering properties of interest are the Atterberg limits, consolidation characteristics and shear strength.

ACKNOWLEDGMENT

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HISTORICAL

The effect of mineralogy and adsorbed cations on the Atterberg limits were investigated by White (1958). Rosenqvist (1955) and Bjerrum (1954) have shown that the concentration of salt in the pore water affects the Atterberg limits of Norwegian clays. Waidelich (1957) determined the effect of variations in the pore fluid on the Atterberg limits.

The effect of physico-chemical variables on the consolidation characteristics of clays has received considerable attention since it provides an experimental check on the validity of the Gouy-Chapman theory. "Ambient stress" consolidometers were developed during the Cornell research project and were used to obtain $e-p$ curves for homoionic montmorillonite and illite. Further, tests of this type have been performed by Bolt (1956), Bolt and Miller (1955), Warkentin, Bolt, and Miller (1957), and Yong and Warkentin (1959). Samuels (1950) performed a series of one-dimensional consolidation tests on homoionic bentonite and kaolinite and obtained both $e-p$ curves and permeability data. Samuels was unable to perform

chemical analyses and so could not determine the salt concentration in the pore water. Salas and Serratosa (1953) performed one-dimensional tests on homoionic bentonite and kaolin, using water and other fluids in the pores. Waidelich (1957) performed a comprehensive series of consolidation tests on bentonite and kaolinite using seven different pore fluids.

The effect of physico-chemical variables on the shear strength has received slight attention. Samuels (1950) and Yong and Warkentin (1959) used direct shear tests to study shear strength.

Many of the above studies, especially those performed by engineers, have suffered from certain defects, *viz*: (1) The mineralogy of the soil was either inadequately defined or too complex to yield meaningful results. (2) Chemical analyses were not performed to substantiate the assumptions regarding purity, pore water salt concentration, etc. (3) Chemical preparation procedures were so harsh that the properties of the clay were altered (*e.g.* by long electro dialysis or acid treatment). (4) The effect of soil structure was not recognized.

SAMPLE PREPARATION

The raw clay was obtained from Fithian, Illinois. The outcrop was sampled at several places and X-ray diffraction patterns were made to study the mineralogy. Most of the samples contained mixed-layer material. The illite used in this research was well crystallized and essentially pure (Fig. 1).

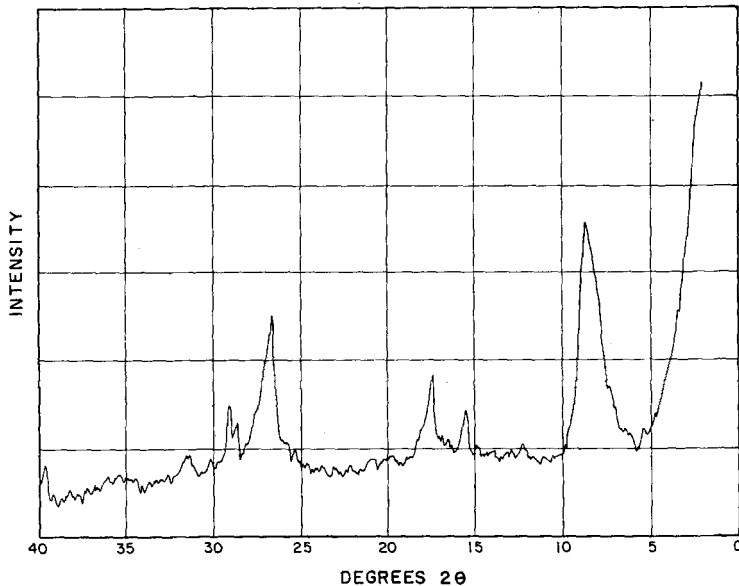


FIGURE 1.—X-ray pattern of illite used in this research. CuK_α radiation, no treatment.

About 600 lb of the raw clay was partially air dried, pulverized, and passed through a #40 sieve. The coarser material was discarded. The minus #40 material was completely air dried, homogenized, and stored. The samples were prepared in 5000 g batches. Each batch was washed with distilled water (about 40 gal) to remove soluble salts, neutralized with NaOH (pH of the raw suspension prior to treatment was 4.2) and allowed to react with an amount of NaCl equal to ten times the base exchange capacity. The clay was then washed with distilled water resulting in dispersion. Each batch was fractionated about twenty times using Stokes' law to calculate settlement velocities.

Calcium and magnesium clays were prepared by washing in 0.1 N solutions of the chlorides. The Gouy double layer theory can be used to show that this procedure will result in an essentially pure homoionic clay (Eriksson, 1952; Bolt, 1955; Bower, 1959; and Lagerwerff and Bolt, 1959). The purity of the clay was determined by comparing the amount of adsorbed calcium or magnesium with the cation exchange capacity. All samples were found to be pure within experimental accuracy. Calcium and magnesium were determined using versenate (Cheng and Bray, 1951). Samples were washed with distilled water to obtain lower pore water salt concentrations. The clay begins to disperse at a pore water CaCl_2 concentration of about 2×10^{-3} N. A steam driven Sharples super centrifuge was used to break down the suspensions for further washing.

ATTERBERG LIMITS

The Atterberg limits of the Ca- and Mg-illite were determined using standard engineering techniques (ASTM, 1958) with the following exception. The water content was defined as the weight of pore fluid (water plus dissolved salts) divided by the weight of solids.

The limits of the Ca-clay are slightly higher than those of the Mg-clay (Fig. 2). The limits are only slightly affected by changes in the pore water electrolyte concentration, being highest around 0.01 N to 0.1 N. Consequently, it is anticipated that the soil will have maximum resistance to confined compression and maximum shear strength at about the same pore water electrolyte concentration.

The Atterberg limits provide a convenient means of comparison between the illite used in this investigation and the illites reported in the literature. Casagrande's plasticity chart (Fig. 3) was used for comparison. White's no. 1 illite was obtained from the same site in the Fithian locality as the illite used in this investigation and the limits are very similar. Some of the illites contained appreciable quantities of mixed-layer material which resulted in higher plasticity.

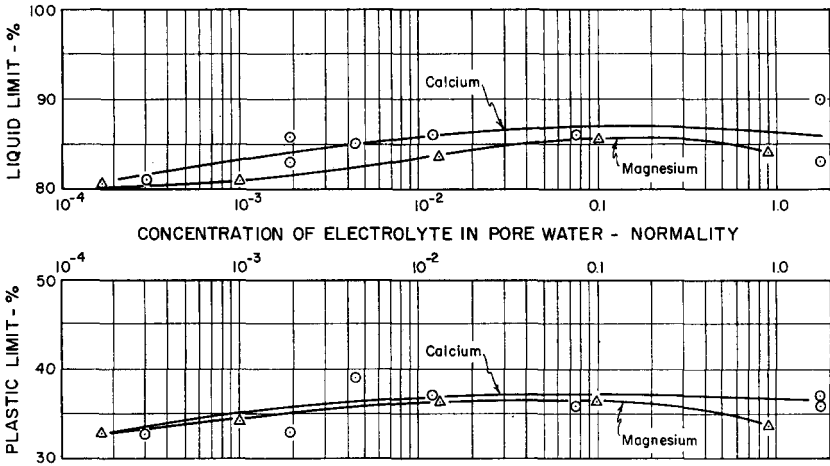


FIGURE 2.—Effect of electrolyte concentration on the Atterberg limits.

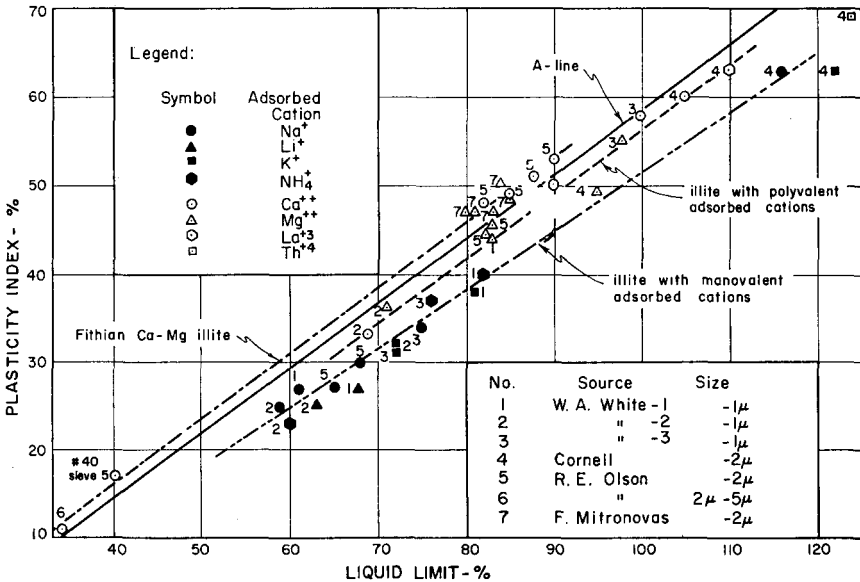


FIGURE 3.—Atterberg limits of illite.

CONSOLIDATION CHARACTERISTICS

Apparatus

One dimensional consolidation tests were performed using fixed ring consolidometers as shown in Figs. 4 and 5. The rings were made of cadmium-coated steel, plastic, or Teflon-coated steel or brass. Pressures were applied in consolidation frames by a lever arm arrangement. The applied pressures ranged from 20 psf to 64,000 psf (0.01 to 30 atm). The soil samples were generally 2.500 in. in diameter.

Theory — Void Ratio vs. Pressure Curves

The Gouy-Chapman theory has been used to calculate the relationship between particle spacing and osmotic pressure at the midplane. If the particles are assumed to be perfectly parallel to each other and to have negli-

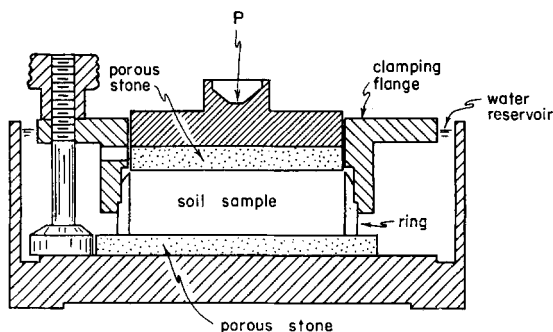


FIGURE 4.—Wykeham-Farrance consolidometer.

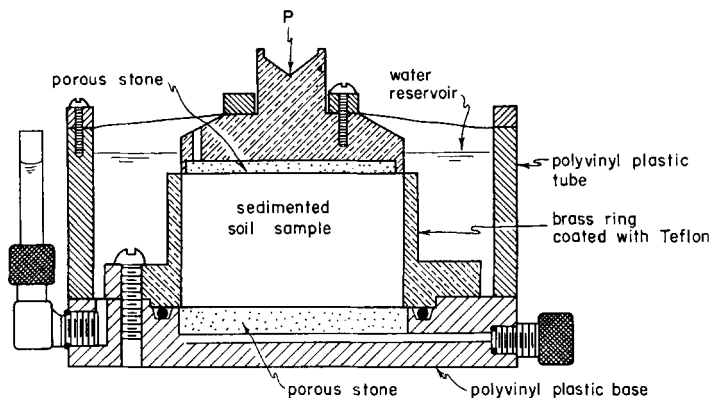


FIGURE 5.—Brass consolidation ring.

gible edge area the particle spacing can be converted to void ratio by the equation:

$$e = \gamma_s S d, \quad (1)$$

where e is the void ratio $\left(\frac{1 - V_s}{V_s}\right)$ where V_s is the volume of solids in a unit volume of soil), γ_s is the density of solids, S is the specific surface (area per unit weight), and $2d$ is the clear distance between particles.

It has been shown (Bolt, 1956) that the Gouy-Chapman theory can be used to calculate theoretically the position of the swelling curve for the $<0.2 \mu$ fraction of Na-montmorillonite. Bolt and Miller (1958) discuss the problems associated with the application of the theory to illite. Bolt (1956) presents experimental data to show that the theory does not describe the position of the swelling curve for Na-illite. He attributes the lack of agreement to the terraced nature of the surfaces.

Alternatively, a combination of the physico-chemical and a mechanical approach can be used. In the early days of soil mechanics the compression characteristics of soils were explained in terms of elastic compression and bending of flakes (reversible) coupled with breakage and slippage of flakes (irreversible). It seems apparent that this approach cannot explain the effect of pore water electrolyte concentration on the compression characteristics of fine-grained Na-montmorillonite. However, the Gouy-Chapman theory is of little use in explaining the compression characteristics of ground mica. Most of the cohesive soils encountered by soils engineers have intermediate properties and probably exhibit both types of compression and rebound.

Experimental Void Ratio vs. Pressure Relationship

In soil mechanics the initial compression curve is referred to as the "virgin" compression curve. The position of the virgin curve is related to the structure of the soil (Casagrande, 1932, p. 81; Mitchell, 1956), being lower as the grains are more nearly parallel. Slow sedimentation of soil from dilute suspension gives individual grains maximum freedom of movement and should result in the most flocculent structures. Application of shearing stresses will break down the structure and result in an irreversible decrease in void ratio. This effect is shown in Fig. 6 where a series of compression curves of Ca-illite are presented. The only difference between the curves is in the water content at which the samples were remolded.

A "normally consolidated" clay is here defined as one having the highest void ratio at the given pressure. The normally consolidated e -log p curve is therefore the curve obtained from the sample sedimented from dilute suspension. The data in Fig. 6 show that a pressure of 50,000 psf is required to merge all curves. If a sample is prepared at the liquid limit (about

84 percent) it will be "overconsolidated" until it has been consolidated to about 20,000 psf. The compression curve of a sample that is being loaded for the first time will be called the "initial compression curve." Consolidation tests were performed on both Ca- and Mg-illite at wide ranges in molding water contents and for a range in pore water electrolyte concentration from 1 N down to about 10^{-4} N. Curves similar to those in Figs. 6 and 7 were obtained in all cases.

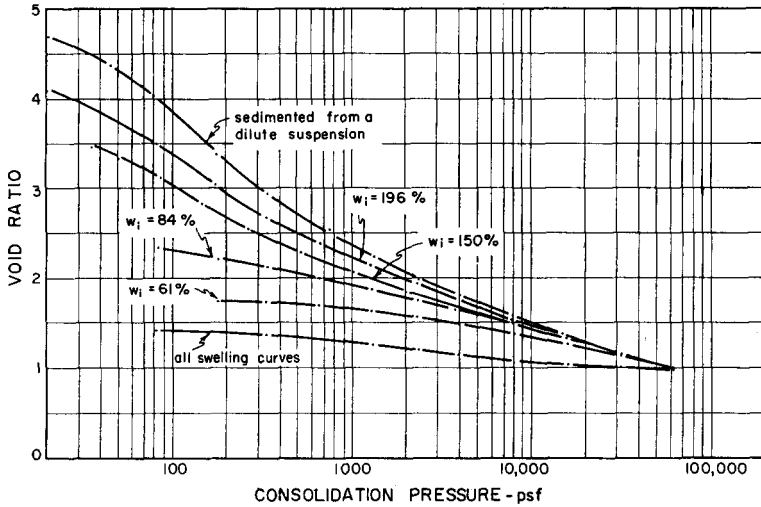


FIGURE 6.—Compression curves of Ca-illite. All tests at $C = 0.01$ N.

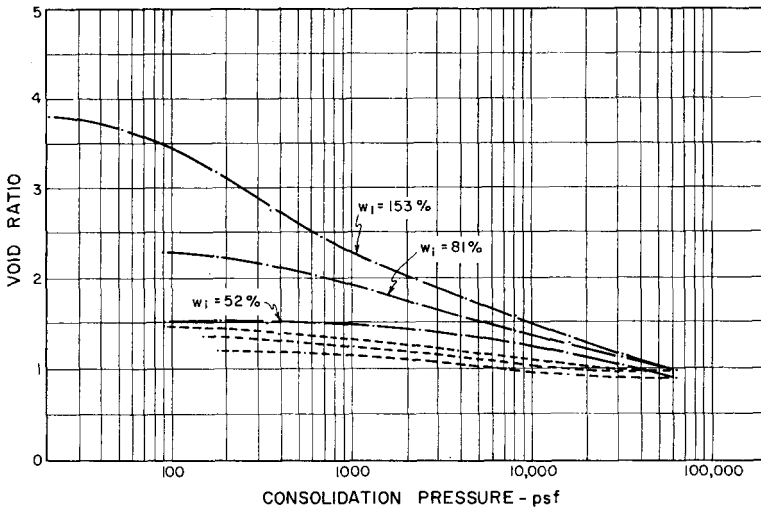


FIGURE 7.—Compression curves of Ca-illite. All tests at $C = 1.9 \times 10^{-3}$ N. Lower dashed lines are rebound curves.

The difference between the initial compression and the rebound curves could be explained by the purely mechanical approach: the rebound represents the elastic part of compression and the difference between the initial compression and rebound curves is the inelastic part resulting from grain slippage and breakage. This approach has been objected to by Salas and Serratosa (1953) and others, and quite rightly too, on the grounds that it does not explain the large changes in the positions of the curves due to

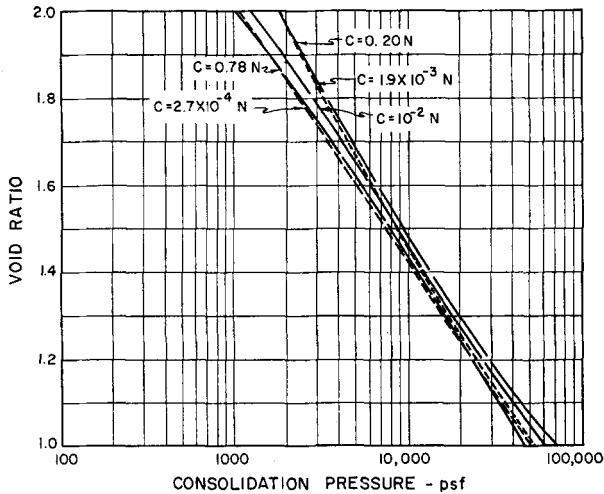


FIGURE 8.—Compression curves of Ca-illite. All curves at an initial water content of 120 percent.

changes in pore water electrolyte concentration, pore fluid, etc. The effect of variations in pore water electrolyte concentration was investigated. Studies of the effect of variations in the pore fluid are in progress.

The effect of electrolyte concentration on the position of the initial compression curves can be determined only if some convention is adopted for the comparison. Three possible conventions are available: (1) Compare samples that were remolded at the same water content. (2) Compare samples that were prepared at the same liquidity index, *e.g.* at the liquid limit. (3) Compare samples that were sedimented from very dilute suspensions.

Graphical interpolation was used to determine the e -log p curves for samples of Ca-illite remolded at a water content of 120 percent (Fig. 8). It is apparent that the position of the initial compression curve is far more sensitive to slight changes in the initial water content (structure) than to very large (10^4 times) changes in pore water electrolyte concentration. The effect of electrolyte concentration is more easily discerned if the curves are replotted as in Fig. 9. At high void ratios the position of the e -log p curve

was so sensitive to slight variation in initial structure that considerable scatter results. As expected from the study of Fig. 6, the effect of variations in initial structure tends to disappear at high pressures. Therefore, at high pressures (low void ratios) a reasonably smooth relationship results. The soil has the greatest resistance to compression at salt concentrations near 0.1 N just as predicted from Atterberg limit data.

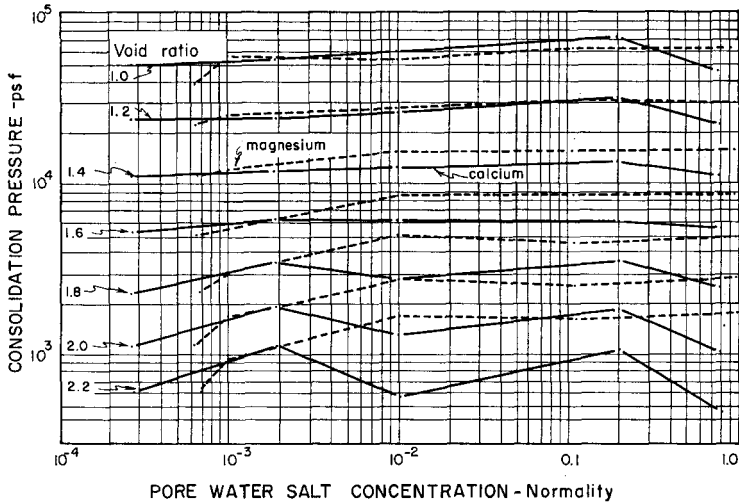


FIGURE 9.—Effect of electrolyte concentration on the compression of illite. All samples sedimented from suspension.

Graphical interpolation was also used to compare e - $\log p$ curves of samples remolded at the liquid limit and similar results were obtained.

Compression curves for samples of Ca- and Mg-illite that were sedimented from dilute suspensions are shown in Figs. 10 and 11. These curves indicate that the soil has its greatest resistance to consolidation at intermediate (0.01 N to 0.1 N) salt concentrations. Further, Ca- and Mg-clay have nearly identical properties. The probable experimental error is ± 0.05 in void ratio. The apparent effect of salt concentration is not much greater than experimental error. The differences in the positions of the calcium and magnesium curves at low pressures probably are due to the fact that experimental procedures had been more refined just prior to the magnesium tests which were, therefore, subject to less disturbance during the test set-up.

Rebound (swelling) curves for Mg-illite are shown in Figs. 12 and 13, and average swelling curves of sedimented samples of both Ca- and Mg-illite are compared in Fig. 14. Again it is apparent that Ca- and Mg-illite behave essentially identically and that the effect of pore water electrolyte concentration is small.

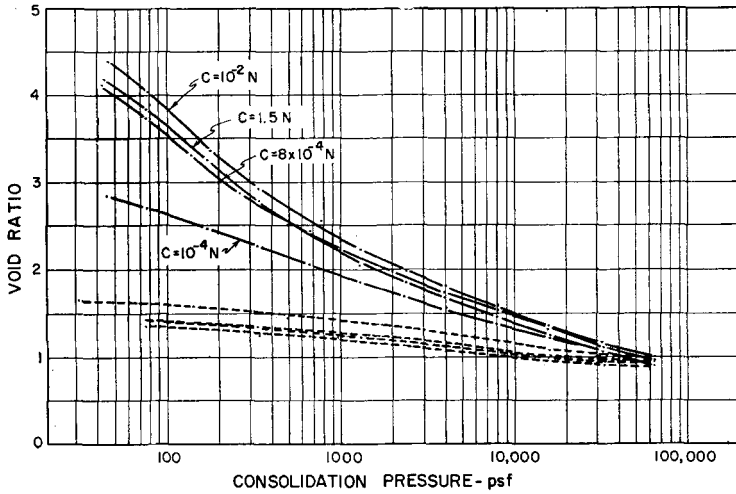


FIGURE 10.—Compression curves of Ca-illite. All samples sedimented from dilute suspension.

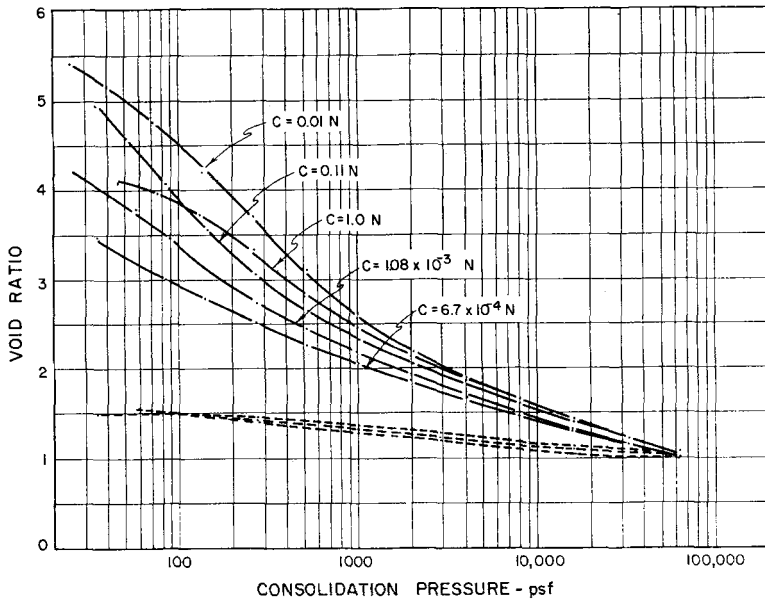


FIGURE 11.—Compression curves of Mg-illite. All samples sedimented from dilute suspension.

The position of the swelling curves can be calculated using the Gouy-Chapman theory. No specific surface determinations were performed. However, specific surface data on the $< 2 \mu$ fraction of illite have been reported by Orchiston (1959), Diamond and Kinter (1958), and in the Cornell Reports (1949-1951). The range was from 81 to 95 m^2/g . The error involved in assuming an average value of 90 m^2/g is probably slight. The

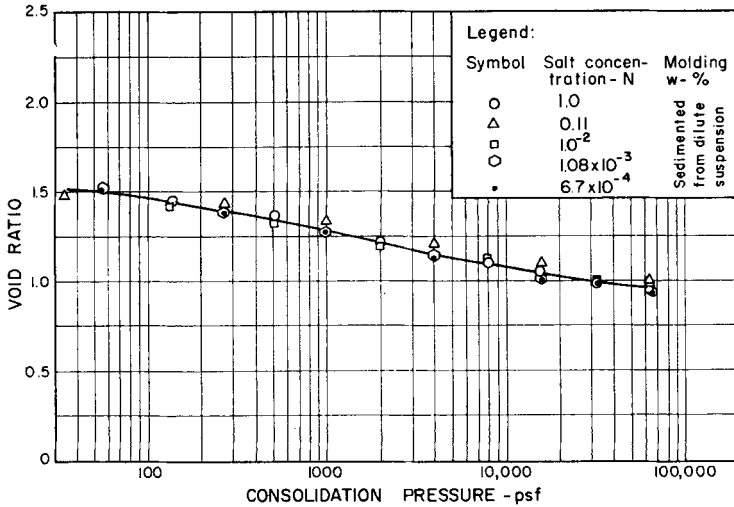


FIGURE 12.—Swelling curves of Mg-illite.

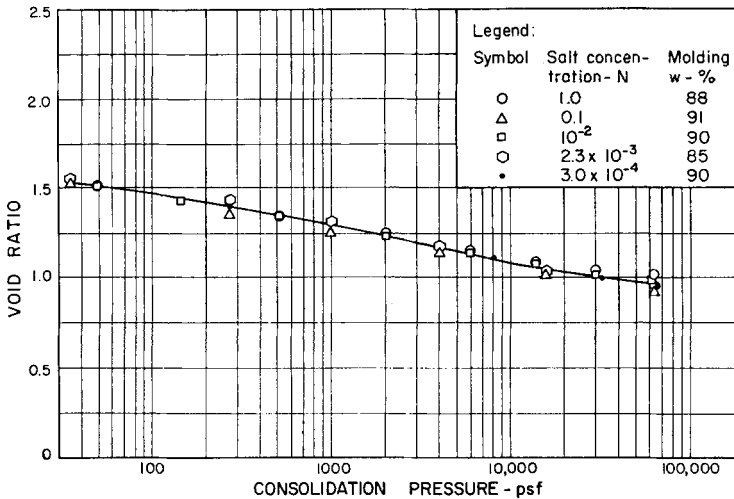


FIGURE 13.—Swelling curves of Mg-illite.

cation exchange capacity determinations averaged about 24 meq/100 g. The surface charge density is therefore about 2.67×10^{-7} meq/cm². The curves in Fig. 15 were calculated using the above figures and the solutions published by Bolt (1956). These curves bear little resemblance to the experimental swelling curves shown in Figs. 12 and 13. Various factors can contribute to this difference.

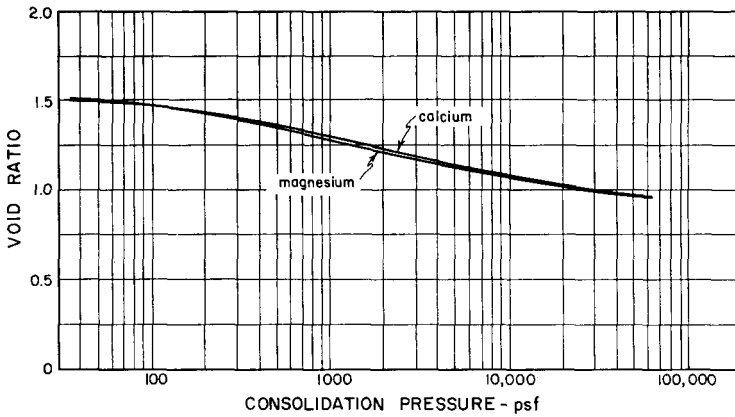


FIGURE 14.—Comparison between swelling curves. Both curves are for illite sedimented from dilute suspension.

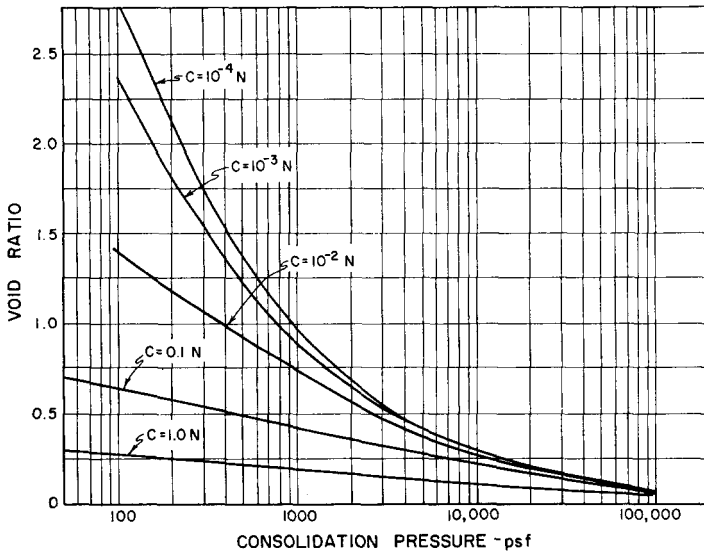


FIGURE 15.—Compression curves based on Gouy-Chapman theory. These calculations were performed for $< 2 \mu$ Ca-illite. The surface charge density was taken as 2.67×10^{-7} meq/cm². The specific surface was 90 m²/g.

Bolt (1956) attributed the difference to the terraced nature of the particles. As a first approximation it could be assumed that there is a "dead volume" of pore space resulting from these terraces. If a soil is dried at 110 °C the terraces should come into contact and the void volume might be a measure of the dead volume previously mentioned (suggested by Yong, personal communication). The void ratio at this so-called "shrinkage limit" is around 0.60. The addition of such a correction to the theoretical curves produces substantial improvements at high pressures; however, it does not result in the required separation of the swelling curves at reduced pressures.

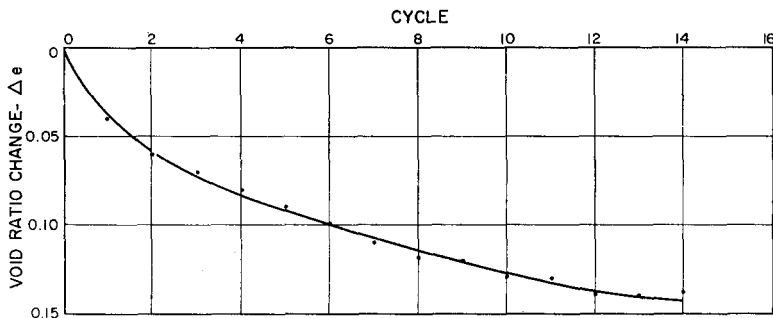


FIGURE 16.—Void ratio reduction resulting from repeated loading. Test performed on $< 2\mu$ Ca-illite at $C = 0.01$ N; $w_i = 144$ percent.

A second possibility is that the particles have not assumed a parallel arrangement at the pressure of 64,000 psf. One sample was cyclically loaded and unloaded between 50 psf and 64,000 psf fifteen times. In Fig. 16 the reduction in void ratio at the maximum pressure resulting from cyclic loading is plotted against the cycle number. The gradually decreasing void ratio is interpreted to mean a gradually increasing degree of parallelism of grains. A correction for the lack of parallelism improves the correlation at high pressure but, again, does not explain the lack of effect of electrolyte concentration.

A third factor is the assumption of negligible void volume between the edges of the particles. This assumption is valid for the very flat and thin flakes of montmorillonite but may not be so valid for the more equidimensional flakes of illite. Kahn (1959) performed a series of experiments to determine the size and shape of clay mineral flakes. Based on this experiment, calculations have been performed for isodispersed disc-shaped particles with thickness of 90 Å and diameters of 1600 Å. Such particles have a specific surface of about 84 m²/g. Both dimensions and the ratio of diameter to thickness fall within Kahn's experimental range. It is assumed that the particles are in perfectly parallel array. Consideration was given to two possible

arrangements of particles in a horizontal plane: (I) Place the particles at the corners of a lattice of equal sided parallelograms (close-packed); and (II) place the particles at the corners of a lattice of squares. In addition we may consider the effect of having the minimum clearance between particles in the horizontal plane being 0 Å, 10 Å, and 100 Å. Calculations were performed for two theoretical (no edge volume) void ratios. The results are shown in Table I. These calculations show that significant errors may result from ignoring the volume between particle edges. The assumption of an isodispersed system results in a certain amount of error, but calculations based on the real soil cannot be performed. Again, the correction seems most use-

TABLE I.—ERRORS IN THEORETICAL VOID RATIOS DUE TO VOLUME BETWEEN PARTICLE EDGES

Horizontal Arrangement	Void Ratio (Theoretical)	Void Ratio after Adjustment for Volume between Edges		
		0 Å	10 Å	100 Å
I	1.00	1.04	1.09	1.32
II	1.00	1.38	1.41	1.68
I	0.20	0.28	0.31	0.47
II	0.20	0.49	0.52	0.69

ful at high pressures. It does not explain the lack of effect of salt concentration on the swelling curves.

van Olphen (1954) has shown that there must be a certain amount of specific adsorption of cations onto the surface of bentonite. If we assume that a major percentage of the adsorbed cations are held very close to the mineral surface by specific adsorption (in a Stern layer) then variations in pore water electrolyte concentration should have a smaller effect than predicted by the Gouy-Chapman theory. The adjusted theoretical void ratios would be lower than those predicted by the Gouy-Chapman theory.

As pointed out by van Olphen (1960) the Gouy-Chapman theory predicts the fluid pressure between particles. If the particles are effectively in contact then the pressures predicted by the theory will be less than the externally applied pressure by this contact pressure. An independent determination of the actual contact pressures (if any) does not seem possible. If the particles are in contact, and remain in contact during rebound, then the possibility of elastic rebound deserves more attention. This possibility can also be studied by replacing the water with other fluids such as decalin and carbon tetrachloride. A consolidation test with air as the pore fluid would be even more interesting but there is no convenient way of obtaining dispersion of a dry soil in air.

An alternative approach might be to perform tests on a mineralogically similar material of coarser grain size. Such tests were performed on ground muscovite. The muscovite was ground to pass the #40 sieve (0.105 mm) with most retained on the #200 sieve (0.074 mm). Consolidation tests were performed on the oven-dry material, on samples settled from dilute suspension in solutions of CaCl_2 , and on samples in decalin (decahydronaphthylene) and carbon tetrachloride. The results of these tests are presented in Fig. 17.

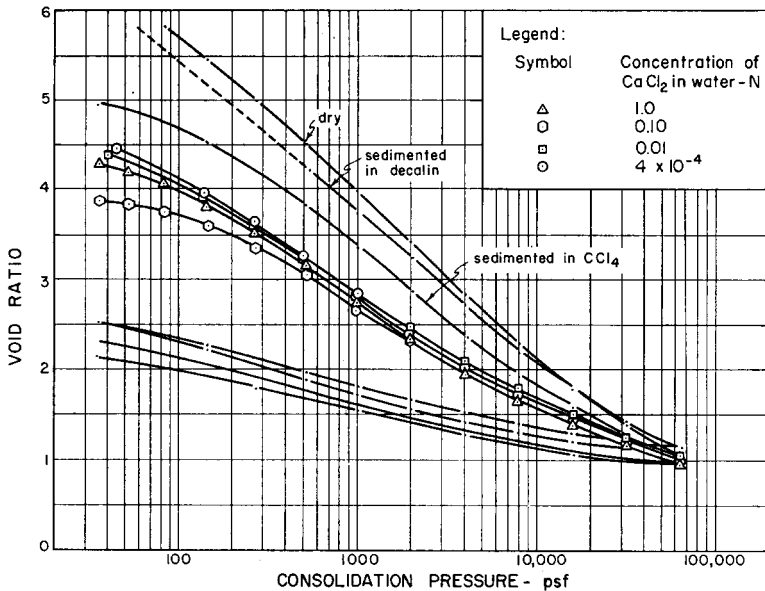


FIGURE 17.—Compression curves of ground muscovite.

Pore water electrolyte concentration has about the same effect on the curves for ground muscovite as for illite, *viz.* almost none, but with the 0.01 N–0.1 N tests having slightly higher void ratios.

The void ratios of the test on oven-dried muscovite are much greater than for tests on muscovite in water. The explanation lies in the magnitude of the friction between flakes at points of contact. Horn (1960) has shown that the coefficient of friction (μ) between parallel plates of muscovite is dependent upon the amount of water adsorbed onto the surface. He obtained coefficients of friction ranging from 0.4 for tests performed at relative humidities less than 10 percent to 0.2 for tests submerged in water.

In a consolidation test on muscovite, the high void ratios (at low pressures) result from a loose particle arrangement involving edge-to-face contact. An increase in the axial pressure results in shearing forces which tend to

break down this structure. The degree of breakdown is determined by the frictional resistance between flakes. Submerged muscovite has the lowest frictional resistance and therefore has the lowest void ratios. Conversely, oven dry muscovite has the highest frictional resistance and the highest void ratios.

The reduced friction between muscovite flakes when water was added to dry material undoubtedly resulted from the adsorption of the water which then prevented actual physical contact between the flakes. The shearing stresses are then assumed to be carried by the adsorbed water. If this assumption is true, then high shearing resistance, and high void ratios in consolidation tests, should result from submergence of muscovite in a liquid that is not adsorbed strongly. Tests were performed by Horn (1960) on samples submerged in decalin, which is nonpolar. He found high coefficients of friction (0.36). Consolidation tests were performed on muscovite in decalin and in carbon tetrachloride and the compression curves were found to be close to that of the dry muscovite just as predicted.

In oven-dry muscovite, the rebound (swelling) cannot be attributed to adsorption of fluid nor to osmosis, since the pore fluid is air. It seems probable that the rebound is a purely elastic phenomenon. The flakes were bent during virgin consolidation and they tend to spring back when the load is removed. The tests with decalin and carbon tetrachloride tend to confirm this concept. Since the rebound curves of all tests on muscovite are essentially parallel it seems logical to attribute the swelling of the samples in salt solutions to elastic rebound also. Adsorption of water and osmotic swelling doubtless occur but the effect seems small.

In comparing tests on muscovite with those on illite certain factors must be considered. First, Kahn's (1959) data indicated that the diameters of illite particles were likely to be from 5 to 50 times the thickness. Examination of ground muscovite particles with a binocular microscope indicated that this ratio had a very similar range. If elastic bending is related to this ratio, then illite and muscovite should be very similar.

Illite particles have a far greater specific surface so that surface adsorption phenomena should have greater effect than for ground muscovite. In particular, interaction between negatively charged faces and positively charged edges would be more important in illite. Adsorption of water onto surfaces would also be more important in illite than in muscovite.

Comparisons between muscovite and illite tests indicate certain similarities. The void ratios at the maximum pressure are essentially the same. The effect of variations in pore water electrolyte concentration produce similar effects too.

The magnitude of rebound differs. Contrary to expectations, the rebound (swelling) was substantially greater in the muscovite. Part of the difference may be explained by ring friction. When a soil swells, a frictional resistance

is developed between the edges of the soil disc and the confining ring (Taylor, 1942). Hence the stress in the soil is larger than the applied stress. The magnitude of this friction can be estimated from the force required to extrude the soil from the ring after the test. This force was essentially zero for muscovite but had a rather appreciable magnitude in illite. An adjustment for ring friction would bring the rebound curves into closer proximity.

The higher void ratios during virgin consolidation of ground muscovite doubtless resulted from the narrow range in grain size of this material. Low void ratios tend to result from a more uniform distribution of particle size since the smaller particles fill in the gaps between the larger ones.

Comparisons between illite and muscovite might indicate the swelling mechanism in illite. Osmotic swelling was eliminated in the case of illite since a very large change (10^4) in salt concentration had essentially no effect on the rebound curve. Two mechanisms seem to remain, *viz.* elastic rebound and adsorption of water. The adsorption of water might produce a repulsive force between particles that would be essentially independent of salt concentration. The choice of mechanism can be made by performing tests in fluids that are not adsorbed by illite. If the same amount of swelling occurs as in water, then surface hydration is eliminated as a mechanism. Conversely, if no rebound occurs, then elastic rebound could be eliminated.

Attempts to perform consolidation tests on illite in decalin and carbon tetrachloride have been unsuccessful. During the several weeks required to perform the tests the pore fluids softened the plastic consolidometers and caused jamming of the loading caps. The high coefficient of friction between metal caps and metal rings prevented free swelling. It appears necessary to cast rings and caps in Teflon (sprayed-on Teflon comes off) in order to perform these tests.

Illite forms very large loose flocs in carbon tetrachloride and decalin. These flocs are far looser than any obtained in water and indicate strong edge-to-face flocculation. Such strong flocculation should produce very elastic structures and result in considerable rebound upon stress release. Some indication of this elastic rebound is found in the void ratios at the shrinkage limit which were higher (1.10 to 1.25) in the nonpolar fluids than in water (0.60 to 0.70).

SHEAR STRENGTH

Introduction

In soil mechanics the shear strength of soil commonly is measured with either the direct shear or the triaxial apparatus. The triaxial apparatus was used in this investigation since it allows the measurement of the pressure in the pore fluid during the test and because it allows independent control

of the major and minor principal stresses. Principles of triaxial shear testing have been covered by Bishop and Henkel (1957). A cross-sectional view of the triaxial cell used in this investigation is presented in Fig. 18. The soil sample is a cylinder with a diameter of 1.50 in. and a height of 3.00 in. It is surrounded by a filter paper drain and is confined laterally by a thin rubber membrane. Hydrostatic pressures are applied via the cell fluid. The pore fluid is squeezed out through the porous plate in the base, the volume change being measured with a 5 ml buret. Fluid pressures between 5 psi and 120 psi can be applied with this cell. Higher pressures are possible with

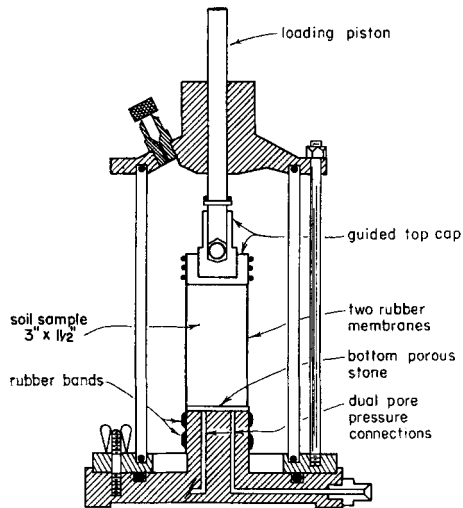


FIGURE 18.—Triaxial cell.

specially designed cells. Axial loads are applied to the soil sample via a stainless steel piston and loading cap.

The pressure in the pore fluid can be measured during a shear test to an accuracy of about 0.2 psi. It is necessary to apply the shearing stresses fairly slowly and to measure the fluid pressure under conditions of no fluid movement. Apparatus for this purpose has been developed at Imperial College (Bishop and Henkel, 1957) and at the Norwegian Geotechnical Institute (Andresen, Bjerrum, Dibiagio, and Kjaernsli, 1957).

The results of triaxial tests may be plotted on Mohr diagrams (Taylor, 1948, chapter 13). Following usual nomenclature, principal stresses will be designated by σ and shear stresses by τ . Effective stresses will be designated by using a bar over the stress symbol.

If the soil particles are effectively in contact, then the applied stress is carried by both the pore fluid and the network of soil particles. If the total

force carried by the soil particles is divided by the total cross-sectional area we obtain a fictitious pressure called the "effective stress." For saturated soils (two-phase system of soil and water) Terzaghi (1936) showed that

$$\sigma = \bar{\sigma} + u, \tag{2}$$

where σ is the total principal stress (external pressure), $\bar{\sigma}$ is the effective stress, and u is the pressure in the pore fluid. In the triaxial test the total stress and the pore water pressure can be measured independently. The results of the tests therefore can be expressed in terms of effective stresses. Extensive research in soil mechanics since 1936 has shown that the physical

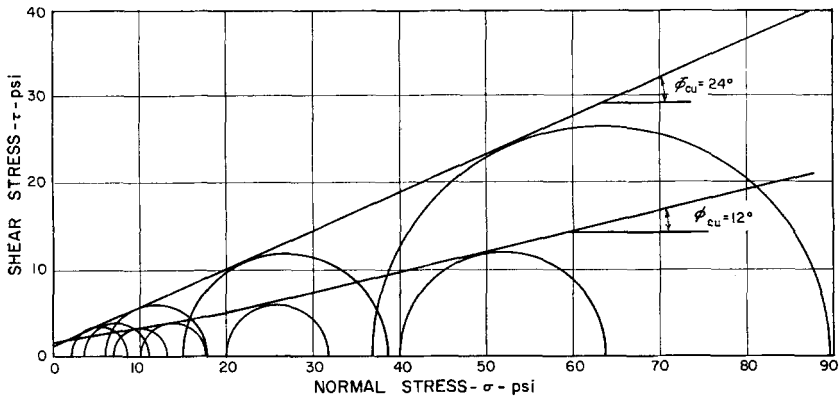


FIGURE 19.—Shear strength of Ca-illite.

properties of soils are related to the effective stress, not to the total stress. Test results expressed in terms of total stresses are often useful in engineering design but they have no place in basic research.

It should be noted that the effective stress, as defined in eq. (2), includes both the contact pressures between particles and the osmotic and hydration repulsion. The pore pressure of eq. (2) is a fluid pressure that is superimposed on both the free pore fluid and the pore fluid between the clay plates. It may be desirable to subdivide the effective stress into components in basic investigations. Such a division is difficult to make, however, since neither the actual contact pressures nor the osmotic pressures can be determined independently.

If a series of soil samples are consolidated under a range in cell pressures and then axially loaded to failure, a series of Mohr circles (at failure) can be drawn (Figs. 19 and 20). The line tangent to these circles is called a "failure envelope." It may be a straight line passing through the origin,

a straight line passing above the origin, or curved. If it is a straight line it can be represented mathematically by the equation:

$$\tau = \bar{c} + (\sigma_n - u) \tan \varphi \tag{3}$$

where τ is the shear stress, \bar{c} is the cohesion, and φ is called the angle of internal friction. Depending on how these functions are defined we may use a wide range in subscripts. These subscripts will be defined when introduced. It is noted that the cohesion is independent of applied stress while the angle of internal friction is directly related to this stress. If φ is defined in terms of effective stresses (as in eq. 3) it is designated as $\bar{\varphi}$. In this case $\bar{\varphi}$

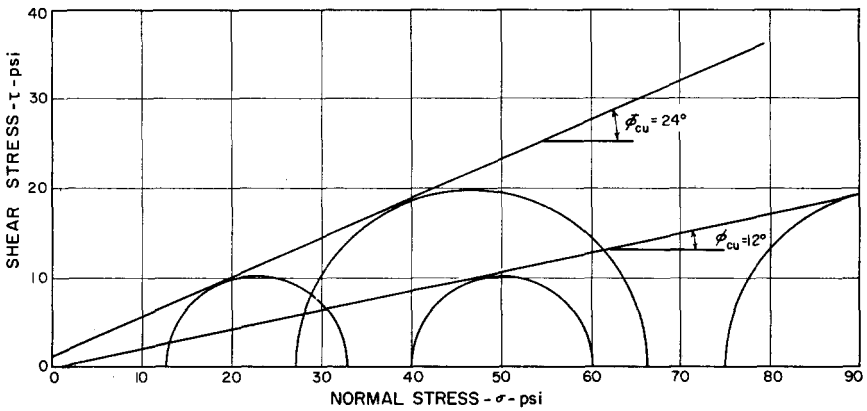


FIGURE 20.--Shear strength of Mg-illite.

is related to both the true intergranular contact pressures and the osmotic pressure between the grains. If $(\sigma_n - u)$ of eq. (3) is replaced by σ_n (the total stress) then φ is used to designate the angle of internal friction. It is then affected by the actual intergranular stress, by the osmotic and hydration pressures, and by pore water pressures. The advantage of using effective stresses in defining $\bar{\varphi}$ is now obvious. It eliminates one term.

Experimental Results

A comprehensive series of triaxial tests were performed on samples of Ca-illite with a pore water salt concentration of 0.01 N. In addition, a few tests have been performed on samples of Ca-illite at different pore water salt concentrations but under the same system of applied stresses. A limited number of tests were performed on Mg-illite.

As shown by the consolidation tests (Fig. 6) the position of the initial consolidation curve is grossly affected by the molding water content. Triaxial tests showed that the shearing properties also were greatly affected

when different molding water contents were used. Consequently it was necessary to standardize the preparation procedure. All samples reported herein were prepared by a special sedimentation procedure. Dilute soil suspensions (about 5 percent solids by weight) were prepared. Plastic tubes (1.50 in. I.D. by about 20 in. long) with a drainage connection at the base were partly filled with salt solution. The dilute soil suspensions were added about 20 ml at a time. Succeeding 20 ml batches were added after all the previous soil had settled out. In this way a "normally consolidated" sediment was formed. This sediment was first allowed to consolidate by syneresis and then under a small head of water. Then a plastic cap with drainage connections was placed in the tube on top of the soil and consolidation pressures were applied in increments up to 7 psi. This procedure generally took between 4 and 6 weeks per specimen. The specimen then was carefully extruded and set up in the triaxial cell. The water content of these specimens was above the liquid limit and very careful procedures were required for handling to avoid serious disturbance. After the specimen was set up in the triaxial cells, fluid pressures were applied in increments and the pore fluid allowed to drain to equilibrium under each pressure. The samples were then pressure saturated and sheared under undrained conditions, and pore pressure measurements were made.

The tests on "normally consolidated" Ca-illite are presented in Fig. 19 in terms of both total and effective stresses. $\bar{\varphi}_{cu}$ designates the effective stress friction angle for consolidated-undrained (consolidated under an ambient stress and then sheared under undrained conditions) tests. φ_{cu} designates the total stress friction angle. The wide difference in results obtained by the elimination of the pore pressure is obvious. $\bar{\varphi}_{cu}$ is a true measure of the effect of actual contact pressure plus osmotic and other surface chemical pressures. The cohesion intercept may be due either to the restraining effect of the filter paper and rubber membrane (Henkel and Gilbert, 1952) or to slight overconsolidation of the sample owing to disturbance during extrusion. Two similar tests with Mg-illite are presented in Fig. 20. Again Ca- and Mg-illite are found to be identical.

Preliminary tests indicate that the shear strength is a maximum for a pore water CaCl_2 concentration of about 0.01 N but sufficient tests have not been performed to establish a definite relationship. As found with consolidation tests, the effect of salt concentration seems only slightly larger than normal experimental error and is, therefore, a difficult variable to study.

Horn (1960) has performed two types of shear tests with muscovite. In one, individual flakes of muscovite were sheared across other flakes. As previously mentioned the shearing resistance was related to the humidity of the atmosphere, being about 0.4 ($\bar{\varphi} = 22^\circ$) in the oven dry state and about 0.2 ($\bar{\varphi} = 11^\circ$) in the submerged condition. Tests also were performed on

ground muscovite in direct shear. Horn obtained $\bar{\varphi} = 27^\circ$ in the oven-dry state and $\bar{\varphi} = 16^\circ$ in the submerged condition. The difference between the oven-dry and submerged tests results from the adsorption of water onto the surface. The difference between tests on large single crystals and on ground material (about 5°) probably results from the added energy needed to rotate the small flakes into a parallel orientation along the failure plane.

The effective stress friction angle of illite (about 24°) is about what would be expected from the other data:

single crystal friction angle	11°
ground muscovite friction angle	16°
illite friction angle	24°

These data imply that the larger friction angle of illite results from the increased amount of work required to rotate particles along the failure plane. A detailed analysis of the frictional properties of illite is being started to explain the shear strength. Torsion shear tests on illite with a wide range in pore water electrolyte concentrations, several types of adsorbed ions, and both polar and nonpolar pore fluids, will be used. It is hoped that a detailed analysis of $\bar{\varphi}$ and c will be possible using these tests.

CONCLUSIONS

- (1) The Fithian illite sample is similar to illites encountered in the literature though it is slightly less plastic.
- (2) The Atterberg limits show that the plasticity of the Ca-clay is slightly greater than that of the Mg-clay but the difference is too small to be of significance.
- (3) The clay has maximum plasticity for a salt concentration between 0.01 N and 0.1 N. The effect of salt concentration is rather small.
- (4) The $e-p$ relationship (one-dimensional confined compression) is very sensitive to changes in molding water content. This effect can be explained in terms of soil structure.
- (5) The $e-p$ relationship is affected by pore water electrolyte concentrations but only to the extent that electrolyte affects the initial structure. The maximum "virgin" void ratios occur when the electrolyte concentration is between 0.01 N and 0.1 N.
- (6) Rebound (swelling) curves are independent of pore water electrolyte concentration within experimental accuracy.
- (7) The consolidation characteristics of Ca- and Mg-illite are essentially identical.
- (8) The consolidation characteristics of illite are very similar to those of ground muscovite implying that some of the rebound (swelling) of illite may be of an elastic nature.

(9) Both Ca- and Mg-illite have an effective stress friction angle of 24° and a total stress angle of 12° .

(10) The effective stress friction angle is about that which would be predicted from Horn's shear tests on mica.

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