Effect of Temperature on Strength Behavior of Cohesive Soil

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The objective of this study was to determine the effect of tempera-
ture on peak shearing strength and creep behavior of two cohesive
soils and thus to characterize their behavior in terms of more fun-
damental parameters than is the current general practice. A simple
model for soil based on bonds at interparticle contacts and an equa-
tion for rate of deformation of a stressed soil mass are proposed
and shown to be consistent with observed soil behavior. The same
model and equation were found to apply to soil behavior in both di-
rect shear and creep tests.

The study was conducted on remolded, statically compacted spec-
imens of a highly plastic clay and a low-plasticity silt using a direct
shear machine. The machine was modified to permit control of
specimen temperature and for use as a controlled stress apparatus
as well as controlled rate of deformation.

●THE SOLUTION of engineering problems involving soils consists of three basic steps:
(a) determination of soil properties, (b) determination of changes in stress or in other
environmental conditions, and (c) prediction of the behavior of the soil when subjected
to the changes. Of these, the weakest is the understanding of the properties of soils.
Although testing methods have been refined to a high degree and behavior under known
conditions can be observed, the reasons for the behavior are most often a matter of
conjecture. In particular, the shearing strength of cohesive soils has been the subject
of much controversy and study from the earliest consideration of soil as an engineering
material. The late Donald W. Taylor (33) in 1948 wrote, ". . . no physical property
of cohesive soil is more complex than the shearing strength. This property depends
on many factors, and the individual factors are themselves complicated but, in addition,
they are inter-related to such a degree that it is extremely difficult to understand their
combined action."

One of the major reasons for the difficulty in understanding the behavior of cohesive
soils is that they interact with water and show colloidal behavior. At low water con-
tents clays exhibit high strength due to water films surrounding individual grains, but
their strength rapidly decreases as water content increases. The nature of these water
films and their influence on the strength behavior of clays is not well understood at
present. Since the viscosity and density of water and the nature of the diffuse double
layer vary with temperature, a study of the effect of temperature on the strength be-
havior of clays should aid in understanding fundamental properties of clays and in par-
ticular their shearing strength. The effect of temperature on the shearing strength of
cohesive soils and its use to characterize their behavior has been considered relatively
little to date.

THEORETICAL CONSIDERATIONS

A simple model of cohesive soil may be envisioned on the basis of bonds formed at
interparticle contacts due to an oriented water structure in the adsorbed water layers
adjacent to clay particles. Various concepts of the nature of the adsorbed water have
been proposed (15, 18, 20, 27), and there is general agreement that at least the first

few layers of water are strongly bound to the mineral surfaces. The high energy of adsorption of water (30) seems to preclude any direct mineral-to-mineral contact but, when two particles are in close proximity, their adsorbed layers intermix and form a continuous structure in this zone causing the particles to be bonded. Similar concepts of bonding were suggested by Terzaghi (34). In a study of Lilla-Edet clay, Bjerrum and Wu (5) found a peak in the curve of cohesion vs consolidation stress for stresses below the preconsolidation pressure. They suggested rigid bonds between particles of the undisturbed clay similar to a chemical cementation. Low (19) stated that clay and other minerals affect the water molecules to give a quasi-crystalline structure which possesses greater rigidity or viscosity than ordinary water. He suggested that the orderliness of this water structure decreased with distance from the mineral surface but that the influence may be from 200 to 300 Å from the surface. Exchangeable ions tend to disrupt this structure to a degree depending on their charge, size, and degree of dissociation. This was investigated by Leonards and Andersland (16) by means of controlled freezing of soils. As more of the free water was frozen the ions were concentrated in the adsorbed layer causing greater disruption of the water structure and subsequently a decrease in strength.

There is evidence to suggest that the formation of these bonds is time-dependent. Leonards and Rahmiah (17) found a quasi-preconsolidation pressure in specimens after a period of rest under a load in the normally consolidated range. This is probably related to the gain in strength during secondary consolidation reported by Crawford (7). The phenomenon of thixotropy in clays (8, 29) also suggests time-dependence of bond formation which takes place through a gradual orientation of the water molecules into positions of least free energy under the influence of the surface energy of the particles and hydrogen bonding between the water molecules.

Stresses applied to the particulate system are transferred through these bonds and deformation of the system occurs by distortion and breaking of bonds. Deformation of a clay mass under low stresses probably occurs largely through deformation of the bonds but, in a completely random system, there will probably be some particles or domains in such an orientation that any virgin deformation will cause some bonds to break. It is, therefore, doubtful that truly elastic deformation can occur except in cases where the system has been prestressed under the same stress components. This condition is approached in preconsolidation but even then a hysteresis effect generally occurs. Higher stresses will cause disruption to most susceptible bonds first and, as stresses become higher, all bonds in the zone of stress will tend to be disrupted, giving rise to plastic deformation. With deformation, new bonds will tend to form at points of contact but, since formation is time-dependent and reorientation of particles tends to occur with deformation, the new bonds will be weaker than those existing prior to deformation. The process of shear deformation will then consist of bond deformation, bond breaking, and bond formation; the latter processes are similar to the "jumping of bonds" suggested by Tan (32). Immediately following the breaking of a bond, an unstable condition exists that may be referred to as the "activated state" in which the "contact zone" between particles consists essentially of oriented water. The resistance of deformation would be very low, depending only on the viscosity of the water. An increase in temperature increases the thermal energy of the water molecules forming the bonds and thereby effectively lowers the resistance of the system to deformation.

Since the particles in a soil mass exist in a large variety of orientations and spacings, a statistical approach is reasonable in considering the stress-deformation behavior of the mass based on a particulate model. The breaking of a bond involves overcoming an energy barrier and then reaching a new equilibrium state. Thus, the rate at which bonds are broken and reformed under a given stress condition should be related to an activation energy and should be a function of applied stress. This is illustrated in Figure 1. The energy barrier is expressed as activation free energy, which is ΔG in the unstressed mass. Application of a shearing stress tends to reduce the barrier height by an amount that is some function of the applied stress τ.

If deformation involves units surmounting an energy barrier, it may be assumed that the rate of deformation is given by an equation of the form of the Arrhenius equation,
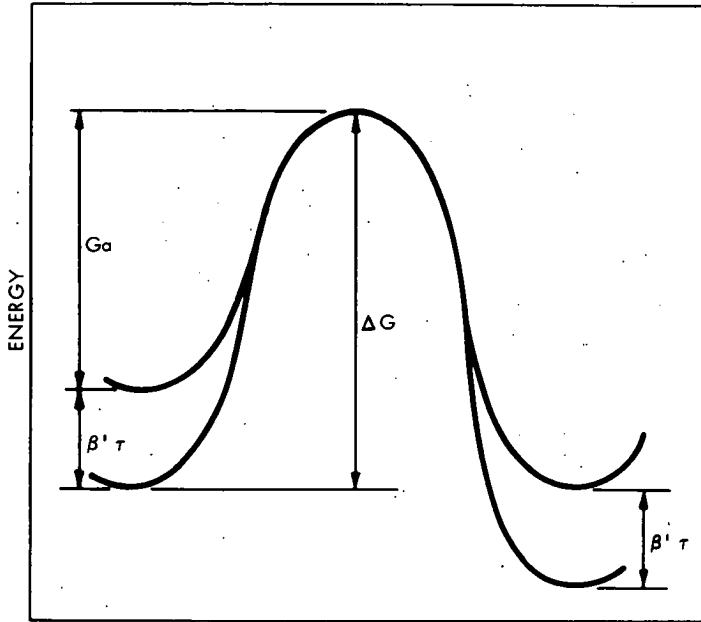
Figure 1. Energy barrier.

which suggests that the rate of deformation is proportional to the fraction of bonds with sufficient energy to surmount the energy barrier. Further, an applied shear stress, τ, will tend to decrease the effective barrier, and the amount of decrease may be represented by an energy term $\beta'\tau$. As illustrated in Figure 1, this gives

$$\Delta G = Ga + \beta'\tau \tag{1}$$

and the deformation rate may be expressed as

$$\dot{\delta} = A' \exp\left(\frac{-\Delta G + \beta'\tau}{kT}\right) \tag{2}$$

where $\dot{\delta}$ is the rate of deformation, A' is a frequency factor, and $\exp\left(\dfrac{-\Delta G + \beta'\tau}{kT}\right)$ is the Boltzmann factor representing the fraction of bonds with energy equal to or greater than Ga. From thermodynamics, the activation free energy may be written as

$$\Delta G = \Delta H - T\Delta S \tag{3}$$

where ΔH is the activation enthalpy, ΔS the activation entropy, and T the absolute temperature. If it is assumed that ΔS is independent of temperature, it may be included in the pre-exponential term. Further, letting

$$\frac{\beta'}{kT} = \beta \tag{4}$$

and combining Eqs. 2, 3, and 4 gives the basic equation for deformation rate,

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \exp(\beta\tau) \tag{5}$$

where the pre-exponential term A includes the entropy. In the substitution of Eq. 4 it is assumed that β is independent of temperature and this assumption is substantiated by the experimental data.

Equation 5 is essentially the same as the relation suggested by Dorn in 1954 on phenomenological grounds as applying to creep of metals. Dorn was concerned about the limitation that Eq. 5 implied a finite creep rate under zero shearing stress, and to overcome this he assumed that for low stresses the exponential stress function could be replaced by a power function τ^n, thus implying different mechanisms at high and low stresses. It would appear more appropriate, based on the proposed model, to consider that the mechanism remains constant regardless of stress level. When $\tau = 0$, Eq. 5 becomes

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \tag{6}$$

Substitution of experimental values of A, ΔH, and reasonable values of temperature for a clay-water system into Eq. 6 gives values of deformation rate so small as to be essentially zero. Further, when no stress is applied to the system, the deformation has no directionality and the meaning of Eq. 6 is probably related to bond changes in the system due to thermal agitation and may be related to thixotropic hardening of the material.

A similar approach to the problem of creep based on Eyring's rate process theory (9, 10, 11) has been proposed for metals (14), for bituminous materials (12), and for soils (2, 21, 23, 24, 25). In this approach, the rate of deformation is given by

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \sinh\left(\frac{\beta'\tau}{kT}\right) \tag{7}$$

When shearing stress is zero, the hyperbolic sine is zero and thus the creep rate is zero. However, for stresses of engineering interest, $\beta'\tau/kT$ will have a value greater than one and therefore the hyperbolic sine is reasonably approximated by the exponential as in Eq. 5. Kauzmann (14) suggested that for some cases the exponential relation may be more valid even when stresses are relatively low.

The energy term $\beta'\tau$ introduced into Eq. 1 may be interpreted as the mechanical energy (free energy) absorbed by a bonding unit in surmounting the energy barrier. This energy is expended only if the barrier is surmounted; otherwise it is stored in deforming a bond. From the dimensionless group $\beta'\tau/kT$ it can be seen that β' has dimensions of volume. This is interpreted to be the average volume of material containing one bond. As the number of bonds per unit volume of soil increases, the size of the zone of influence of a bond decreases. This may occur as a result of increased confining pressure or a decrease in temperature. This interpretation agrees with that of Andersland and Akili (2) who refer to β' as the volume of a flow unit. In the derivations based on rate process theory (11, 21), the energy term $\beta'\tau$ arises as the product $f\lambda/2$ where f is the shearing force per interparticle contact and λ is the distance between successive equilibrium positions. This is also in essential agreement, since it is reasonable to assume that the distance between successive equilibrium positions would be the dimensions of one side of a flow unit. From the foregoing, β' and thus β may be expected to vary with mineralogy and confining stress. As stated earlier, β is independent of temperature and thus β' must vary directly with the absolute temperature in conformity with this interpretation.

The pre-exponential term A in Eq. 5 was assumed to include the entropy of activation and appears to be most related to the soil structure. The relation between entropy and order in a system is well established in statistical thermodynamics. Factors that affect structure in a soil include consolidation conditions and water content. The known behavior of clays (4, 13) indicates that strength varies approximately linearly with consolidation stress and normal stress during shear and exponentially with water content at failure. Since a small range of water content variation was used in the experimental program, it was assumed for simplicity that the relation with water content was linear. Before these variables are included in the relation for deformation rate, the relation between strength and deformation rate must be considered.

A convenient form of Eq. 5 is obtained by taking logarithms:

$$\ln \dot{\delta} \; = \; \ln A \; - \; \frac{\Delta H}{kT} \; + \; \beta\tau \tag{8}$$

For the case of direct shear tests, the shear stress is the dependent variable and Eq. 8 may be written in the form

$$\tau \; = \; \frac{1}{\beta}\left(\frac{\Delta H}{kT} \; + \; \ln \dot{\delta} \; - \; \ln A\right) \tag{9}$$

Because of the method of derivation, Eq. 9 does not necessarily express a failure criterion but rather states that a given level of shear resistance is afforded by the soil when subjected to given levels of the variables on the right side of the equation. However, justification for the assumption that Eq. 9 does express a failure criterion is obtained from the experimental results, which indicate that the stress τ is closely related to the peak failure stress obtained from direct shear tests. With this assumption the effects of normal stresses and water content can be introduced into Eq. 9 to give

$$\tau m \; = \; \frac{1}{\beta}\left(\frac{\Delta H}{kT} \; + \; \ln \dot{\delta} \; - \; \ln M'\right) \; + \; \rho Pc \; + \; \mu Pns \; - \; \gamma w \tag{10}$$

where τm, the shear strength, replaces the shear stress τ; M' is a new constant; Pc is the consolidation stress; Pns is the normal stress during shear; w is the water content; and ρ, μ, γ are linear coefficients.

The effect of temperature of consolidation (as distinct from temperature of shear) on strength properties has apparently not been reported previously. Based on the experimental evidence, this variable was also considered to have a linear relation to strength. Introducing this into Eq. 10 gives the relation

$$\tau m \; = \; \frac{1}{\beta}\left(\frac{\Delta H}{kTs} \; + \; \ln \dot{\delta} \; + \; \alpha Tc \; - \; \ln M\right) \; + \; \rho Pc \; + \; \mu Pns \; - \; \gamma w \tag{11}$$

where Ts is the temperature of shear, Tc is the temperature of consolidation, and M is a constant.

The introduction of these variables into Eq. 9 may be considered a subdivision of the term A. The relationship for deformation rate may now be written in terms of all the variables by rearranging to give

$$\ln \dot{\delta} \; = \; \ln M \; - \; \frac{\Delta H}{kTs} \; + \; \alpha Tc \; + \; \beta\tau \; - \; \beta\rho Pc \; - \; \beta\mu Pns \; + \; \beta\gamma w \tag{12}$$

Equations 11 and 12 are the model equations proposed for soil deformation processes. It is reasonable to suppose that deformation of soil is governed by some mechanism (such as breaking of bonds) that is the same regardless of the method of stress application for the intensity of stress within reasonable limits. The equations cannot be proven correct but can be shown to fit the known facts of soil behavior as well as the observed experimental results.

The relationships of normal stresses and water content to shear strength were discussed when they were introduced. The undrained strength of soils has been shown to vary with the logarithm of time to failure while the drained strength variation with time to failure may be approximated by a logarithmic relation (4, 13, 21, 35). From the coefficients for shear temperature ($\Delta H/k$ for creep tests and $\overline{\Delta H/\beta k}$ for direct shear tests), values for activation enthalpy may be determined.

The proposed model equation bears considerable similarity to that proposed by Mitchell (21). Although derived differently, their basis lies in chemical rate theory and their final form consists of a sum of linear terms, which accounts for the variables known to affect deformation strength behavior.

TABLE 1

SOIL PROPERTIES

Property	Clay	Silt
Specific gravity	2.74	2.68
Liquid limit	89	34
Plastic limit	30	24
Percent passing No. 200 sieve	99.4	98.2
Percent finer than 2 microns (by hydrometer analysis)	80.4	25
Mineralogical composition (by X-ray diffraction)	Calcium mont-morillonite	Quartz Calcite Dolomite Montmorillonite Mica Kaolinite
AASHO classification	A-7	A-6

EXPERIMENTAL PROGRAM

Properties of Soils

The two Iowa soils used in the experimental program were an alluvial clay and a clayey silt. The properties of the soils were determined using standard procedures and are summarized in Table 1.

Soil Preparation

The soils were air-dried and pulverized to pass a No. 40 sieve, mixed with water to give compacted specimens of about 100 percent saturation, and stored for several weeks prior to testing.

Individual test specimens approximately $\frac{1}{2}$ in. thick were formed by removing a predetermined weight of the stored soil, molding it by hand into the shear rings and statically compacting it.

A total load of 1,000 lb was applied to the highly plastic clay and 500 lb to the clayey silt. The load was applied at a rate of about 0.08 in. per minute, maintained for 1 minute in the case of the clay and $\frac{1}{2}$ minute for the silt and then released at the same rate.

Shear Apparatus and Test Methods

The experimental study was conducted using a direct shear apparatus. Some of the advantages of this method of testing include simple temperature control, simple specimen preparation, rapid drainage during consolidation, and relatively simple laboratory techniques compared with triaxial testing. The direct shear apparatus provided a rapid means of obtaining a wide range of information.

The disadvantages include lack of drainage control during shear, nonhomogeneity of stress, and indeterminancy of stress components. Most of these factors are discussed in the ASTM Symposium on Direct Shear Testing of Soils (1) and by Sowers (31). It was suggested by Bishop (3) that the principal stress conditions at failure are not known precisely in shear boxes and that the error in undrained cohesion may be of the order of $(1 - \cos \varphi_e) \times 100$ percent, which is generally less than 5 percent for clay of high plasticity. The disadvantages of the method were well appreciated but were considered to be outweighed by the advantages.

Temperature control was achieved by constant-temperature water baths. The temperature of the sample was controlled by circulating the water through the shear rings, which were designed to permit circulation inside each ring. Specimen temperature was measured by a copper-constant thermocouple and a potentiometer recorder which could be read with a precision of 0.2 deg. After equilibrium was achieved, the temperature of the specimen could be maintained to within 0.25 C.

In all tests the shear temperature Ts was equal to or less than the consolidation temperature Tc. This was necessary because any increase in temperature above the
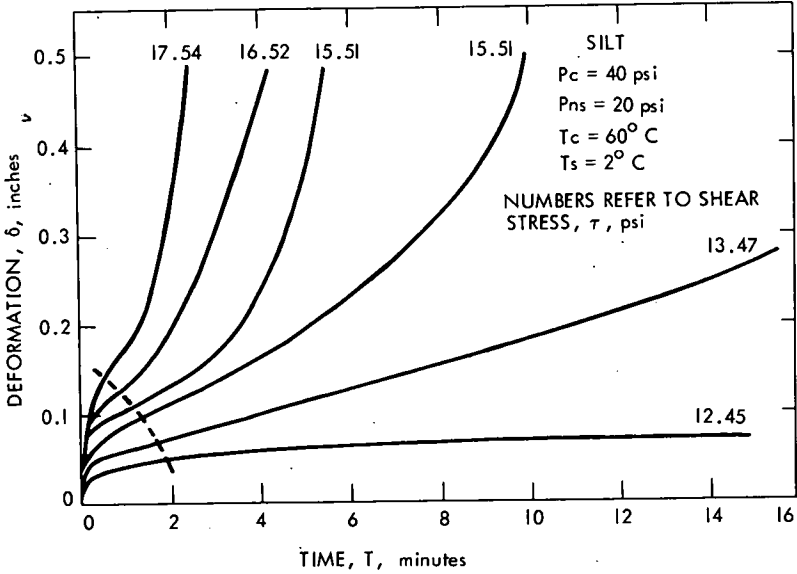
Figure 2. Sample deformation-time curves for silt.

consolidation temperature would result in further consolidation occurring under the higher temperature, with the result that strength would be the same as if the soil had been consolidated under the higher temperature.
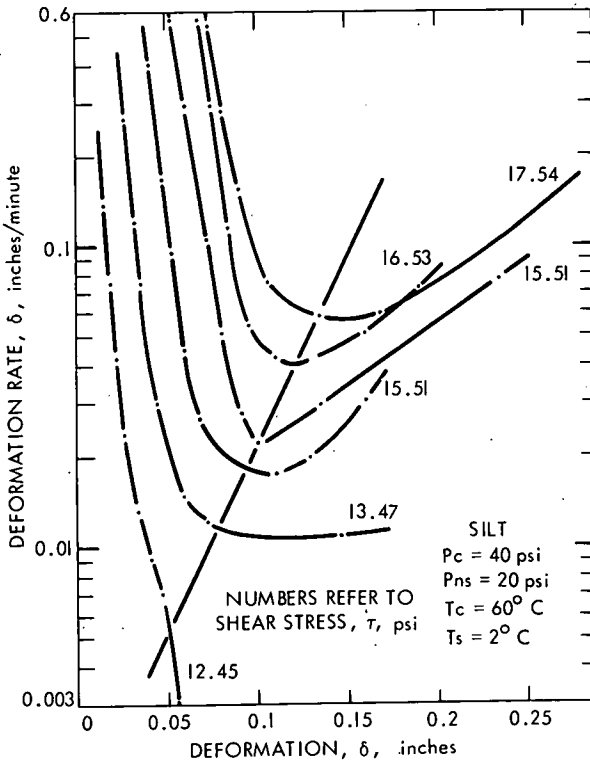

Figure 3. Sample deformation rate curves for silt.

The effect of temperature on the soil pore water pressure was illustrated by the behavior of the soil specimens subjected to temperature changes. On cooling, the volume of the specimen initially decreased and then, as the soil reached the new temperature and the rate of cooling decreased, the specimen reswelled about half of the original volume decrease. In all cases of cooling, negative pore pressure, measured with a transducer pore pressure device, was indicated and cavitation of the water occurred if the temperature change was larger and rapid. Conversely, heating the sample caused a volume increase and positive pore pressures that decreased with time. This behavior is similar to that reported by Mitchell and Campanella (22) and Campanella and Mitchell (6).

The laboratory tests performed in this investigation were of two basic types. The first type of test was a standard, controlled rate of strain, direct shear test. In this test, a given rate of deformation was imposed on the

specimen and the developed shear resistance was recorded at regular intervals of deformation. The second type of test was a creep test in which a given shear stress was applied to the specimen and the resulting deformation was recorded at regular time intervals and simultaneously by a strip-chart recorder. Each type of test was run on each soil type giving four series of tests. Within each series, the consolidation and shearing temperatures were varied in fixed increments.

ANALYSIS

In order to relate the results of the creep tests to the proposed model equation, it was necessary to choose some criterion for selection of the most significant value of deformation rate. The problem is complicated by the possibility of two different types of curves, as shown in Figure 2. Schoeck (28) stated that the basic equation for creep (Eq. 5) is valid ". . . only if there exists a unique and time independent relation between strain and structure." He further suggested that this condition can be satisfied in certain ranges of stress and temperature where (in metals) either no recovery takes place or the rate of recovery is fast compared to the rate of work hardening. In soils, these conditions are probably related to the rate of structure change and the rate of consolidation. An inspection of the basis of Eq. 5, i.e., the rate of passage of flow units over an energy barrier, indicates a validity only for the case of the second derivative with respect to time equal to zero, which is satisfied either for a steady-state condition or a point of inflection in the deformation-time curve. A second condition, which would seem necessary for the determination of valid parameters for soils, is that soil structure be the same for each specimen. For a given pretest history (consolidation temperature, stress, etc.), the structure of the soil at the beginning of the test should be essentially constant. Variations during the tests are probably related to deformation, shear stress, and temperature, with deformation having the primary influence. Mitchell et al (23) worked entirely with terminal creep curves and suggested that it was necessary to compare specimens at the same time after the start of creep. This approach appears inconsistent with the requirement of time-independence of Eq. 5.

In this study, the minimum points of the rate of deformation vs deformation curves, which are equivalent to points of $\ddot{\delta} = 0$, were joined by a straight line, which was then extrapolated to intersect the terminal creep curves and the points of intersection between the line, and the curves were assumed to satisfy the two criteria. These lines are shown in Figure 3 for silt. The points of intersection were transferred to the deformation time curves (Fig. 2) to show the relationship with these curves. For all the tests on clay the lines either were approximately vertical, indicating constant deformation, or had a large negative slope, indicating greater deformation at lower stresses. For all the tests on silt the intersecting line had
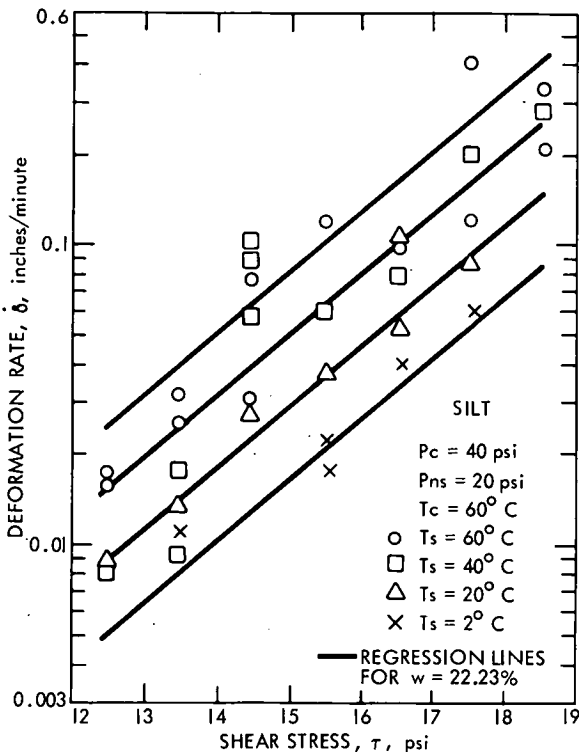

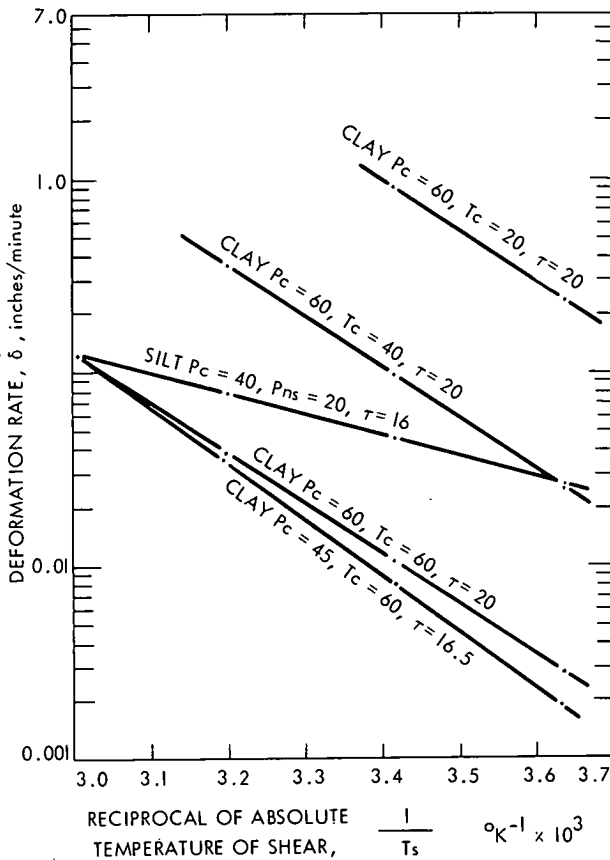Figure 4. Deformation rate variable with shear stress for silt.

SILT

Pc = 40 psi
Pns = 20 psi
Tc = 60° C
O Ts = 60° C
□ Ts = 40° C
△ Ts = 20° C
× Ts = 2° C

——REGRESSION LINES FOR w = 22.23%

DEFORMATION RATE, $\dot{\delta}$, inches/minute

SHEAR STRESS, τ, psi

Figure 5. Deformation rate variable with shear temperature.

a positive slope in the order of 3:1, as illustrated in Figure 3.

The points of intersection obtained from curves in Figure 3 gave the values of deformation rate plotted against shear stress, as in Figure 4 for silt. The slope of the straight lines on these plots is equal to the coefficient β while the differences in deformation rate for different shear temperatures at a given level of shear stress give the value of $\Delta H/k$. These relationships are plotted in Figure 5 for silt and clay.

The results of direct shear tests on clay are shown in Figure 6, where peak shear strength τm is plotted against the reciprocal of the absolute temperature of shear Ts. The different points show effects of different consolidation temperatures Tc.

Points on Figures 4 and 6 show considerable scatter from the expected straight lines. This scatter can be attributed to two main factors: general experimental errors including variations in initial water content and structure, and variations due to differences in water content during the test. The first factor could not easily be determined but may be due to differences in the internal structure of the soil specimens as a result of small differences in initial water content during compaction. Duplications of tests showed variations in shear strength that did not correlate well with the variations in density or water content, tending to confirm the hypothesis of variable internal structure. The water content of all specimens was determined at the end of the test. For given temperatures of consolidation and shear and a given normal stress, the variation of water content was generally less than 0.5 percent of the dry weight of soil, but variations over 1 percent occasionally occurred. However, differences in temperature of consolidation and, most particularly, of shear resulted in differences in water content of over 2 percent for the clay. A part of the variation may be attributed to nonuniform distribution of moisture in the stored batches of soil. Since the variation in water content was small, random, and partially masked by experimental variation, its effect was not readily distinguishable. However, as the proposed model equation had all linear terms, which were assumed to be independent, and there were a reasonable number of data points for most test series, the advantages of treating the data statistically were obvious and a multiple-regression program for an IBM 360 computer was used to treat all experimental data.

Regression Analysis

The dependent variables (τm in Eq. 11 and $\ln \dot{\delta}$ in Eq. 12) were regressed on the applicable independent variables to obtain values for the linear coefficients in the equations. It was possible to treat the test results by considering groups of tests in which
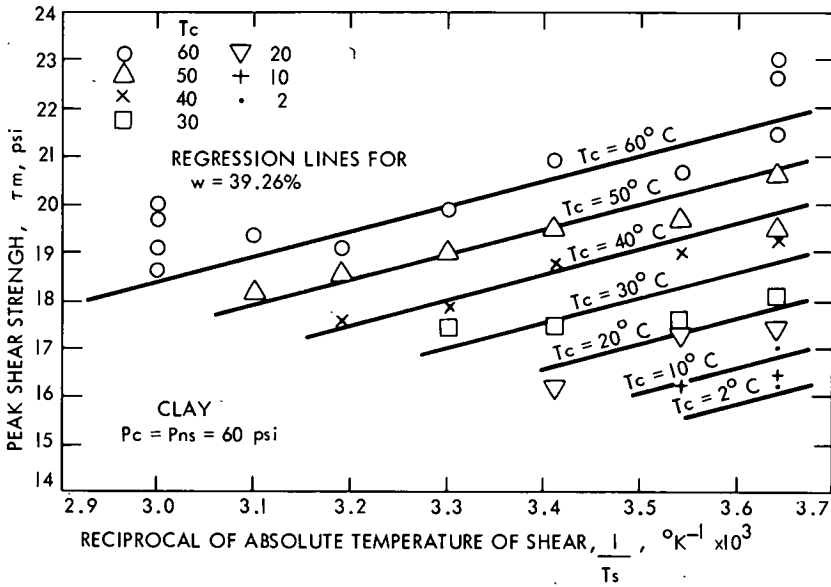
Figure 6. Variation in shear strength of clay with temperature of consolidation and shear for series B4.

TABLE 2

COMPARISON OF COEFFICIENTS OF EQUATION 11 OBTAINED BY REGRESSION FOR CREEP AND DIRECT SHEAR TESTS ON CLAY

Series	Tc (deg C)	Ts (deg C)	Pc (psi)	Intercept	$\frac{\Delta H}{\beta k}$	$\frac{1}{\beta}$	ρ	$\frac{\alpha}{\beta}$	γ	Avg. w	Avg. Gm
				(a) Expected Coefficients From Creep Tests							
A1	60 40 20	60 40 20 2	45	-22.629	4.843	0.739		0.084	0.595	40.70	1.287
A2	60 40 20	60 40 20 2	60	-25.290	5.321	0.894		0.099	0.636	39.86	1.324
A1 & A2	60 40 20	60 40 20 2	45 60	-14.771	5.089	0.829	0.182	0.093	0.625	40.36	1.307
				(b) Experimental Coefficients From Direct Shear Tests							
B1	14 59.5	14 58.8	15 120	-15.901	12.288	8.062	0.144	0.138	0.733	35.85	1.407
B2	65	1.5 65	45	-18.324	7.977				0.520	38.69	1.373
B1 & B3	14 60	2 60	15 120	-5.022	7.361	1.173	0.148	0.074	0.467	38.21	1.370
B4	2 60	2 60	60	-16.258	5.303			0.098	0.501	39.26	1.346
B5	20 60	2 60	60	-30.714	6.366			0.094	0.936	39.26	1.340
B1-B5 incl.	2 65	1.5 65	15 120	-11.898	8.267	1.934	0.112	0.143	0.654	38.76	1.360

TABLE 3

COEFFICIENTS OF EQUATION 12 OBTAINED BY REGRESSION FOR CREEP TESTS ON SILT USING COMBINED GROUPS OF TESTS

Series	Tc (deg C)	Ts (deg C)	Pc (psi)	Pns (psi)	Intercept	$\frac{\Delta H}{k}$	β	$\beta\rho$	$\beta\mu$	$\beta\gamma$	Avg. w	Avg. Gm
C2	40	40 2	60	60	-13.354	3.975	0.236			0.864	20.76	1.684
C2	40	40 2	60 40	40	-32.120	4.152	0.206	0.016		1.866	21.50	1.650
C2	40	40 2	60 40 20	20	-35.088	3.752	0.348	0.038		1.825	22.41	1.644
C2	40	40 2	60 40 20	60 40 20	-22.876	3.555	0.308	0.043	0.003	1.294	21.81	1.659
C1	60 40 20	60 40 20 2	40	20	-14.257	2.460	0.464			0.547	22.23	1.690
C1 & C2	60 40 20	60 40 20 2	60 40 20	60 40 20	-13.465	2.663	0.407	0.067	0.047	0.749	22.00	1.676

some of the parameters were constant and regressing on the remaining variables. Further, all similar tests on a given soil could be treated as one group and regression carried out on all the variables. This leads to a large number of possible combinations and results, examples of which are given in Table 2 for clay and in Tables 3 and 4 for silt.

It was previously proposed that the deformation mechanism of soil should be consistent regardless of the method of stress application. This would be substantiated by agreement of the coefficients of Eq. 11 obtained from direct shear tests with those of Eq. 12 obtained from creep tests. The comparison can be made by multiplying the direct shear coefficients or by dividing the creep coefficients by their respective values of β. The latter method was used since the β values obtained from the creep tests were much better defined than those from direct shear tests, where a relatively small range of deformation rate was used, and these variations were largely related to other changes in procedure or technique. This comparison for clay is given in Table 2, which shows the converted coefficients for two series of creep tests run at different normal stresses and the coefficients for these two series combined. In the combination, the normal stress Pc was included as an additional variable. The lower part of the table gives the experimental coefficients obtained from five series of direct shear tests on clay and some combinations of these series. The tables include the range of variables used and average values of water content w and mass specific gravity Gm for the groups.

TABLE 4

COMPARISON OF COEFFICIENTS OF EQUATION 11 OBTAINED BY REGRESSION FOR CREEP AND DIRECT SHEAR TESTS ON SILT

Series	Tc (deg C)	Ts (deg C)	Pc (psi)	Pns (psi)	Intercept	$\frac{\Delta H}{\beta k}$	$\frac{1}{\beta}$	ρ	μ	γ	Avg. w	Avg. Gm
(a) Expected Coefficients From Creep Tests												
C1 & C2	60 40 20	60 40 20 2	60 40 20	60 40 20	-33.084	6.543	2.457	0.165	0.115	1.840	22.00	1.676
(b) Experimental Coefficents From Direct Shear Tests												
D1	60 40 20	60 40 20 2	60 40 20	60 40 20	-30.346	4.832	1.240	0.123	0.359	1.769	21.90	1.659

Table 3 gives the coefficients of Eq. 12 obtained by regression for creep tests on silt. The first three lines give coefficients for tests of series C2 combined according to normal stress during the shear phase of the test (Pns); the fourth line gives coefficients for all tests of series C2 combined; the fifth line gives coefficients for series C1 in which the consolidation pressure Pc was constant at 40 psi and the normal stress during shear Pns was constant at 20 psi; the sixth line gives coefficients for all creep tests on silt combined with the regression conducted on all independent variables.

Table 4 gives a comparison of coefficients of Eq. 11 for creep and direct shear tests on silt obtained similarly to those for clay. The expected coefficients from creep tests were obtained from the sixth line of Table 3.

Regression lines shown in Figures 4 and 6 were plotted using the coefficients obtained from the group of tests performed at the corresponding normal stresses and using the average water content for that group. Thus, water contents different from the average cause the points to fall above or below the regression lines and illustrate the effect of water content.

DISCUSSION OF RESULTS

The coefficient β, although somewhat variable for the smallest group of tests based on Tc and Ts, became quite constant for the larger groupings of tests on the clay and for the silt tests at fixed values of normal stresses. There was, however, a significant difference in the values for the two sets of tests performed on the clay at different normal stresses (Table 2) and in the values obtained at different normal stresses on silt (Table 3). In both cases the values of β were lower at the higher normal stresses. These results indicate a possible relationship between the coefficient β and the normal stress during shear Pns. However, tests on the clay were performed at only two levels of normal stress, which was the same for the consolidation and shear phases of the tests. The higher normal stresses used during shear for tests on the silt gave the poorest results in terms of variability. These tests appeared to be very sensitive to variation in water content and test techniques. Because of these limitations the relationship between the coefficient and normal stress during shear is not well defined. However, the decrease in β with increased normal stress is in agreement with the proposed interpretation of β' as the volume of a flow unit. An increase in normal stress would tend to increase the number of bonds and thus decrease the average volume of influence of a bond. A similar reasoning may be used to account for the observation that the value of β for silt is about one-fourth that for clay. The value of β' for clay under 60 psi normal stress is calculated to be 0.6711×10^{-18} cm^3, which is equivalent to the volume of a cube 87.6 Å on a side whereas, for silt under the same normal stress, the value of β' is 0.142×10^{-18} cm^3, or the volume of a cube 52.1 Å on a side. In the case of the clay, the volume is on the order of size of a clay particle while for the silt the volume is considerably smaller, which implies that the bonds and adjacent equilibrium positions occur between asperities on the surface of the silt particles.

The variation of β between the smallest groups of tests did not show any consistent relationship with variables other than normal stress. Specifically, no consistent relationship with temperature could be discerned and this observation substantiates the assumption made in deriving Eq. 5. Regardless of other possible functional relationships of the coefficient β, all the creep test results indicate a linear relation between the logarithm of deformation rate and shear stress, in accordance with Eq. 5.

Values of the coefficient $\Delta H/k$ or $\Delta H/\beta k$ (and hence activation enthalpy) determined from the regression analyses appear to be reasonably constant for each group of tests having the same normal stress. However, it does increase slightly with increased normal stress, indicating an increase in bond strength with a decrease in particle spacing. Mitchell et al (23) report a decrease in activation energy with increased deviator stress. The energy barrier should be related to bonds in the oriented water and the strength of these bonds would be expected to increase slightly with decreased particle spacing due to surface induction.

The value of activation energy was found, from the values of $\Delta H/k$ and $\Delta H/\beta k$ given in the tables, to range from 12 to 21 kilocalories per mole for the clay and from 5 to 8 kilocalories per mole for the silt. For both soils the higher values were obtained from the direct shear tests.

The magnitude of activation enthalpy obtained for the silt was about half that obtained for the clay. This gives an indication of the effect of particle size and, indirectly, of mineralogy. The bound water layer associated with calcium montmorillonite would be more oriented and relatively thicker than the layer associated with silt. This suggests a stronger bond and higher activation energy for the clay. It would be expected that activation energy for other minerals would reflect their relative surface activity. The activation enthalpy measured for the silt is only a little higher than that for unbound water, which suggests that the bonds are not strongly influenced by the surface energy of the particles.

Under a normal stress, such as during consolidation, the contact zones between particles or domains are probably subjected to localized shear stresses, and a breaking of bonds would then occur to permit particle reorientation accompanying the deformations of consolidation.

From this model it is reasonable to consider consolidation as a rate process having a mechanism similar to that proposed for shear deformation. This approach has been suggested by Wu et al (36), who considered consolidation on the basis of rate process theory. The effect of temperature on consolidation is related to an increase in thermal energy, which permits more rapid passage of flow units over the energy barrier and allows deformation to progress more rapidly. This increase in the rate of consolidation is also related to the decreased viscosity of the water, which permits more rapid drainage. These effects have been reported by Paaswell (26) and others. In the case of clay, the consolidation temperature shows the additional effect of increasing the bonding, as indicated by higher values of $\Delta H/k$ similar to the effect of increased normal stress. The silt, however, showed no significant effect due to temperature of consolidation. This behavior further supports the suggestion of a weak influence of particle surface energy in the case of silt.

The coefficient of water content $\beta\gamma$ varied quite erratically. This is not surprising in view of the small and random variation in water content between the samples, but unfortunately leaves the coefficient poorly defined. The average value of $\beta\gamma$ for the creep tests on clay with 45 psi normal stress was 0.8, and with 60 psi normal stress was 0.7; but the value of γ in both cases was approximately 0.6, which is of the same order as found from the direct shear tests. The creep tests on silt with Pc = 40 psi and Pns = 20 psi gave a value of $\beta\gamma$ of about 0.55, while the average for all creep tests on silt was 0.75, giving an average value of γ = 1.8, which was the value obtained as an average from all direct shear tests. From this it may be inferred that water content variations of a given magnitude have more effect on silt than on highly plastic clay, a fact that is well recognized and that may be explained on the basis of the large differences in specific surface area between these materials.

As previously discussed, the coefficient β is probably related to the normal stress during shear and the coefficient $\Delta H/k$ is probably related to consolidation stress; however, the results recorded (for example, in the top three lines of Table 2) indicate that these variations were of insufficient magnitude to account for the increased strength with increased normal stresses. Therefore, in the linear equation, significant coefficients for Pc and Pns where also obtained.

The general agreement between the values of the coefficients obtained from creep and direct shear tests tends to confirm the postulate that there is one consistent deformation mechanism for soils that is independent of method of load application. These confirmations were obtained as a result of performing the creep tests at stresses that were on the order of the peak shear strength of the soil. These stresses were higher than those generally used for creep testing and gave minimum points of deformation rate rather than terminal creep curves.

CONCLUSIONS

The experimental part of the study, consisting of 434 creep and direct shear tests on two different soil types, yielded the following observations:

• Creep tests show a linear relationship between the logarithm of deformation rate and shear stress, and the proportionality coefficient is linearly related to the volume of the flow unit and may be related to the normal stress.

• Activation energy values obtained from creep tests are approximately equal to those obtained from direct shear tests and for the clay are of the same order of magnitude as other reported values. The values of activation energy obtained for the silt are about 40 percent of those obtained for the clay.

• The coefficients relating deformation rate and shear strength to water content were also found to agree quite well. The values for silt were about three times larger than the values for clay, as would be expected due to the differences in specific surface area between these soils. Because of the narrow range of water content values used, the relationship was assumed to be linear and the coefficients were rather poorly defined for smaller groups of tests.

• The agreement between the coefficients obtained from the two test procedures tends to confirm the hypothesis of a single deformation mechanism that is independent of test procedure.

This study of the effect of temperature on the shear strength and creep behavior of soils has yielded a model equation in terms of simple parameters that reasonably characterizes the deformation and strength behavior of the soils. Modifications of the model equation may be required as further investigation reveals the interdependencies of some of the variables and, specifically, the functional relationship of water content. The test results were analyzed by multiple regression, which aided interpretation of the variability due to differences in water content.

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