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Effects of Water Adsorption on Kaolin Clay During Shear

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Stress-controlled direct shear tests were made on laboratory-desiccated specimens of kaolin clay ($w_D \sim 1.5$ percent). The standard apparatus was modified to permit progressive adsorption of water through the bottom porous plate. Various levels of constant shearing stress τ and a vertical stress of 24 kPa (3.5 lbf/in^2) were applied, and shearing deformation measurements to failure were taken. When the deformation rate accelerated markedly, failure was defined for that particular τ and water content w. Failure surface w determinations were confirmed by an independent study of time versus wetting rate. Deceleration in wetting rate with increasing water content indicated decreasing negative pore pressure or soil water suction. Suction increases shearing resistance by its "reinforcement" of effective normal stress between soil solids; it is greater at lower w and reduces most at w > 23 percent—the point of contraflexure in the wetting rate versus w curve—which is followed by swelling. Higher constant stress levels produced initial deformations at lower w and at higher deformation rates. The approximately linear relation between applied τ and w at initial deformation indicates that increases in w account for decreases in interparticle shearing resistance and corresponding losses of strength. This is confirmed by qualitative analysis of test results in relation to the Coulomb equation and the principle of effective stress. Because the maximum strength of specimens at w_D was greater than maximum strengths between 1.5 and 23 percent, stress-controlled tests were replaced by strain-controlled tests; therefore, peak and residual strengths were obtained on desiccated specimens rewet to w in this range and held constant during stress application. These tests were extended to w_E (equilibrium w ~ 35 percent) for comparison with stress-controlled test results.

Most of the available laboratory test data on strength and strain in soils have been obtained over relatively short periods of time, generally from a few minutes to a few hours. However, in field situations, long-term loads maintain stress for years and have been observed to produce continuing small strains. This phenomenon, known as "creep," occurs in soils under sustained high stress, normally without change in water content w. Many environmental factors affect soil response but are difficult to identify. High on any list of the causes of the instability of clays in relation to decrease in strength are adsorption of water and resultant swelling, pore water pressure, and strain. Thus, the most obvious and perhaps most significant of the environmental factors is water; its effects are increased shearing deformation or creep accompanied by loss of shearing strength.

As the ratio of shearing resistance to applied shearing stress for the potential failure surface approaches unity, the specimen approaches failure. As soon as the applied stress becomes about half the peak value, the clay is likely to deform or creep at constant shearing stress (1). The typical plot of deformation as a function of time indicates higher rates for higher constant stress levels; failure is indicated by a markedly rapid increase in deformation rate and a simultaneous decrease in stress.

The contributing factors to shearing resistance of soil are related in the following form of the Coulomb equation:

$$\tau_{ff} = \bar{c} + \bar{\sigma}_{ff} \tan \bar{\phi} \quad (1)$$

where τ_{ff} is peak shearing strength, which is known to depend on the effective stress in the soil (subscripts ff indicate "on the failure plane at failure").

The angle of internal friction $\bar{\phi}$, measured as the slope of the Mohr strength envelope, based on effective stresses, varies from zero for a saturated clay to about 30° for some fairly dry clays; $\bar{c}$, the cohesion or the minimum shear strength exhibited by the clay at zero normal stress, is measured as the intercept of the Mohr envelope on the shear axis. For partially saturated clay, τ_{ff} increases with increasing effective normal stress $\bar{\sigma}_{ff}$ on the failure plane at failure. The original Terzaghi equation for effective stress in fully saturated soils (1) provides the understanding of the principle and is expressed as

$$\bar{\sigma} = \sigma - u \quad (2)$$

Bishop and Blight (2) provide a general definition:

The effective stress ($\bar{\sigma}$) is that function of total stress (σ) and pore pressure (u) which controls the mechanical effects of a change in stress.

For example, if the total or applied stress in Equation 2 were held constant, the pore water pressure would have to change for the effective stress to change and to result in a change of strength.

The most convenient expression of the principle of effective stress for partially saturated clay is

$$\bar{\sigma} = (\sigma - u_a) + \chi(u_a - u_w) \quad (3)$$

where

- σ = total or externally applied stress,
- u_a = pore air pressure (relative),
- u_w = pore water pressure, and
- χ = a variable factor that represents the proportion of the suction ($u_a - u_w$) that contributes to $\bar{\sigma}$ (χ depends primarily on the degree of saturation and soil type and ranges from zero to one).

Bishop and Donald (3) have demonstrated the validity of Equation 3, concluding that it gives an adequate description of effective stress in a partially saturated soil. Analysis based on this equation is a means of studying the behavior of desiccated clays in contact with water that increase in degree of partial saturation while under a constant shearing stress. Solids in such a soil are under conditions of decreasing isotropic boundary or all-around stress because of decreasing negative pore water pressure or suction $-u_w$ (absolute value of $-u_w$ decreases), which occurs with increasing water content. Each increment of decrease in this positive surrounding stress induces increments of decrease in the effective normal contact stress $\bar{\sigma}$ between the particles undergoing shear without inducing any shear contact stress τ. These conclusions are based on analysis of the applicable static equilibrium equations. Decrease in suction has the effect on the Mohr plot of translating the effective normal stress circles from other loads toward the origin without change in shear stress.

EXPERIMENTAL PROCEDURE

Plane strain is essentially duplicated in the conventional direct shear apparatus, which has been widely used for studying the behavior of overconsolidated clays at very large shearing strains. In this study, the stress-controlled tests were used to observe the effects of different levels of applied shearing stress on shearing deformation of laboratory-desiccated clay during rewetting; therefore, the apparatus was modified to permit access of water to the specimen through the bottom porous plate from a source with a phreatic surface near the elevation of the potential failure plane. In addition, strain-controlled tests were performed on desiccated specimens that had been rewet to the w that corresponded to the w obtained at failure in the stress-controlled tests. This w was held constant during each test conducted at a constant rate of deformation.

Relatively pure kaolinite from the vicinity of Perth Amboy, New Jersey, was used. To obtain a dispersed structure (a structure with an approximately parallel orientation of particles), the bulk material was compacted in a split CBR mold by using an impact energy that approximated modified Proctor. The water content was near the liquid limit value of 55 percent. After removal from the mold and partial desiccation, specimens were trimmed to fit the shear box so that particle orientation (Figure 1) was predominantly perpendicular to the potential failure plane. Then the specimens were allowed to desiccate slowly under laboratory conditions to w ~ 1.5 percent (w_D). It is well known that, as w_D is approached, there is a rebound or small volume increase in kaolinite. This is probably caused by the reduction in effective contact area over which suction acts with decreasing w.

RESULTS AND DISCUSSION

To correlate and interpret the shearing stress and shearing deformation data, time and water content were chosen as bases for comparison. Each test was calibrated to zero time--the time when a change in water content was first detected in the potential failure plane--by a series of tests of specimens without shearing loads. In each test, a desiccated specimen was placed in the shear box and a small vertical load of 24 kPa (3.5 lbf/in^2) was applied to hold the top platen firmly in contact with the specimen. This procedure was duplicated in all other tests. Water was then permitted to enter through the bottom porous plate. Periodic removal and replacement of the specimen permitted determinations of water content. This provided the data for changes in water content in the potential failure plane as a function of time (Figure 2). The water content at which swell first occurred in this plane (w ~ 25 percent) was also detected. Because suction measurements cannot be made in the failure plane, time rates of wetting as a function of increasing water content were calculated to indicate the effect of suction in the soil (Figure 3). A suction versus water content curve for a similar clay is shown for comparison in Figure 4. The decrease in wetting rate with increase in water content is noted to represent a decrease in suction.

Results from the stress-controlled and strain-controlled tests are plotted on the same graph for comparison in Figure 5. In the former tests, it should be noted that different levels of constant

Figure 1. Physicochemical relations in partially saturated kaolinite.

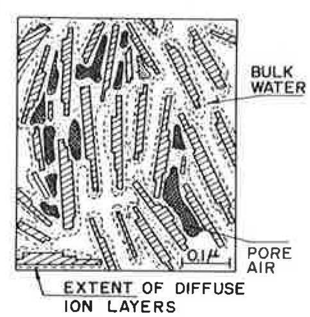
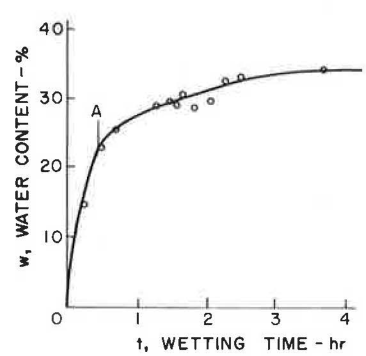


Figure 2. Increase in water content with wetting rate in the potential failure plane.



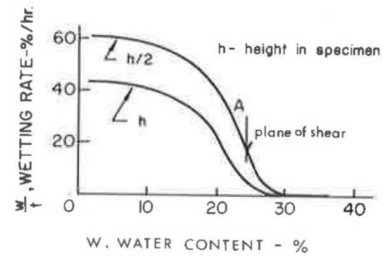
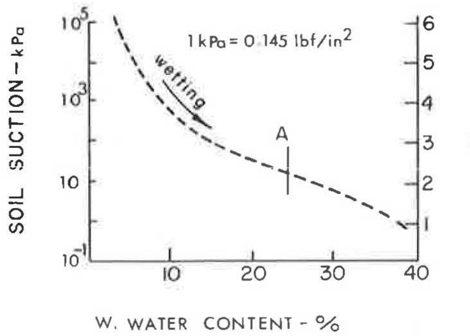


Figure 3. Decrease in wetting rate with increase in water content in the potential failure plane.

Figure 4. Decrease in suction with increase in water content for a pottery clay (hypothetical suction curve).



shear stress <320 kPa (<46 lbf/in^2) were applied to specimens before rewetting from ~1.5 percent. Two specimens at higher stress were rewet to w ~ 10 percent before loading and rewetting proceeded to failure. Failure was considered to occur at that water content at which the deformation rate increased rapidly for a given level of applied constant shear stress (Figure 6). The deformation versus time curves for selected specimens from Figure 5 are superimposed on this graph. For the selected specimens shown in Figure 5, the times at which initial deformation (or creep) occurred are superimposed on the graph, which relates levels of constant applied shear stress and times at which failure occurred (Figure 7). The dashed lines denote the elapsed time from initial deformation to failure. Higher constant stress levels produced initial deformations at lower w and at higher deformation rates. The approximately linear relation between applied τ and w at initial deformation for all specimens indicates that increases in w account for decreases in interparticle shearing resistance and corresponding losses of strength. Particularly at w < 23 percent (point A in this range), there is a rapid decrease in the factor of safety.

It should be noted that points on the curve before A are peak strengths obtained from strain-controlled tests. In such tests, the maximum stress occurs over such a short period of time that creep data were not obtained.

ANALYSES AND CONCLUSIONS

The conclusion reached in the earlier discussion of Terzaghi's effective stress equation (Equation 2) for saturated soils was that changes in pore water pressure control changes in effective stress when the total or applied stress is held constant. Equation 3 extends this concept to partially saturated soils and incorporates all the factors necessary to evaluate $\bar{\sigma}$, which may be introduced into Equation 1 to calculate shearing strength.

The following qualitative analysis of Equation 3 is made to assess the results obtained during rewetting of soil specimens in this study.

Assume that the small magnitude of applied stress used for vertical confinement is negligible in its effect compared with other contributions to shearing resistance in the highest strength specimens. On this basis, $\sigma = 0$ and Equation 3 becomes

$$\bar{\sigma} = -u_a + \chi(u_a - u_w) \qquad (4)$$

In the laboratory, the relative pressure in the air voids is such that $u_a = 0$; thus, Equation 4 becomes

$$\bar{\sigma} = \chi(-u_w) \qquad (5)$$

It was indicated in previous discussion that $-u_w$ would have less than its full effect on $\bar{\sigma}$ in most partially saturated clays ($\chi < 1$ for $S <$ ~85 percent). The present study shows that decreases in suction with rewetting of specimens are indicated by decreases in the wetting rate (Figures 3 and 4). No data were available on χ for this kaolinite although saturation S ranged from ~5 percent (w_D) to ~70 percent (at w ~

Figure 5. Change in shear strength with increase in water content in the potential failure plane.

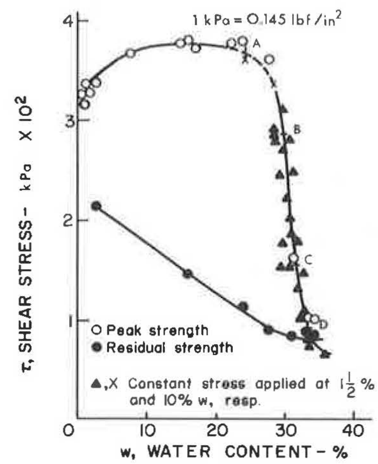


Figure 6. Decrease in deformation with rate of creep in the potential failure plane.

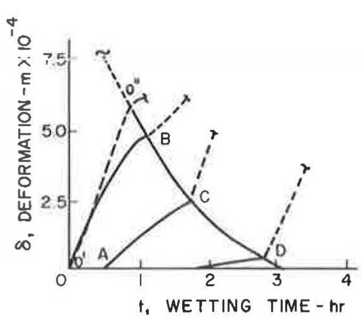
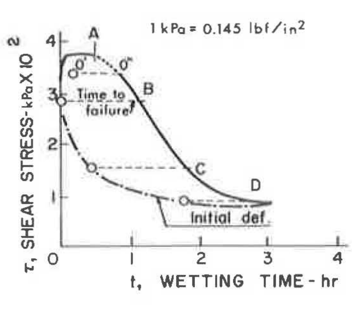


Figure 7. Change in shear strength with rate of strength loss in relation to creep time.



23 percent) to ~85 percent (at w_E); therefore, it is only possible to make a qualitative analysis in which higher wetting rates are taken to indicate higher suction and higher effective normal stress and so on.

Considering the implications of the preceding discussion and analyses in relation to the strength and deformation data obtained in the rewetting tests results in the following conclusions:

1. A decrease in negative pore pressure ($-u_w$) (i.e., a decrease in its absolute value) may be indicated by a decline in wetting rate.

2. High peak strengths at lower water contents indicate the benefits of densification from desiccation; stronger interparticle bonds may be caused in part by development of true cohesion. A portion of the shearing resistance probably also comes from friction and from forces required to partially reorient clay particles parallel to the potential failure plane.

3. Shearing resistance caused by stress σ from the small vertical load, interparticle bond strength, or cohesion $\bar{c}$ is reinforced, in effect, by an all-around normal stress caused by negative pore water pressures ($-u_w$), particularly at w < 23 percent. This reinforcement results from additional interparticle normal contact stress, which produces increased shearing resistance. These stresses are not transmitted through the grains and do not affect contact shear stress. When Equations 1 and 3 (with $u_a = 0$) are combined, these parameters are reflected in the following equation [note that $\bar{\sigma}_{ff}$ is replaced by the expression in brackets and that $\chi(-u_w)$ becomes positive for a negative pore pressure]:

$$\tau_{ff} = \bar{c} + [\sigma + \chi(-u_w)] \tan \bar{\phi} \qquad (6)$$

The higher magnitude of contact normal stress in specimens at w < 23 percent probably accounts for their sudden failure under shearing load (brittle behavior).

4. The increased shearing resistance caused by suction (negative pore pressure) is gradually lost with increasing w > 10 percent and rapidly lost at w > 23 percent with initiation of swelling pressure and actual swelling at w ~ 25 percent.

5. Loss of shearing resistance after swelling resulted partially from less difficulty in particle reorientation in the failure plane at higher w.

6. The Mohr envelope cannot be drawn exactly for the direct shear test because, in partially saturated soils, shear stresses are only known on a horizontal plane and normal stresses can only be inferred. A hypothetical failure envelope in this study may be curved as the origin is approached (rather than linear as indicated by Equation 1 or Equation 6).

7. The decrease in suction would have the effect of translating the applied normal stress circles of the Mohr plot toward the origin and thus making them effective stress circles.

8. As w_E is approached (~35 percent), a near-zero wetting rate is attained; the saturation level is near 85 percent. Swelling measurements are practically zero. Thus, shearing resistance is attributable to interparticle friction caused by the static vertical load.

9. The near coincidence of rewet, peak, and residual strength at w_E indicates that the clay particles are close to a face-to-face orientation, parallel to the potential failure plane, with $\bar{\phi}$ near the ultimate for sheet minerals.

10. The significance of a study such as this can be seen in Figures 5, 6, and 7. Control of water content is obviously necessary if imminent loss of strength at w > 23 percent and subsequent swelling are to be avoided. Below this w, for the shearing load on specimen C, FS > 2; below w ~ 15 percent, negligible creep or deformation occurs. For its lower shearing load, specimen D has FS ~ 4 at w < 23 percent. At higher w, FS diminishes to 2 at w ~ 30 percent when deformation is initiated. In both specimens, FS approaches 1 with increasing time (and water content) to failure. It may also be noted that initial deformation provides an advanced indication of impending failure.

11. Standard direct shear tests only provide data under strain-controlled conditions; peak strength results at the same rewet w are approximately the same as those obtained in the rewetting experiment.

Evaluation of the study as a whole suggests that the method would be useful in studies of soil stability. A slope subject to a rising groundwater table can be analyzed by simulating to a considerable extent in the laboratory the probable soil conditions of an arid region.

Another study is under way that is similar to this one except that pressures twice as high and even higher are applied. Preliminary results indicate that volume reductions occur immediately on rewetting and that shear strengths increase.

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