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Effect of temperature on strength of soils

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EFFECT OF TEMPERATURE ON STRENGTH OF SOILS

by

Calvin Athelward Noble

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In Charge of Major Work

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INTRODUCTION

The solution of engineering problems involving soils consists of three basic steps: 1) determination of soil properties, 2) determination of changes in stress or in other environmental conditions and 3) 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 (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".

Although the large amount of work done in the past two decades has helped to shed light on the problem, many more questions have been raised and, today (1968), Taylor's statement is still applicable. 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 contents clays exhibit high strength due to water films surrounding individual grains but their strength rapidly decreases as water content is increased. 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 behavior of clays should aid in the understanding of fundamental properties of clays and in particular 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.

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

The study was carried out on remolded, statically compacted specimens of a highly plastic clay and a low plastic 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 a controlled rate-of-deformation apparatus.

LITERATURE REVIEW AND THEORY

The starting point for most discussions on the shearing strength of soils is with the Coulomb equation:

$$s = c + p \tan \phi \tag{1}$$

wherein the shearing strength, s, is equal to the sum of cohesional, c, and frictional, $p \tan \phi$, components. Terzaghi (1943) modified this to

$$s = c + P_e \tan \phi \tag{2}$$

where effective stress, P_e is equal to $p - u$, the total normal stress minus pore water pressure. For granular soils, the cohesional term may be omitted and the strength is given by the product of the effective normal stress, P_e, and the coefficient of internal friction, $\tan \phi$ where ϕ is referred to as the angle of internal friction. The same relation has been shown to hold for cohesive soils when the normal pressure is expressed in terms of effective stress. This has been well documented for normally consolidated clays, e.g. by Bishop and Bjerrum, (1960) and more recently for overconsolidated clays by Gibbs, (1965). However, clay soils tested without allowing time for drainage of water or readjustments of water structure at particle contacts will show a stress-independent component of strength referred to in soil mechanics terminology, as cohesion. Based on the fact that cohesion is proportional to the consolidation pressure, Equation 2 may be put in the form:

$$s = Pc \tan \phi c + Pns \tan \phi r \tag{3}$$

where Pc is the consolidation pressure, ϕc is the friction angle for consolidation, Pns is the normal pressure during shear and ϕr is the angle of friction for overconsolidation. Hvorslev (1960) refers to this equation as the Krey-Tiedemann failure criterion.

Bishop and Bjerrum (1960) stated, "...the difference between the shear characteristics of sand and clay lies not so much in the difference between frictional properties of the component particles as in the very wide difference--about a million times--in permeability". This view is expressed to show the general validity of the effective stress concept and appears valid in view of the above evidence supporting the Coulomb equation. However, it overlooks the fact that in many practical cases involving clays, time does not permit the full development of effective stresses through drainage or swelling and an apparent difference in behavior results. Further, a difference in behavior would be expected due to the difference in specific surface area between granular and cohesive soils of about a million times. Thus, neglecting differences in surface energies which should be in the same order of magnitude for silicates, the higher specific surfaces of clays influence a much greater proportion of the soil water. This adsorbed water largely accounts for the cohesive and plastic properties of clays.

Friction concepts

Application of the Bowden and Tabor (1950) concepts of friction between solids to soils indicates the reason for the macroscopic behavior. The real area of contact between solids is very small in comparison to the total area and is related to the applied load on the contact area. Surfaces of solids are relatively rough compared to molecular dimensions and the load is carried by the highest points of irregularity. Stresses are high at these points of contact and plastic deformation of the material occurs until the area of contact is large enough to carry the load. This may be expressed as:

$$A = \frac{P}{Py} \qquad (4)$$

where A is the area of contact, P the load and Py the yield stress of the material. In the case of metals, if the surfaces are "clean" (no oxide or adsorbed films), the high contact pressures cause localized welding or adhesion to occur. However, if release of load results in elastic strain release sufficient to rupture the bonds, no permanent adhesion occurs. This occurs with harder metals while softer metals may form permanent bonds. The bond strength between two clean metals in contact is given by Goodzeit (1959) as related to their relative atomic size, crystal structure, electrochemical activity, valency and type of forces that hold the atoms in their lattices. If S is the mean tangential stress necessary to shear the junctions, the total force required to shear all junctions is given by AS = F, the total frictional force. Combining with (4) gives

$$F = \frac{PS}{Py} \qquad (5)$$

which expresses the basic law of friction viz. that the frictional force is dependent only on the load and is independent of the total area of the surfaces.

Granular soils tend to act similar to hard metals so that they obey the law of friction but, if load is released, elastic rebound breaks bonds at the points of contact and there is no frictional resistance under zero normal load. With clays however, bonds probably form in the adsorbed water layers adjacent to the particles rather than between atoms of the particles, and these bonds, similar to the case of soft metals, are not broken when the load is released. Thus the clay will exhibit a shearing resistance under zero load, i.e. cohesion. It is generally agreed that all shearing resistance in soil is of a frictional nature (Rosenqvist, 1955; Lambe, 1960; Trollope, 1960; Crawford, 1963) with cohesion due to internal attractive forces. However, in view of the above concepts of friction it could just as well be stated that all shearing resistance is due to cohesion (or more strictly, adhesion).

Physico-chemical concepts

The composition of the mineral phase of soil systems is primarily silicates with generally smaller amounts of sesquioxides and other minerals. The coarser fractions of soils may be composed of any or all the basic silicate structures which include Nesosilicates (single tetrahedra), Sorosilicates (disilicates), Cyclosilicates (rings), Inosilicates (chains), Phyllosilicates (layers) and Tektosilicates (3-D network). The crystallographic principles which apply to silicate minerals was reviewed by Hauth (1951). The finer fractions of soils are mainly composed of phyllosilicates which include the clay minerals. A comprehensive outline of the crystal structure of clay minerals is given by Grim (1953). In brief, clay minerals may be described as composed of various combinations of silica sheets, (A), and either gibbsite or brucite sheets, (B). These sheets are combined in different proportions resulting in ABA, (ABA)B and AB type minerals. The differences in mineral species arises as a result of different cations substituting in different positions and proportions in the crystal lattice (isomorphous substitution) and as a result of difference in stacking between adjacent layers in the particle unit. Particularly in the finer fractions of soils and in clays the properties of the system are largely governed by the mineralogical composition. Other constituents such as organic matter, exchangeable ions and soluble salts and the texture of the material also have important effects on the soil properties.

The particle size of clay minerals is generally taken to be less than two microns. This falls in the range of colloidal sizes, and clays display colloidal behavior in dilute suspensions. The principles of colloidal behavior aid in understanding of clay properties. The behavior of colloids is related to the electric charge associated with the surface of the colloidal particles. Two ways in which a particle may be charged are given by van Olphen (1963):

i) imperfections within the crystal lattice (includes isomorphous substitution) and

ii) preferential adsorption of specific ions on the particle surface (potential determining or peptizing ions).

The net charge of the particle is balanced by an accumulation of ions of opposite and equivalent charge in the adjacent solvent forming a diffuse layer. An extended treatment of the diffuse double layer is given by Verwey and Overbeek (1948) and a more abbreviated treatment considering clay minerals is given by van Olphen (1963). Because of its limitations, the double layer theory is not directly applicable to quantitative evaluation of soil behavior. It does, however, afford useful qualitative explanations of some soil behavior related to the electrolyte concentration of the pore water.

The double layer theory gives equations for the repulsive potential between adjacent particles which may be combined with van der Waals' attractive potential and Born repulsion to obtain net potential between particles (van Olphen, 1963).

The adsorption of water on clay particle surfaces takes place to reduce the surface energy of the particle. It was proposed by Hendricks and Jefferson (1938) that inter-layer water had a hexagonal structure superimposed on the hexagonal structure of the silicate layers. Iler (1955) suggests evidence that the first molecular layer of water might be bonded to the surface oxygen of the silicate structure in the form of hydroxyl groups. The experimental work on adsorbed water was reviewed independently by Low (1960) and Martin (1962). Low concluded that adsorbed water had a high degree of structural order and rigidity, making it more resistant than ordinary water to shear and normal stresses. Martin suggested a two-dimensional fluid model in which the water is fluid parallel to the solid surface but rigid normal to it. Rosenqvist (1959) showed that freezing of a clay involves a much smaller energy change in the water phase than the freezing of pure water and he concluded that the free energy of water in a clay lies between that in free water at room temperature and ice at a low temperature. He later (1962) discussed the nature of adsorbed water and suggested structure intermediate between normal water and ice. Terzaghi (1941) suggested rigid bonds between clay particles due to "rigid water" (adsorbed water) surrounding the particles. In a study of Lilla-Edet clay, Bjerrum and Wu (1960) found a peak in the curve of cohesion versus consolidation stress for stresses below the preconsolidation pressure. They suggested rigid bonds between particles of the undisturbed clay similar to a chemical cementation. Leonards and Ramiah (1960) found a quasi-preconsolidation pressure in specimens after a period of rest under a load in the normally consolidated range which would indicate a time-dependent bond formation. Low (1962) 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 total 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 point was investigated by Leonards and Andersland (1960) 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 a decrease in strength. A further argument in favor of the concept of time-dependent bonds in the adsorbed water, is the energy of adsorption of water to mineral surfaces. Senich et al. (1967) showed that expansion energies exerted by a calcium montmorillonite-water system are of the order of 440 kg/cm^2 at zero separation and 40 kg/cm^2 at a separation due to four layers of water. Since even the latter value is relatively high compared to most natural or applied pressures on soils, it is doubtful that any direct mineral-to-mineral contact really exists in clay soils at any but the lowest moisture contents. The time-dependence of bond formation is further supported by such phenomena as thixotropy (Eichler and Kazda, 1965) and the gain in strength during secondary consolidation (Crawford, 1964).

Soil structure

The formation of bonds and the behavior of a clay soil is related to the soil structure. "Structure" is defined by Martin (1966) as, "...the combination of the geometrical arrangement of particles and the forces operating between them". He further defines "fabric" as "...the geometrical arrangement of particles and associated voids", and as, "...the component of structure most amenable to measurement". Suitable techniques for the measurement of soil fabric are still limited and many of the present concepts are based on inference from soil behavior. Available techniques include the petrographic microscope, electron microscope and X-ray diffraction. In each case the quantitative determination of soil fabric involves two problems:

(1) Preparation of specimens for examination which have the original fabric preserved, and (2) the examination of some property which expresses the geometrical arrangement of the crystalline particles. Advances in understanding soil structure have developed as a continuing series of postulates and confirmations by measurements.

Terzaghi (1925, 1941) presented some of the earliest concepts of soil fabric in engineering. He suggested that clay particles were small enough to be influenced by molecular forces and could develop a "honey-comb" fabric. Casagrande (1932) expanded these ideas to include mixed-particle size materials which he suggested could build up a random flocculent structure. Goldschmidt (1926) suggested that clay properties were the result of crystalline minerals surrounded by adsorbed water films which could adhere to one another to form a "cardhouse" fabric as exemplified by sensitive marine clays. Lambe (1953) reasoned that the colloidal properties of clays could result in a cardhouse type of fabric in a marine or high electrolyte concentration environment while fresh water or low electrolyte concentrations would result in more parallel orientations.

Mitchell (1956) replaced water in moist clays with a polyalcohol and thus obtained sufficient rigidity to prepare thin-sections for study under the petrographic microscope. He studied marine, fresh water and remolded clays and showed that more preferred orientation was present in fresh-water clays than marine clays and that remolding caused a higher degree of orientation. He obtained semi-quantitative relationships between fabric and engineering properties. Smart (1967) illustrated the ambiguity in interpretation of electron micrographs of replicas.

The influence of structure on the properties of compacted clays was discussed by several investigators (Seed and Chan, 1957, 1959; Lambe, 1958a, 1958b; Seed et al., 1960). Tan (1959) proposed an edge-to-face type of structure for clays which Rosenqvist (1959) confirmed by electron micrographs of marine clays. Wu (1958) showed different degrees of orientation in glacial lake clays. Trollope and Chan (1960) argued that "packets"[1] of oriented clay particles could develop by remolding and weathering and suggested that shear strength was based on friction between these packets. Electron microscope studies of Aylmore and Quirk (1962) confirmed the existence of these packets in natural clays. Based on about a hundred stereoscopic electron micrographs of unweathered marine clays, however,

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[1] Synonyms include, "domains", "tactoids", "clusters", and "turbostratic arrangement" and consist of aggregates of oriented platelets between which the orientation may be random (Meade, 1964).

Rosenqvist (1962) reported that he had not observed anything resembling a domain type of structure.

Some clarification of terminology would be desirable in reference to the units involved in soil fabric. The smallest unit in clays is a platelet of unit cell thickness which, in the case of montmorillonites, is about 9.6 Å. However, except in the case of fully dispersed sodium montmorillonite, it is doubtful that individual platelets exist in soils; the main evidence being X-ray diffraction studies which would fail to reveal C-axis dimensions in the absence of repeated units. Thus the smallest natural unit would be a pseudo-crystallite composed of stacks of 10's of platelets. These pseudo-crystallites are probably the "particles" referred to by most writers. A domain would then be a large particle or a stacking of particles in parallel orientation to a degree where this stacking is observable with a petrographic or electron microscope.

Compacted clays show somewhat different fabrics depending on the method of compaction and the compaction water content. At higher compaction water contents, more preferred orientation can be expected while at water contents below optimum, a more random fabric is generally believed to occur. It has been suggested that kneading compaction imparts greater preferred orientation than static compaction (Seed and Chan, 1959) but the recent work of Sloane and Kell (1966) would tend to cast some doubt on this. Their studies of compacted kaolinite and those of Smart (1967) on remolded compacted and sheared kaolinite using the electron microscope showed that packets formed under all compaction conditions studied. Kneading and impact compaction resulted in oriented trajectories of particles while static compaction tended to orient the packets perpendicular to the direction of applied load. An oriented domain structure formed during consolidation and particles were oriented parallel to local failure planes. Hvorslev (1960) reviewed briefly the effects of interparticle forces and clay structure and indicated how a simple slaking test would reveal preferred particle orientation.

Brindley (1953) was the first to show that the X-ray powder camera could be used to study soil fabric. Buessem and Nagy (1954) studied the increased orientation of kaolinite under compression. Various studies of orientation due to compression, carried out by means of X-ray techniques, are quoted by Meade (1964). Recently Martin (1966) prepared wet kaolinite with polyalcohol to form specimens of 22mm square by 6-7mm thick. By means of an X-ray diffractometer fitted with a Geiger detector and pole figure device he was able to quantitatively specify the orientation of the particles in terms of the peak ratio:

$$PR = \frac{\text{amplitude of 002 reflection}}{\text{amplitude of 020 reflection}}$$

In addition to the foregoing discussion on clay structure which has dealt with those postulated and observed fabrics pertinent to soil engineering, seven modes of particle association in clay suspensions are suggested by van Olphen (1963). In his terminology, edge-to-edge and edge-to-face association is "flocculated" while face-to-face association is termed "aggregated". Dispersed refers to the presence of individual particles (as opposed to aggregation) while deflocculated designates the opposite state to flocculated. Thus, in the case of marine clays the fabric would be referred to as "flocculated but dispersed" while in the case of weathered or compacted clay having a "packet" structure the fabric would be "flocculated and aggregated".

In most recent discussions of soil fabric, an idealized arrangement based on relatively uniform particle size is usually considered. This is not applicable to most natural clays which are composed of a wide range of particle sizes, at least ranging from silt sizes down to individual clay platelets which are 100 to 1000 times smaller in mean particle dimension. When this is considered, it becomes obvious that the fabric of clay soils will normally be much more complex than idealized models infer.

The foregoing may be summarized as follows:

1) Natural clays deposited in a marine (saline) environment will have an edge-to-face fabric.

2) Natural clays deposited in a fresh water (non-saline) environment will show a variable amount of preferred orientation.

3) Natural clays subjected to weathering will probably show a domain structure as a result of leaching, wetting and drying, freeze-thaw action and perhaps ion exchange.

4) Compacted clays show somewhat different fabrics depending on the method of compaction and the compaction water content but should show a predominantly domain structure.

Because of remaining uncertainties and differences in interpretation of observations on soil structure, its influence on the strength and other soil behavior is also subject to interpretation. Many of the authors quoted above have related their concepts of soil bonding and structure to macroscopic behavior. Lambe (1953) related physico-chemical properties to soil structure, remolding, consolidation, cohesion, dry strength, secondary compression and strength regain. In two later papers (Lambe, 1958a,b) he applied the same concepts to compacted clay and showed that dry-side compaction gave a more random orientation and higher strength than did wet-side compaction which gave more parallel orientation and a weaker structure. These findings were confirmed and extended by Seed and Chan (1957, 1959) and Seed et al. (1960). Rosenqvist (1959, 1962) discussed the relations between interparticle contacts, structure and mechanical properties of clay soils. Crawford (1961, 1963) explained the strength behavior of sensitive clay on the basis of bonds and structure. Tan (1957, 1959, 1961, 1966) extensively considered soil structure and its influence on strength and consolidation properties. He suggested that deformation in a clay takes place through the "jumping of bonds"; that is, as stress on a bond increases, the bond will be broken and the particles will move relative to one another until a new equilibrium position is attained where a new bond will form.

Soil strength

The strength behavior of soils, as with other materials, may be investigated by several different procedures based on how the specimen is loaded. In standard strength testing, (a) the load may be applied at a constant rate and the resulting strain measured continuously or at given time intervals; (b) the load may be applied in increments and the resulting strain recorded at a given time after load application; or (c) the load may be applied at a constant rate of strain and the resistance of the specimen recorded at given increments of strain or time or continuously with a recorder. Rheological testing involves, (a) the application of a given load increment and the measurement of strain with time after application, which will be referred to herein as creep testing, or (b) the application of a given strain after which the decrease in load is measured with time, referred to as relaxation testing.

In most direct shear apparatuses the deformation or failure is restrained to a relatively narrow zone of undetermined thickness within the specimen. Because of this, it is incorrect to use the term strain which, for shear deformation is defined as an angular displacement. Thus, throughout this dissertation, the term deformation is used and refers to the relative displacement of the shear rings in the direction of the shear force.

In the experimental program of this study, two different types of tests were used. The first type is referred to as a direct shear test and was performed as a constant rate of deformation test. A constant rate of deformation was imposed on the specimen by a motor and gear drive and the resulting soil resistance was measured by means of a proving ring with a mounted dial extensometer. The load was recorded at given increments of deformation giving stress-deformation curves as shown in Figure 1. Curve 'a' illustrates the relationship obtained on the clay soil and curve 'b' that for the silt soil.

The second type of test was a creep test as defined above. The deformation-time relationship for creep is generally considered to consist of different stages, (Schoeck, 1957). This is illustrated in Figure 2 where stage I represents instantaneous deformation during load application; II is the transient creep in which the rate decreases and may terminate

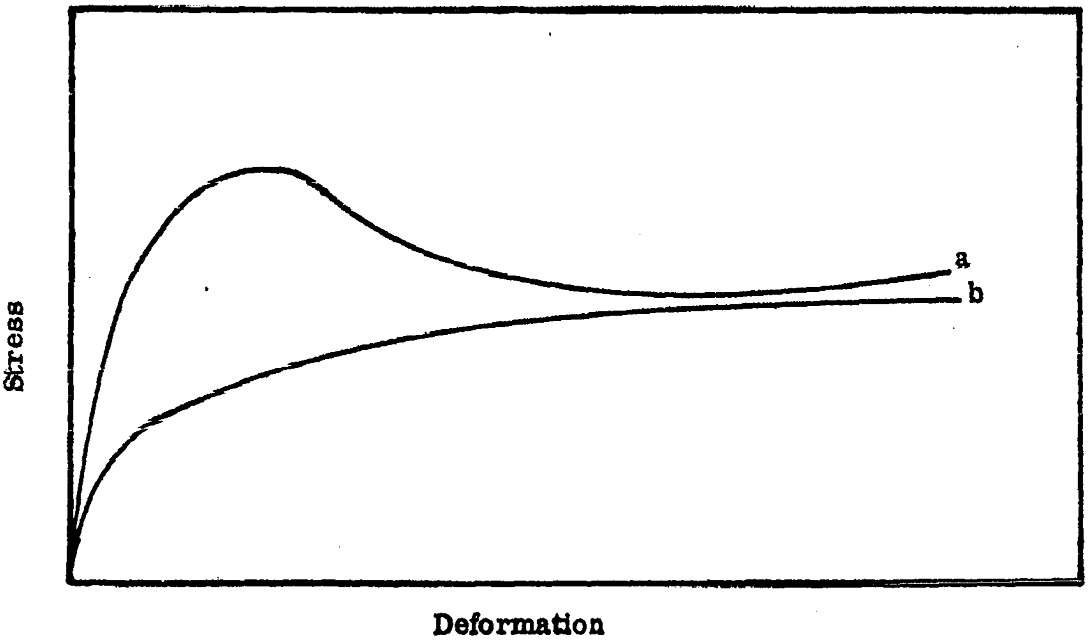

Figure 1. Stress-deformation relationships

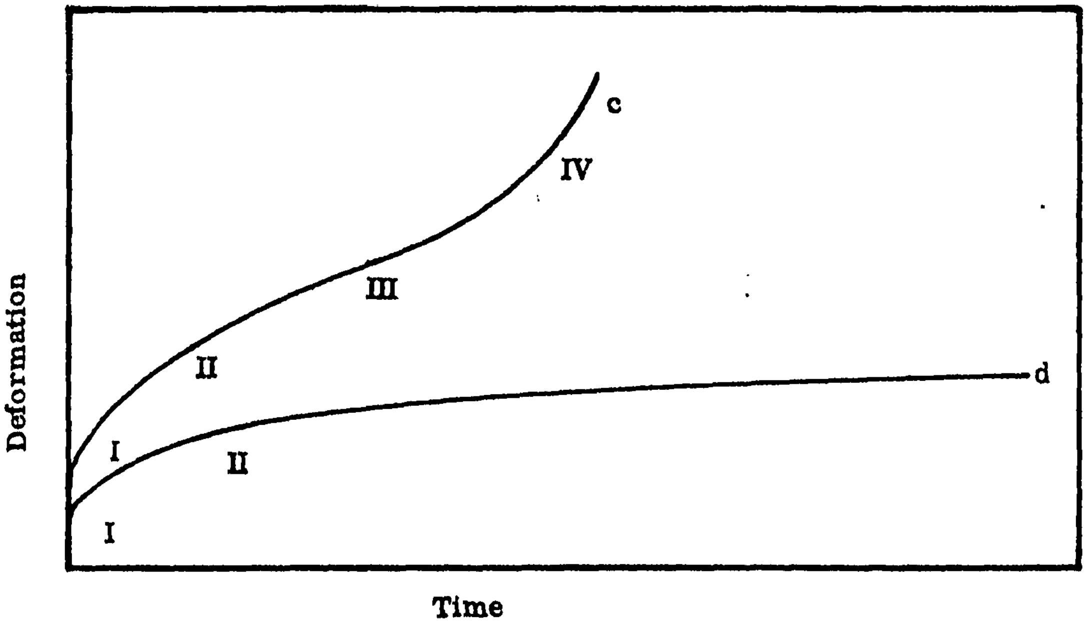

Figure 2. Deformation-time relationships

deformation under low stresses as in curve 'd'; III is the steady state creep characterized by constant rate; and IV is the "tertiary" creep in which deformation accelerates and leads to failure. The latter two stages are illustrated by curve 'c'. In the tests reported herein, the applied stresses in the majority of cases were large enough to cause failure in a relatively short period of time, ranging from 15 seconds to 15 minutes following curve 'c'. At lower stresses, the rate of deformation would rapidly decrease towards a value of zero following curve 'd'. The reason for the rather short times involved are related to the geometry of the shear apparatus in which the shear zone is restricted to a narrow zone and as shear deformation takes place, the shearing area is decreased resulting in an increased shear stress.

During deformation, "strain hardening" occurs resulting in the decreasing rate of deformation as shown by the 'd' curves. In the case of soils this strain hardening can be related to a change in particle orientation and bonding. The tendency for the soil to consolidate in the direct shear test also contributes to this hardening but is not the only factor since similar behavior has been reported for undrained triaxial tests (Singh and Mitchell, 1968). Bishop (1966) reported that the creep rate for undisturbed clay in drained creep tests did not stay constant under any constant stress or at any stage of the test.

Temperature effects

The thermal properties of soils and the influence of temperature on soil properties and behavior have been considered in a number of studies.

Kersten (1949), Carter (1951) and Scott (1961) investigated the specific heat, latent heat and thermal conductivity of soils. Winterkorn (1947), Rollins et al. (1954) and Hutcheon (1958) studied the movement of moisture under a thermal gradient. Penner (1963) related measured anisotropic thermal conduction in clays to their fabric. Eichler and Kazda (1965) studied temperature and temperature gradients in soils using a theoretical analysis and laboratory measurements. Rao and Wadhawan (1953) found permeability varied due to thermal pretreatment of soils while DuBose (1953) and Youssef et al. (1961) reported that optimum water content increased while dry density decreased with decreased temperature and both liquid limits and plastic limits decreased with increasing temperature for three clays.

The effect of temperature on soil consolidation was considered by Finn (1951) who found that the rate of consolidation was increased by increased temperature in accordance with the viscosity decrease of water while the compressibility of the soil was little affected. A comprehensive investigation of temperature effects on consolidation was reported by Paaswell (1967). His studies showed increased strain effects produced by an increase in temperature at a given stress level. He also noted that small temperature differences gave rise to effects of a secondary nature while large rapidly applied temperature differences produced primary strain effects. He explains these primary and secondary strain effects on the basis of viscosity decrease in the bulk pore water and thermal agitation of the bound water resulting in a decreased shearing resistance in the boundary layer. Although the differences in thermal expansion of soil minerals and water and a water bonding mechanism are not mentioned, Paaswell's concepts are in essential agreement with the model presented herein. Lo (1961) presented data showing the effect of temperature on secondary compression.

The variations in volume, pore pressure and effective stress with temperature variations are discussed in detail by Campanella and Mitchell (1968) for saturated soils. They considered thermal expansion of soil components, compressibility of the soil and physico-chemical effects and concluded that for isotropic consolidation the compressibility was essentially independent of temperature but the higher the temperature the lower the void ratio for any consolidation pressure. Based on measured pore pressure changes, they defined a temperature-induced pore pressure parameter which had a narrow range for many tests on different clays.

The influence of temperature on interparticle forces, pore pressures and swelling have been considered by Lambe (1953, 1958a), Rosenqvist (1962), Yong et al. (1963), Scott (1963), Mitchell (1964), Leonard and Low (1964). These studies show differences in results as well as interpretation. The Gouy-Chapman theory for colloids (Verwey and Overbeek, 1948) indicates that repulsion between particles increases with increasing temperature and thus the swelling pressure should likewise increase. This is in agreement with Rosenqvist's suggestion that the bound water increases with decreasing temperature leading to increased shearing resistance and with Yong's findings that swelling pressure decreased with decreased temperature. The decreased moisture tension found at increased temperature by Leonard and Low seems to be opposite to the trend but may be the result of increased pore water volume at increased temperatures satisfying the increased potential.

The effects of temperature on soil strength and creep behavior have been considered by Leonards and Andersland (1960). Murayama and Shibata (1961), Mitchell and Campanella (1964), Mitchell (1964), Andersland and Akili (1967) and Mitchell et al. (1968). In all cases except the first, a rate process approach has been used and the strength was found to vary with the inverse of the absolute temperature.

Model

For the purposes of this study it is assumed that the strength of clays arises through bonds formed by oriented water molecules at points of nearest approach. The soil particles are not considered to be in direct mineral-to-mineral contact due to the high adsorption energy of the minerals with respect to water molecules. The formation of these bonds is time dependent giving rise to greater strength with time after remolding or deposition through a gradual orientation of the water molecules into positions of least free energy under the influence of the surfaces 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 most 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. As new equilibrium positions are reached, new bonds will tend to form at points of contact but, as bond 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 (1959). The steady state condition for this process would depend on the structure, electrolyte concentration, water content and stress conditions. Immediately following the breaking of a bond, an unstable condition would exist which may be referred to as the "activated state" in which the "contact zone" between particles would consist essentially of unoriented water and 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 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 would be related to an activation energy and should be a function of applied stress. The energy barrier, illustrated in Figure 3, 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 which is some function of the applied stress, τ.

In chemical kinetics the Arrhenius equation is used to represent the rate constant of a chemical reaction (Moore, 1963):

$$K = A \exp(-Ea/RT) \tag{6}$$

where K is the rate constant, A is a frequency factor including an entropy term and exp (-Ea/RT) is the Boltzmann factor which represents the fraction of molecules which obtain the activation energy, Ea, above the average energy level of the system and thus react. This equation may also be written in the form:

$$K = A' \exp(-Ga/RT) \tag{7}$$

where A' is a new frequency factor not including an entropy term and Ga is the activation free energy. Since deformation in a soil involves units surmounting an energy barrier, it is assumed here that the rate of deformation is given by an equation of the form of the Arrhenius equation 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 3, this gives:

$$\Delta G = Ga + \beta'\tau \tag{8}$$

and the deformation rate may be expressed as:

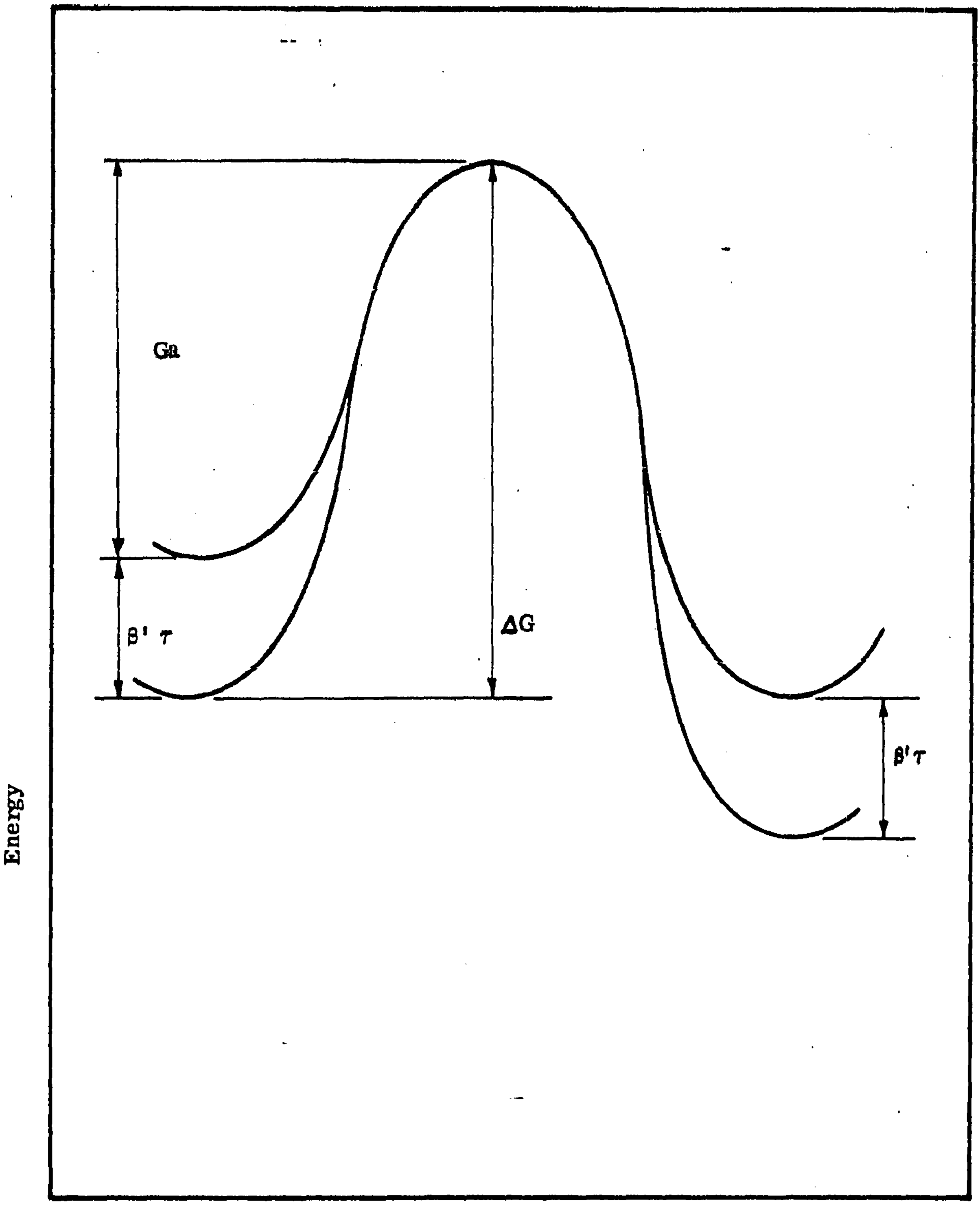

Figure 3. Energy barrier

$$\dot{\delta} = A' \exp\left(\frac{-\Delta G + \beta' \tau}{kT}\right) , \tag{9}$$

From thermodynamics, the activation free energy may be written as:

$$\Delta G = \Delta H - T\Delta S \tag{10}$$

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 $\frac{\beta'}{kT} = \beta$ (11)

and combining Equations 9, 10 and 11 gives the proposed basic equation for deformation rate:

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \exp\left(\beta \tau\right) \tag{12}$$

where the pre-exponential term A includes the entropy. In the substitution of Equation 11 it is assumed that β is independent of temperature and this assumption is substantiated by the experimental data.

Equation 12 is essentially the same as the relation suggested by Dorn (1954) on phenomenological grounds as applying to creep of metals. Dorn was concerned about the limitation that Equation 12 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 the mechanism to remain constant regardless of stress level. When $\tau = 0$, Equation 12 becomes

$$\dot{\delta} = A \exp\left(\frac{-\Delta H}{kT}\right) \tag{13}$$

Substitution of experimental values of A, ΔH and reasonable values of temperature for a clay-water system into Equation 13 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 Equation 13 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 (Eyring, 1936; Glasstone et al., 1941; Eyring and Powell, 1942) has been proposed for metals (Kauzmann, 1941), for bituminous materials (Herrin and Jones, 1963) and for soils (Murayama and Shibata, 1958, 1961 and 1964; Mitchell, 1964; Andersland and Akili, 1967; Mitchell et al., 1968). In this approach, the rate of deformation is given by:

$$\dot{\varepsilon} = A \exp \left(\frac{-\Delta H}{kT}\right) \sinh \left(\frac{\beta' \tau}{kT}\right) \tag{14}$$

When shearing stress is zero, the hyperbolic sine is zero and thus the creep rate is zero. However, for stresses of engineering interest, $\frac{\beta' \tau}{kT}$ will have a value greater than one and therefore the hyperbolic sine is reasonably approximated by the exponential as in Equation 12. Kauzmann (1941) 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 Equation 8, 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 a deformed bond. From the dimensionless group, $\frac{\beta' \tau}{kT}$, it can be seen that β' has dimensions of volume. This is interpreted to be the average volume of material containing one bond. Thus as the number of bonds per unit volume of soil increases, the size of the zone of influence of a bond would decrease. This may occur as a result of increased confining pressure or a decrease in temperature. This interpretation is in essential agreement with that of Andersland and Akili (1967) who refer to β' as the volume of a flow unit. In the derivations based on rate process theory (Glasstone, et al., 1941; Mitchell, 1964, etc.), the energy term β' arises as the product $\frac{f\lambda}{2}$ where f is the shearing force per interparticle contact and 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 of the order of the dimension 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 the above interpretation.

The pre-exponential term, A, in Equation 12 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 which affect structure in a soil include consolidation conditions and water content. The known behavior of clays (e.g. Hvorslev, 1960; Bishop and Henkel, 1962) 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 Equation 12 is obtained by taking logarithms:

$$\ln \dot{s} = \ln A - \frac{\Delta H}{kT} + \beta \tau \qquad (15)$$

For the case of direct shear tests, the shear stress is the dependent variable and Equation 15 may be written in the form:

$$\tau = \frac{1}{\beta} \left(\frac{\Delta H}{kT} + \ln \dot{s} - \ln A \right) \qquad (16)$$

Because of the method of derivation, Equation 16 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 hand side of the equation. However, justification for the assumption that Equation 16 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, τm, obtained from direct shear tests. With this assumption, the effects of normal stresses and water content can be introduced into Equation 16 to give:

$$\tau m = \frac{1}{\beta} \left(\frac{\Delta H}{kT} + \ln \dot{s} - \ln M' \right) + \rho Pc + \mu \, Pns - \gamma w \qquad (17)$$

where τm, the shear strength, replaces τ, the shear stress; 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 Equation 17 gives the relation:

$$\tau_m = \frac{1}{\beta}\left(\frac{\Delta H}{kT_s} + \ln\dot{\delta} + \alpha T_c - \ln M\right) + \rho P_c + \mu P_{ns} - \gamma w \qquad (18)$$

where T_s is the temperature of shear, T_c is the temperature of consolidation, α is a linear coefficient and M is a constant.

The introduction of these variables into Equation 16 may be considered as 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:

$$\ln\dot{\delta} = \ln M - \frac{\Delta H}{kT_s} - \alpha T_c + \beta\tau - \beta\rho P_c - \beta\mu P_{ns} + \beta\gamma w \qquad (19)$$

Equations 18 and 19 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) which is the same regardless of the method of stress application or 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 above 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 may at least be approximated by a logarithmic relation (Hvorslev, 1960; Whitman, 1960; Bishop and Henkel, 1962; Mitchell, 1964). From the coefficients of shear temperature ($\frac{\Delta H}{k}$ in Equation 19 for creep tests and $\frac{\Delta H}{\beta k}$ in Equation 18 for direct shear tests), values for activation enthalpy may be determined. Values of activation energy for various materials including reported values for soils were taken from publications and this study and are summarized in Table 1. It may be noted that the flow units for these materials vary from atoms and molecules to probably domains in the case of soils.

The proposed model equation bears considerable similarity to that proposed by Mitchell (1964). Although derived differently, their basis lies in chemical rate theory and their final form consists of a sum of linear terms which account for the variables known to affect deformation and strength behavior.

Table 1. Activation energies

Material	Reference	Activation energy Kcal/mole
Metals	Finnie and Heller, 1959	50
Concrete	Polivka and Best, 1960	54
Asphalt	Herrin and Jones, 1963	14-20
Plastics	Ree and Eyring, 1958	7-14
Water	Glasstone et al., 1941	4-5
Frozen soils	Andersland and Akili, 1967	93.6
Soils, Illite, San	Mitchell, 1964	28.7
Francisco Bay mud	Mitchell et al., 1968	31-40
Na-montmorillonite suspension	Ripple and Day, 1966	20.1-25.6
Highly plastic clay	Murayama and Shibata, 1958, 1961	28.7
Clay	This study	12-29
Silt	This study	4-7

EXPERIMENTAL PROGRAM

Properties of soils

Two different Iowa soils were used in the experimental program. The first was an alluvial clay obtained from a clay plug in the Missouri River flood plain about three miles west of the town of Missouri Valley in western Iowa. The second soil was a clayey silt obtained from an inter-till stratum near Ames, Iowa. The properties of the soils were determined using standard procedures and are summarized in Table 2.

Table 2. Soil properties

Property	Clay	Silt
Specific gravity	2.74	2.68
Liquid limit	88.8%	34.0%
Plastic limit	30.1%	24.1%
Percent passing #200	99.4%	98.2%
Percent finer than 2 microns (by hydrometer analysis)	80.4%	26.0%
Mineralogical composition (by X-ray diffraction)	Calcium montmorillonite with small amounts of Quartz Mica Kaolinite	Quartz Calcite Dolomite Montmorillonite Mica Kaolinite

Soil preparation

Except for some preliminary tests on the clay which are discussed under Results, the soils were air-dried and pulverized to pass a No. 40 sieve. This dried powder was then mixed with water to give a moisture content of 42%-45% for the highly plastic clay and 26% for the clayey silt. These moisture contents were chosen to give compacted specimens of 100% saturation. The soil was then placed in large polyethylene bags and stored in a room having 100 percent relative humidity and a temperature of 74°F for several weeks prior to testing. A few drops of formaldehyde were added to the mixing water to prevent the soil from molding during storage.

Individual test specimens were formed by removing a predetermined weight of the stored soil and molding it by hand into the shear rings. The weight was chosen to give specimens approximately one half inch in height after consolidation.

The soil was covered top and bottom with moistened filter paper which provided some lubrication during compaction and prevented the soil from entering the porous stones used during the testing period. Cylindrical steel blocks slightly smaller than the shear rings were placed above and below the soil and load was applied to statically compact the soil from both ends. A total load of 1000 pounds was applied to the highly plastic clay and 500 pounds to the clayey silt. The load was applied at a rate of about 0.08 inches per minute, maintained for one minute in the case of the clay and one half minute for the silt and then released at the same rate. The shear rings and soil specimen were then placed in the direct shear machine with porous alundum stones above and below the soil.

Shear apparatus

The experimental study was carried out by means of direct shear tests. 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 to triaxial testing. It provided a rapid means of obtaining a wide range of information in keeping with the purpose of the study. The disadvantages include: lack of drainage control during shear, non-homogeneity of stress and indeterminancy of stress components. Most of the above factors are discussed in the American Society for Testing and Materials (1953) Symposium on Direct Shear Testing of Soils and by Sowers (1964) and are further considered here only as they apply to specific aspects of the study. It was suggested by Bishop (1966) 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 $(1 - \cos \phi) \times 100\%$ which is generally less than 5% for clay of high plasticity. The disadvantages of the method were well appreciated but were considered to be outweighed by the advantages. Direct shear testing provided an assessment of the best method of testing for future work which the writer now believes would be a simple shear apparatus providing drainage control.

The direct shear apparatus used in the study was a Karol-Warner Direct Shear Machine Model No. 570 modified in several ways to meet the test requirements. Shear rings were designed and built to permit water to be circulated inside each ring as shown in Figure 5. Controlled temperature water was then circulated in series through the top and bottom rings to afford temperature control in the specimen.

For the drained shear tests an additional 50:1 gear reducer was incorporated between the drive motor and the shear apparatus. This permitted shear deformation rates of less than 0.001 inch per hour.

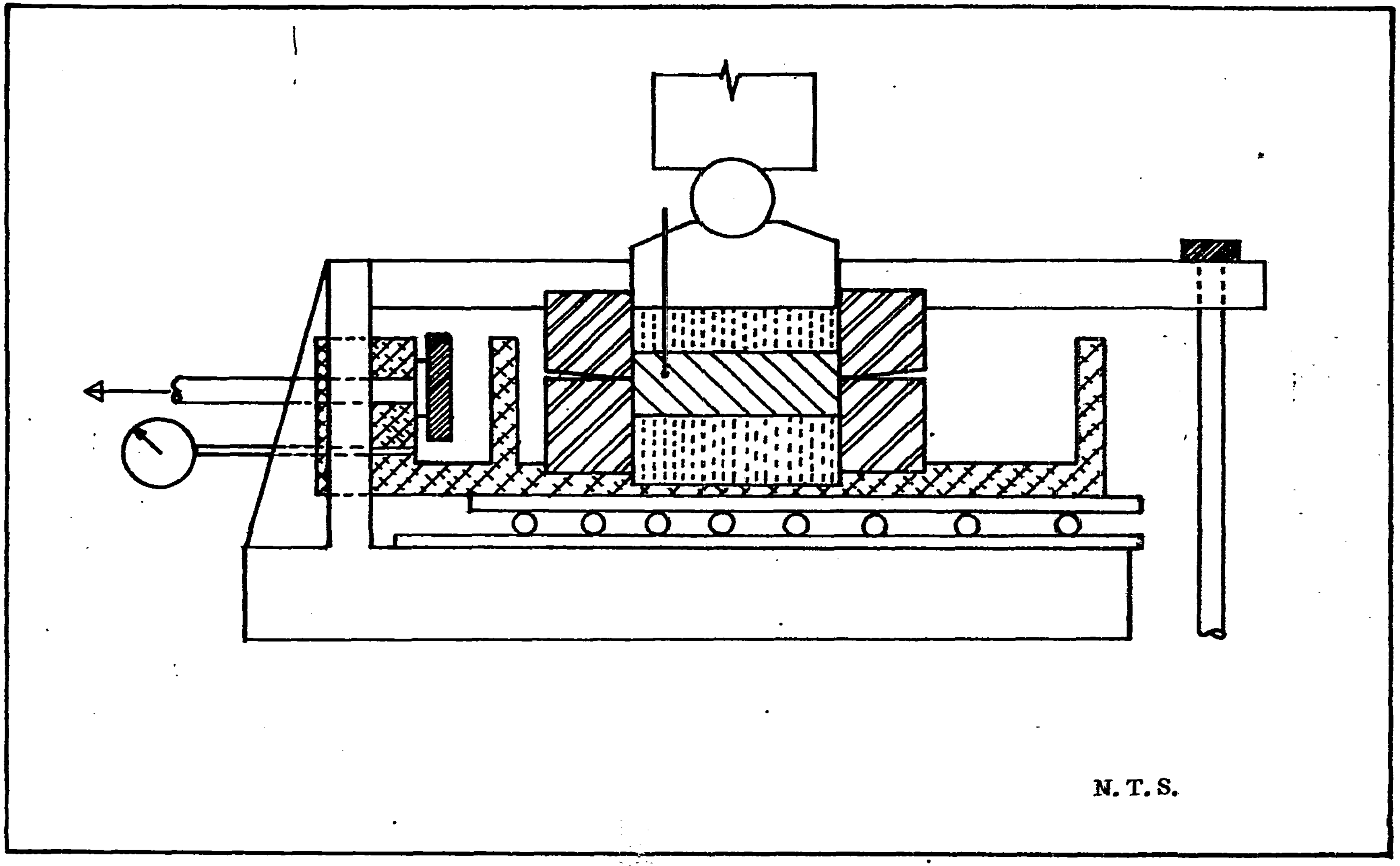

Figure 4. Shear apparatus

For creep tests, the proving ring and gear drive were removed and load was applied by weights through a pulley-supported cable. A schematic drawing of the assembled shear apparatus is shown in Figure 4.

The loading cap and porous alundum stones had two 3/32 inch holes drilled 7/8 inch from the center to permit a thermocouple and a pore pressure measuring needle to be inserted into the soil specimen from the top.

The immersion chamber of the direct shear apparatus was filled with distilled water and was entirely surrounded with about 3/4 inch of glass wool insulation. To prevent excessive evaporation from the sample and immersion chamber, a ring of moistened felt was placed around the loading cap and the loading cap and immersion chamber were covered by a sheet of plastic wrap.

Temperature control was achieved by means of two controlled temperature water baths. For higher temperatures a 0.7 cubic foot insulated glass jar was heated by means of two immersed light bulbs equipped with rheostats. One bulb operated intermittently by means of an immersion controller while the other operated continuously. The second bath had a capacity of about 1½ cubic feet and was equipped with two immersion heaters and a refrigeration coil. The refrigeration unit was run continuously for temperatures of 20°C and below and the temperature of the bath was controlled by an immersion controller actuating one of the heaters. Ethylene glycol was added to the water in this bath to prevent ice forming around the refrigeration coil. The temperature of the sample and the water in the immersion chamber were controlled by circulating the water or solution through the shear rings by means of small circulating pumps.

In both cases the bath temperatures could be controlled accurately with less than 1/10°C variations except during initial heating or cooling of the soil at which time the variation in temperature of the bath was generally less than 1 or 2°C depending on the magnitude of the temperature change occurring. Specimen temperature was measured by means of a copper-constantan thermocouple and a Brown potentiometric recorder. Temperature could be read with a precision of 0.2 degrees with this recorder. After equilibrium was achieved, the temperature of the specimen could be maintained to within ¼°C with small fluctuations primarily due to changes in room temperature. If the temperature did drift from its preset value no attempt was made to correct it immediately before shearing as the error due to temperature difference was considered to be small compared to the error which would result from a temperature change and its accompanying pore-pressure changes. Unfortunately some of the insulation had to be removed from the apparatus during shear testing and at the highest and lowest shear temperatures very small fluctuations in temperature could occur during the testing period thus introducing errors into the results.

Pore pressure measurements

The pore water pressure measuring device was built using a pressure transducer (No. 4-312-0001) manufactured by Consolidated Electrodynamics. This transducer is an unbonded wire type having a range of 0-100 psi absolute pressure with an output of 4 millivolts per volt. It has a flush diaphram, ½ inch in diameter and a length of 3/4 inch. The transducer was placed in an adapter (No. 4-017-0042) and a stainless steel plug, drilled

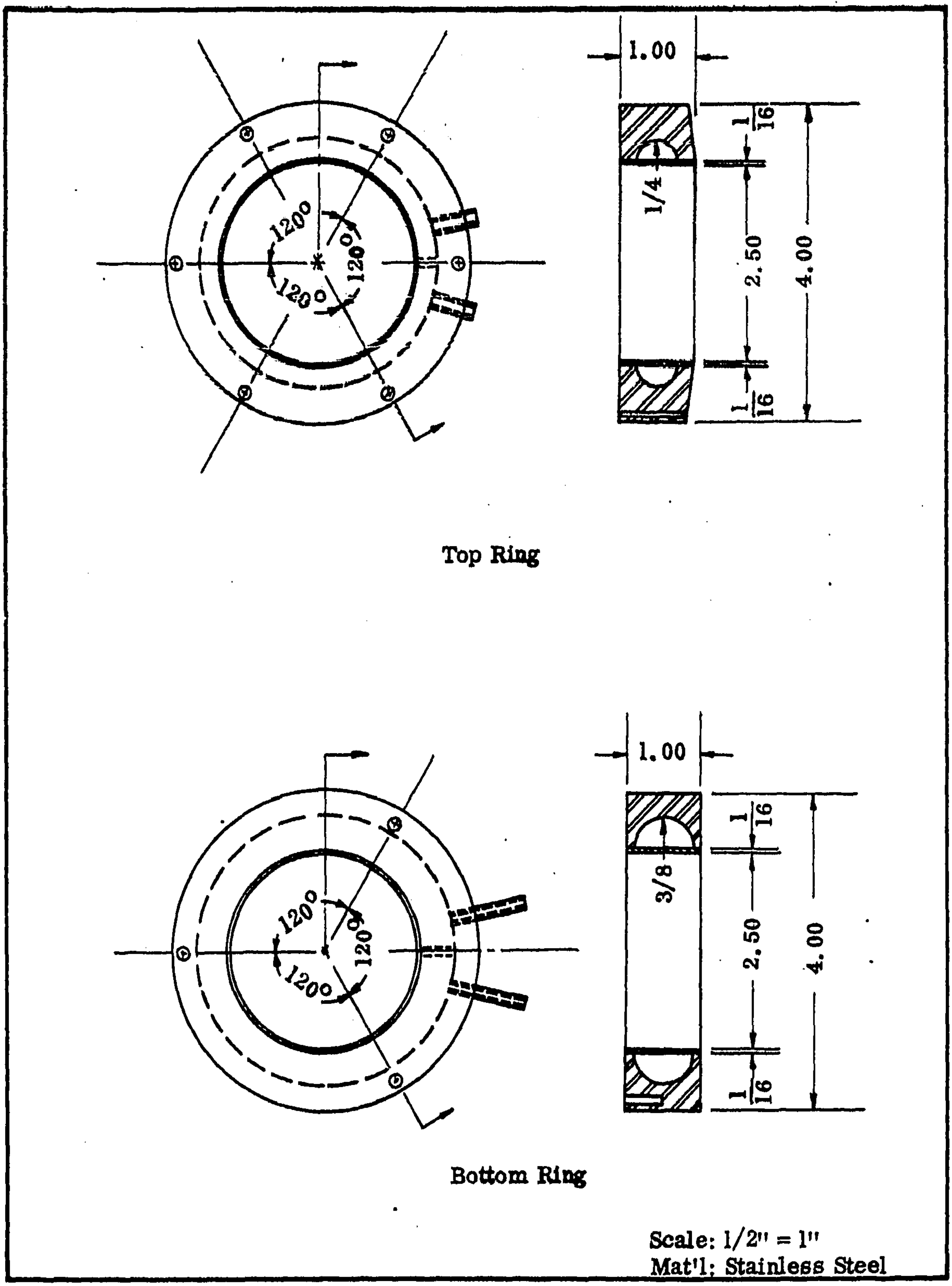

Figure 5. Shear rings for temperature control.

to take a No. 14 hypodermic needle which was fitted with a porous sintered stainless steel cylinder 0.065 inches long supplied by the Parker Pen Company (Code No. S-564). This tip prevented soil from entering the needle when it was inserted into the soil. This arrangement, shown in Figure 6, gave a very rigid system between the soil and the transducer. Burn (1964) and Barden (1964) discuss the use of pressure transducers for measuring pore water pressures and the effects of system rigidity.

The transducer was read by means of a Model 300D Transducer Amplifier-Indicator with a Type 93 Strain gage input module manufactured by Daytronic Corporation. The indicator was calibrated to read directly in pounds per square inch with a reading of 10 psi taken at atmospheric pressure for convenience. On the 20% scale the indicator could be read directly to 0.2 psi and estimated to 0.05 psi. Pressures more than 10 psi below atmospheric could not be read with this setting but this was sufficient since pressures less than 10 psi below atmospheric would cause cavitation due to dissolved air in the water system (Gibbs, 1965).

The use of the pore pressure measuring device was a side issue to the main investigation and was an attempt to determine whether pore water pressures could be measured in direct shear tests by this method. As would be expected, the main difficulty in the method was maintaining a deaired condition in the needle. Any small air bubble in a system of this sort will cause time lags in pressure readings (Whitman and Richardson, 1961). The system would acquire air during the course of the tests, probably from the soil specimen which was not initially saturated, and thus it was necessary to disassemble the transducer and porous tip after each one

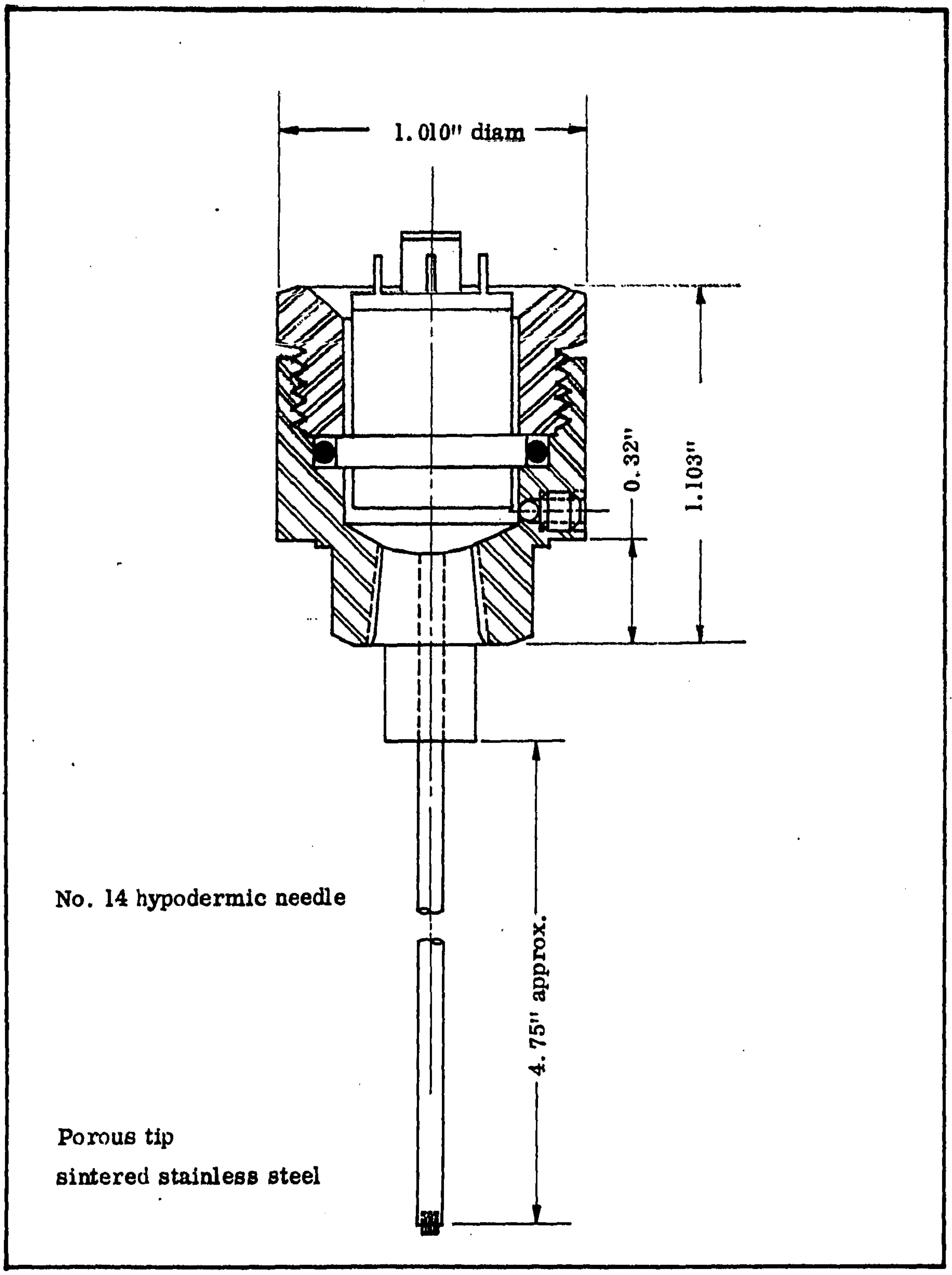

Figure 6. Pore pressure measuring device

or two tests and flush with fresh de-aired distilled water before reassembling. The use of de-aired water retarded but did not prevent the accumulation of air in the system.

Pore pressure changes were noted during the consolidation phase of the tests. Generally, for the clay, a maximum of one quarter to one third of the applied stress was registered as pore pressure. This pressure built up to the maximum about thirty minutes after the load was applied. The lag was probably due to air in the soil which limited the buildup of pore water pressures by transferring load immediately to the soil skeleton and also preventing movement of water within the soil. Towards the end of primary consolidation as the specimen became more saturated, pore pressures responded more rapidly and were more nearly equal to the magnitude of stress changes. The pore pressure response of silt to load changes, because of its much greater permeability, occurred in less than one minute but the magnitude of the response was quite erratic.

Pore pressure changes were also noted during shear or creep phases of some tests. In general for the clay, the pore water pressure increased during shear as would be expected for normally consolidated soil (Bishop and Henkel, 1962). This is interpreted to indicate a break-down of the soil structure and in all cases was accompanied by consolidation of the soil as indicated by the vertical deformation indicator. For the silt which was over consolidated, pore pressures were negative during shear deformation and the vertical deformation indicator showed dilation of the specimen.

Temperature changes tended to cause pore pressure changes. This can be partially attributed to the measuring device since the coefficient of volumetric thermal expansion of water is about 100 times that for steel (42.1×10^{-4} for water and 35.5×10^{-6} for steel, Lange, 1949, pp. 1641). Thus the water in the needle would change volume more than the needle and the relatively impervious soil would resist the flow necessary to achieve equilibrium. However, the volumetric expansion of the clay particles is of the same order as steel (30×10^{-6} for slate and sandstone and 38×10^{-6} for quartz, from Lange (1949), pp. 1641, 1642) and thus, neglecting effects due to adsorbed water, it can be assumed that cooling a saturated clay soil will result in negative pore pressures and similarly, heating will cause positive pressures in the pore water. Since the coefficient for the soil particles is of the same order as steel, it may be assumed that, in a saturated soil, the change in pressure due to temperature would be of the same order of magnitude as that in the measuring device and thus the indicated pressures could be taken as reasonably valid for the soil. In the case of partially saturated soil, the pressure in the soil would probably be lower but some transfer would take place with the needle and the indicated pressures would still be of the right magnitude. This point is not critical for the investigation as the pressures were used only as an indication of soil behavior at any stage in the test.

The effect of temperature on the soil pore water was confirmed by the behavior of the soil specimens subjected to temperature changes. On cooling the volume of the specimen would initially decrease and then, as the soil reached the new temperature and the rate of cooling decreased, the specimen would reswell about one half the original volume decrease. In all cases of cooling, negative pore pressures were indicated and cavitation of the water would occur if the change in temperature was large and rapid. Conversely heating the sample would cause a volume increase and positive pressures which would decrease with time. This behavior is similar to that reported by Mitchell and Campanella (1964) and Campanella and Mitchell (1968).

Testing procedure

In order to eliminate extraneous variability from one soil specimen to the next, all steps in the testing procedure were standardized as much as possible. After the shear rings with the compacted specimen were placed in the apparatus, the water lines were connected, the thermocouple and pore pressure needle were inserted, the chamber was filled with distilled water and the entire chamber was enclosed in about 3/4 inch of glass wool insulation. An initial token load of 3 psi was applied to the specimen and water circulation was started to bring the specimen to the consolidation temperature. The soil had essentially reached equilibrium temperature in one half hour for the extreme changes in temperature so, for consistency, all specimens were left for one half hour at this stage of the procedure. At the end of this period the entire consolidation load was applied as one increment and deflection readings were taken at approximately doubled time intervals throughout the consolidation period.

All specimens of a series were consolidated for the same total time even though 100% primary consolidation was attained in a shorter time at higher temperatures than at lower temperatures. It was felt that the total time under the consolidation pressure would be of as much significance in the formation of bonds as the stage on the consolidation-time curve. Some early tests indicated that long secondary consolidation gave only a slight increase in strength and thus a constant time period was used.

The temperature of the specimens was changed from the consolidation temperature to the shear temperature during the final 3 hours of the consolidation period. Again the time was fixed for consistency even though equilibrium under 10°C changes took considerably less time to achieve than under 60°C changes.

In all cases the shear temperature was equal to or less than the consolidation temperature. This was necessary because any increase in temperature above the 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.

It has been noted by several authors (e.g. Olson, 1964; Henkel and Sowa, 1964; Campanella and Mitchell, 1968) that pore pressures tend to increase irreversibly with temperature fluctuations in undrained triaxial specimens. The effects of temperature on consolidation have been noted (Lo, 1961; Paaswell, 1967; Campanella and Mitchell, 1968). In all cases the data presented are consistent with the soil model presented herein and can be explained on this basis. An increase in temperature of a clay specimen results in an increase in pore water pressure, a decrease in the viscosity of the water and an increase in the thermal energy causing selfdiffusion of the bound water. If drainage is not permitted (as in an undrained triaxial specimen) the pore pressure will increase and bonds carrying stress will tend to relax through diffusion of the bound water thus throwing some load onto the pore water. On subsequent cooling the pressure will decrease due to thermal contraction of the water but the additional load, transferred to the water from the skeleton on heating, will still be carried by the water resulting in a residual pore water pressure. Further temperature cycles will further increase this residual pressure by the same mechanism.

From this it is obvious why an increase in temperature causes an increase in magnitude and rate of consolidation. In making the larger temperature changes, the rate of temperature decrease was kept as small as possible, consistent with achieving equilibrium in three hours, in order to prevent large negative pore water pressures from occurring. The procedure used for the silt was similar except that times were shortened considerably due to the much higher permeability of the silt as compared to the clay.

Discussion of procedure

Although care was taken in all phases of the testing program and procedures were maintained as consistent as possible, considerable difficulty was encountered as a result of scatter of results. The initial water content of the soil and compaction pressure were chosen to give specimens with 100% saturation. Using the specimen dimensions after consolidation and the water content after the end of the test, computed saturations were consistently between 100 and 110 percent with the majority between 100 and

105 percent. This anomaly was probably due to swelling during the time it took (1 to 5 minutes) to remove the sheared specimen from the shear device and rings. It does indicate, however, that specimens were near saturation during the test. The specimen densities were also very consistent with a maximum deviation of about 2%. These differences occurred in specimens consolidated at the same temperature and under the same load and showed up as differences in amount of consolidation. The most reasonable explanation of this variation is slight differences in the internal structure of the soil specimens due to differences in initial water content and remolding. Duplications of tests showed variations in shear strength but these variations did not correlate well with the variations in density or water content also tending to confirm the hypothesis of variable internal structure.

Some variations in shear strength and creep behavior of apparently identical specimens may be attributed to the particular shear apparatus, which has some weaknesses. The upper ring stop was free to move and tended to lift under shear load. This allowed the upper shear ring to lift or rotate away from the lower ring. To prevent this, lugs were added to the front support blocks and the anchor bolt was inserted at the rear. However, any pressure on the anchor bolt or tendency for the upper ring to twist would result in significant pressure between the rings, giving a frictional resistance to shear deformation. It was determined that variations in strength of ¼ psi could easily result from this source. However, for tests run in the latter part of the program, the lugs and anchor bolt were removed and the upper ring was permitted to move in an attempt to obtain less variability of results.

Experimental program

The laboratory tests performed in this investigation were essentially 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 strain 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.

The following comments are designed to clarify terminology used. Each test, whether creep or direct shear, was considered to have two phases: a consolidation phase and a shear phase. Thus the normal stress during shear refers to the normal stress on the shear plane during the shear phase and is given the symbol Pns. All tests on clay were performed with the consolidation pressure, Pc, equal to Pns and this will be referred to simply as normal stress or Pc. Other symbols used in the tables are defined as follows:

Tc = temperature during consolidation phase.

Ts = temperature during shear phase.

τ = shear stress.

τ_m = shear strength.

w = water content.

δ = deformation.

$\dot{\delta}$ = deformation rate.

Each type of test was run on each soil type giving four series of tests designated A, B, C and D. Within each series, the consolidation and shearing temperatures were varied in fixed increments. The majority of the tests were performed at consolidation temperatures, Tc, of 60, 40 and 20 degrees centigrade and with shearing temperatures, Ts, equal or less than the consolidation temperature as previously explained. The effect of consolidation stress, Pc, and normal stress during shear, Pns, was studied by varying these stresses over a certain range. For direct shear tests on the clay (Series B), the consolidation stress was varied from 15 to 120 psi with the majority performed at 45, 60 and 90 psi. The normal stress during shear was equal to the consolidation stress for all tests on the clay. Direct shear tests on silt (Series D) were performed at consolidation stresses of 20, 40 and 60 psi and normal stresses during shear of 20, 40, or 60 psi so that the soil was either normally consolidated (Pns = Pc) or overconsolidated (Pns $<$ Pc). Creep tests on the clay were performed at consolidation stresses of 45 psi (Series A1) and 60 psi (Series A2) and at the same values of normal stress during shear. The majority of creep tests on the silt were performed at a consolidation stress of 40 psi and a normal stress during shear of 20 psi (Series C1). Tests with a temperature of consolidation of 40^oC and shear temperatures of 40 and 2^oC were performed at consolidation pressures of 20 and 60 psi with normal stress during shear of 20, 40 and 60 psi and 40 psi consolidation pressure with normal stress during shear of 40 psi.

Standard consolidation tests were run on each soil at temperatures of 60°C and 2°C to show the effect of temperature on the consolidation properties of these soils.

RESULTS AND ANALYSIS

Results

The principal data of all tests performed in the experimental program are given in Tables 3 to 12 inclusive. Table 3 gives the data for Series A1 which consists of 42 creep tests performed on the highly plastic clay (referred to hereafter simply as clay) with a normal stress, Pc, of 45 psi. The table shows values of Tc, Ts, τ , w and the resulting $\dot{\delta}$ where the symbols are as defined on pages 46 and 47. Table 4 gives the same data for Series A2 which consists of 50 tests performed on clay with Pc = 60 psi. The soil for both these series was prepared as discussed under Soil preparation and in each case the specimens were consolidated for 19 hours at the consolidation temperature and 3 hours were used to change to the shear temperature. The method of obtaining the deformation rate is discussed under Analysis.

The 189 direct shear tests on the clay were divided into 5 series (B1 to B5) and the data summarized in Tables 5 to 9 respectively. These 5 series reflect variations in soil preparation, test variables and technique. Series B1 and B3 were carried out with a broad range of variables to determine what factors would affect the strength. The other series were performed with several fixed variables to determine the effects of temperature.

The soil of series B1 was ground to pass a No. 8 sieve, mixed with water, formed into balls of specimen weight and stored in a humid room for one or more weeks prior to testing. For Series B2 the soil was ground to pass a No. 40 sieve and, for each specimen, was individually mixed with water and was stored in a loose condition for one week prior to testing at

Table 3. Creep tests on clay Series A1 (Pc = 45 psi)

Ts °C	τ psi	w %	$\dot{\delta}$ in./min	Ts °C	τ psi	w %	$\dot{\delta}$ in./min
Tc = 60°C							
60	18.52	38.27	0.20	40	16.52	40.64	0.05
	17.54	39.83	0.33		15.51	40.80	0.0135
	16.52	39.86	0.65		15.05	41.08	0.0057
	14.49	39.60	0.0019		17.08	40.39	0.049
	16.52	39.76	0.067				
	15.51	40.23	0.044	2	18.57	39.17	0.0075
	17.08	40.40	0.188		19.58	39.95	0.0096
	14.49	39.47	0.0034		17.54	40.94	0.0052
	15.05	39.13	0.0034		19.58	42.03	0.106
	15.05	39.56	0.0115		18.57	41.87	0.044
					18.10	41.57	0.0315
20	17.54	41.22	0.150				
	16.52	40.99	0.142				
	17.54	41.03	0.068				
	17.08	41.24	0.0375				
Tc = 40°C							
40	16.52	41.82	0.550	20	14.03	41.73	0.005
	14.49	42.77	0.147		15.05	42.41	0.058
	13.47	42.29	0.0037		16.07	41.83	0.230
	14.03	42.15	0.041				
2	16.07	42.55	0.09				
	15.51	42.04	0.041				
	14.49	42.32	0.0061				
Tc = 20°C							
20	14.49	40.05	0.038	2	15.05	40.25	0.012
	15.51	40.35	0.115		17.08	40.51	0.24
	16.52	40.59	1.25		16.52	40.59	0.14
	15.05	40.74	0.20		16.07	40.20	0.066
	14.03	40.96	0.044				
	15.05	40.46	0.136				
	14.49	41.22	0.038				
	16.07	40.96	0.58				

Table 4. Creep tests on clay Series A2 (P_c = 60 psi)

Ts °C	τ psi	w %	$\dot{\delta}$ in./min	Ts °C	τ psi	w %	$\dot{\delta}$ in./min
Tc = 60°C							
60	20.60	39.09	0.12	40	21.62	39.78	0.173
	20.13	38.43	0.052		21.15	40.02	0.115
	19.58	38.73	0.0275		20.60	39.77	0.079
	19.12	38.64	0.021		19.58	39.71	0.028
	18.57	39.10	0.010		18.57	39.91	0.0048
	17.54	38.88	0.003				
				2	22.64	39.67	0.041
20	22.64	40.03	0.240		22.17	40.10	0.0224
	21.62	40.19	0.051		21.62	40.36	0.0218
	20.60	40.09	0.020		21.15	39.79	0.0087
	19.58	39.81	0.008		20.60	39.90	0.004
Tc = 40°C							
40	19.58	40.14	0.415	20	19.58	40.35	0.197
	19.12	40.16	0.300		19.12	40.34	0.147
	19.58	39.44	0.165		19.12	40.55	0.125
	18.57	39.48	0.108		18.57	40.39	0.051
	18.10	39.37	0.041		18.10	40.27	0.0308
	17.54	39.34	0.0148		17.54	39.09	0.0178
	17.08	40.58	0.010		17.08	40.52	0.0069
	16.52	40.19	0.0058				
1.5	20.60	40.46	0.114				
	20.13	40.31	0.043				
	19.58	40.60	0.0355				
	19.12	40.28	0.0255				
	18.57	40.42	0.0065				
Tc = 20°C							
20	18.57	39.58	0.180	2	19.58	40.39	0.150
	18.00	39.78	0.081		19.01	39.72	0.0220
	17.54	39.91	0.045		18.57	39.63	0.0265
	16.97	39.20	0.0138		17.54	40.36	0.0147
	16.52	39.65	0.0118		17.54	39.55	0.009

Table 5. Direct shear tests on Clay Series B1

Pc psi	Tc °C	Ts °C	δ in./min	w %	τ_m psi
30	14.0	14.0	.05	32.50	18.42
30	50.0	50.0	.05	32.60	16.30
30	23.0	23.0	.05	38.70	9.54
60	50.0	50.0	.05	37.00	16.80
120	50.0	50.0	.05	32.60	30.50
30	50.0	50.0	.05	38.40	12.82
15	50.0	50.0	.05	39.30	8.95
45	50.0	50.0	.05	37.00	15.80
90	50.0	50.0	.05	34.00	23.90
90	15.0	15.0	.05	34.90	21.80
30	15.0	15.0	.05	40.50	11.80
15	15.0	15.0	.05	40.50	9.13
15	22.5	22.5	.05	40.00	9.47
120	23.0	23.0	.05	33.80	26.20
90	20.0	20.0	.05	35.70	22.40
90	50.0	16.0	.05	34.70	28.30
90	50.0	50.0	.05	35.00	21.56
90	50.0	50.0	.05	34.50	23.50
90	15.0	15.0	.04	35.00	24.20
90	50.0	50.0	.05	34.75	26.80
90	50.0	15.5	.05	35.20	28.20
90	50.0	30.0	.05	35.70	29.25
90	50.0	20.0	.05	35.70	28.00
90	50.0	39.0	.05	34.60	27.00
90	50.0	30.0	.05	35.00	26.60
90	58.8	30.0	.06	34.40	27.45
90	58.8	58.8	.04	34.40	24.85
90	58.8	16.5	.05	35.30	29.10
90	40.0	16.5	.05	35.20	27.20
90	30.0	15.0	.05	35.00	25.00
90	20.0	15.0	.05	36.10	23.30
90	15.0	15.0	.05	36.00	22.35
90	59.5	15.0	.05	35.40	28.90
45	23.0	23.0	.10	34.50	23.45
45	22.0	22.0	.10	34.20	23.30
90	23.0	23.0	.04	35.40	23.40
45	22.2	22.2	.10	38.80	20.05
45	22.0	22.0	.10	39.80	20.60
45	23.3	23.3	.10	39.00	19.90
45	23.3	23.3	.10	38.90	20.50
45	22.8	22.8	.10	38.90	21.60
45	22.0	21.7	.10	38.80	21.60
45	23.4	23.4	.10	39.20	21.40

Table 6. Direct shear tests on clay Series B2 (Pc = 45 psi; Tc = 65°C; $\dot{\delta}$ = 0.10 in./min)

Ts °C	w %	τ_m psi	Ts °C	w %	τ_m psi
65.0	37.10	23.60	30.0	38.41	23.54
65.0	37.60	21.10	10.0	39.02	26.47
65.0	37.00	23.70	3.0	39.39	26.69
5.5	39.60	25.60	5.0	38.43	26.26
5.0	39.00	27.00	3.0	38.82	27.00
4.5	38.60	25.70	2.0	39.14	26.38
21.0	39.40	22.25	1.5	38.43	26.16
6.0	40.00	26.90	2.0	39.25	26.38
65.0	37.70	24.40	1.5	38.91	26.38
30.0	40.00	24.62	2.0	39.16	27.34
30.0	39.30	26.14	2.0	38.98	27.88
30.0	38.40	25.49	2.0	39.15	27.99
30.0	39.60	25.27	65.0	38.07	22.68
65.0	38.70	22.57	2.0	39.49	27.03
45.0	39.00	23.44	12.0	39.83	26.91
45.0	38.60	23.11	12.5		26.81
45.0	39.40	21.40	12.0	38.93	26.38
65.0	39.00	20.71	12.0	39.15	26.69
65.0	37.00	22.00	12.0	39.30	26.16
20.0	39.30	25.00	40.0	38.16	23.93
65.0	38.00	21.70	40.0	39.33	24.20
20.0	38.80	24.80	40.0	38.44	24.15
65.0	37.10	21.70	65.0	36.68	22.35
20.0	38.00	25.70	40.0	39.78	23.33
20.0	38.51	25.38	40.0	38.99	23.65
30.0	37.72	26.57	40.0	39.08	23.11
30.0	37.60	27.25	40.0	37.90	23.87
30.0	38.50	25.00	65.0	38.72	21.70
30.0	38.41	23.50	65.0	38.74	20.49
30.0	38.81	23.30	50.0	39.02	21.80
30.0	38.91	24.52			

Table 7. Direct shear tests on clay Series B3 (Tc = 60°C)

Pc psi	Ts °C	$\dot{\delta}$ in./min	w %	τ_m psi	Pc psi	Ts °C	$\dot{\delta}$ in./min	w %	τ_m psi
45	60	.1	42.59	15.27	120	60	.01	35.06	31.72
45	5	.1	44.08	17.28	120	2	.01	37.22	33.84
45	15	.1	43.69	16.77	15	60	.01	49.57	6.79
45	25	.1	43.43	15.77	15	2	.01	51.11	8.85
45	25	.01	44.47	15.26	15	60	.01	50.19	6.91
45	60	.01	41.93	14.42	15	2	.01	50.35	8.38
45	60	.1	43.19	14.13	45	2	.01	44.23	16.76

Table 8. Direct shear tests on clay Series B4 (P_c = 60 psi; $\dot{\delta}$ = 0.01 in./min)

Tc °C	Ts °C	w %	τ_m psi	Tc °C	Ts °C	w %	τ_m psi
60	60	37.92	19.71	40	40	39.30	17.62
60	60	38.20	18.63	40	10	39.67	19.05
60	60	37.92	19.11	40	20	39.64	18.80
60	2	39.57	22.66	40	30	39.31	17.96
60	2	39.11	23.05	30	30	39.31	17.49
60	10	39.59	20.72	30	20	39.72	17.50
60	20	38.47	20.96	30	10	39.65	17.45
60	30	39.59	19.98	30	2	39.78	18.12
60	40	38.60	19.13	20	20	39.61	16.25
60	50	38.45	19.39	25	24	39.78	16.19
50	50	38.71	18.20	20	2	39.99	17.50
50	40	39.09	18.59	20	10	39.04	17.41
50	30	38.68	19.00	10	10	39.58	16.29
50	20	39.37	19.56	10	2	40.34	16.47
50	10	39.28	19.75	2	2	39.98	16.25
50	2	39.81	19.52	50	2	38.96	20.73
40	2	40.23	19.44	60	2	38.60	21.50
40	40	39.21	17.02				

Table 9. Direct shear tests on clay Series B5 (P_c = 60 psi; $\dot{\delta}$ = 0.025 in./min)

Tc °C	Ts °C	w %	τ_m psi	Tc °C	Ts °C	w %	τ_m psi
50	50	38.89	19.14	40	40	38.65	18.74
50	20	39.75	19.55	40	10	39.26	20.29
50	10	39.28	21.43	40	20	39.16	19.43
50	2	39.60	21.11	40	30	38.54	19.17
40	2	40.36	20.12	50	30	38.42	19.86
40	10	39.99	19.87	50	40	38.77	20.31
40	20	39.39	19.14	50	20	39.61	20.44
40	30	39.25	19.38	50	50	37.62	19.82
40	40	38.69	18.29	20	20	39.53	17.13
50	40	38.98	19.19	60	20	39.10	21.01
50	30	38.95	20.10	60	30	38.43	21.91
30	30	39.56	17.41	60	30	38.79	21.93
30	20	39.64	17.83	60	60	36.66	19.82
30	10	40.12	18.32	60	10	41.33	18.52
30	2	40.28	18.81	60	10	39.45	22.70
21	21	40.00	17.58	60	2	39.20	23.52
20	2	39.99	18.06	60	20	38.60	21.54
20	10	40.39	17.39	60	40	38.90	21.23
21	21	39.61	17.34				

Table 10. Creep tests on silt Series C1 (Pc = 40 psi; Pns = 20 psi)

Ts °C	τ psi	$\dot{\delta}$ in./min	w %	Ts °C	τ psi	$\dot{\delta}$ in./min	w %
			Tc = 60°C				
60	18.57	0.800	22.70	20	17.54	0.087	21.83
	18.57	0.335	21.86		16.52	0.105	22.41
	18.57	0.210	22.03		16.52	0.053	22.10
	17.54	0.405	22.44		15.51	0.037	21.96
	17.54	0.120	21.95		14.49	0.0275	22.19
	16.52	0.700	22.35		13.47	0.0135	22.09
	16.52	0.098	22.38		12.45	0.0087	21.93
	15.51	0.120	21.78				
	15.51	0.120	22.26	2	17.54	0.0610	22.32
	14.49	0.076	23.38		16.53	0.0405	22.09
	14.49	0.030	21.98		15.51	0.0225	22.62
	13.47	0.032	21.76		15.51	0.0180	22.36
	13.47	0.0255	22.01		13.47	0.0112	22.45
	13.47	0.0255	22.22		12.45	0.0034	22.10
	12.45	0.0172	22.17				
	12.45	0.0160	21.76				
40	18.57	0.280	22.08				
	18.57	0.280	22.16				
	17.54	0.200	22.06				
	16.53	0.079	21.75				
	15.51	0.060	21.91				
	14.49	0.102	22.18				
	14.49	0.089	22.59				
	14.49	0.058	22.21				
	13.47	0.0092	21.67				
	13.47	0.0176	22.05				
	12.45	0.008	21.31				
			Tc = 40°C				
40	18.57	0.285	22.38	20	17.54	0.096	22.70
	17.54	0.115	22.28		16.52	0.082	23.02
	17.54	0.090	22.12		15.51	0.043	22.54
	16.52	0.130	22.51		14.49	0.0263	22.33
	15.51	0.062	22.18		13.47	0.0337	22.15
	14.49	0.050	22.39				
	14.49	0.019	21.85	2	18.57	0.046	22.65
	13.47	0.0375	22.36		17.54	0.0355	22.54
	12.45	0.0145	22.22		16.52	0.0125	22.20
					15.51	0.022	23.03
20	18.57	0.172	22.52		14.49	0.006	22.21

Table 10. (Continued)

Ts °C	τ psi	$\dot{\delta}$ in./min	w %	Ts °C	τ psi	$\dot{\delta}$ in./min	w %
Tc = 20°C							
20	19.58	0.200	21.94	2	18.57	0.163	22.47
	19.58	0.200	22.19		17.54	0.050	22.42
	18.57	0.320	22.39		16.52	0.060	22.44
	17.54	0.133	22.37		16.51	0.050	22.40
	16.52	0.070	22.25		14.49	0.018	22.43
	16.52	0.070	22.25		14.49	0.013	21.97
	15.51	0.0315	22.06		12.45	0.0081	22.75
	14.49	0.025	21.78				
	13.92	0.024	22.99				
	13.47	0.0096	21.41				
	12.45	0.0084	22.12				
	12.45	0.010	21.84				

Table 11. Creep tests on silt Series C2 (Tc = 40°C)

Ts °C	τ psi	$\dot{\delta}$ in./min	w %	Ts °C	τ psi	$\dot{\delta}$ in./min	w %
Pc = 60psi; Pns = 60psi							
40	31.20	0.35	20.77	2	33.24	0.29	21.90
	30.18	0.60	20.59		31.20	0.050	20.86
	30.18	0.80	20.17		30.18	0.090	20.79
	30.18	1.00	21.45		25.09	0.026	21.05
	29.71	0.12	20.37		24.20	0.0245	20.75
	29.16	0.25	20.52				
	28.15	0.059	20.29				
	24.20	0.043	20.32				
Pc = 60psi; Pns = 40psi							
40	25.09	0.35	21.51	2	26.11	0.052	21.38
	24.20	0.11	20.88		26.11	0.070	21.62
	23.65	0.046	20.80		25.09	0.054	21.49
	21.62	0.043	21.08		23.65	0.113	21.63
					22.64	0.0185	21.02

Table 11. (Continued)

Ts °C	τ psi	$\dot{\delta}$ in./min	w %	Ts °C	τ psi	$\dot{\delta}$ in./min	w %
Pc = 60psi; Pns = 20psi							
40	19.58	0.155	22.32	2	19.58	0.029	21.96
	17.54	0.096	21.85		18.57	0.0230	22.32
	15.51	0.021	21.69		17.54	0.0320	22.24
	13.47	0.0068	21.61		16.52	0.0084	22.03
Pc = 40psi; Pns = 40psi							
40	25.09	1.280	21.97	2	23.65	0.091	22.05
	23.65	0.500	21.66		22.64	0.110	21.83
	22.64	0.230	21.30		20.60	0.100	22.04
	20.60	0.220	21.27		20.60	0.021	21.68
	19.58	0.150	21.52		19.58	0.044	21.78
Pc = 20psi; Pns = 20psi							
40	14.49	0.625	23.01	2	14.49	0.195	23.24
	13.47	0.227	22.55		12.45	0.089	23.35
	12.45	0.150	22.71		11.44	0.022	22.72
	10.42	0.038	22.50		10.42	0.012	22.78
	10.42	0.030	21.91		16.52	1.650	23.55

Table 12. Direct shear tests on silt Series D1 ($\dot{\delta}$ = 0.05 in./min)

Pc psi	Pns psi	Tc °C	Ts °C	w %	τ_m psi
40	20	60	2	22.46	16.00
40	20	60	20	22.35	14.80
40	20	60	40	22.40	15.00
40	20	60	60	21.57	15.20
40	20	40	2	22.29	18.40
40	20	40	20	22.39	15.20
40	20	40	40	22.44	13.50
40	20	20	2	22.69	15.40
40	20	20	20	22.25	16.20
60	60	40	40	20.48	35.50
60	40	40	40	21.14	25.20

Table 12. (Continued)

Pc psi	Pns psi	Tc °C	Ts °C	w %	τm psi
60	20	40	40	21.92	16.00
40	40	40	40	21.23	24.00
20	20	40	40	22.38	12.80
60	60	40	2	20.88	35.00
60	40	40	2	21.41	26.80
60	20	40	2	22.23	21.80
40	40	40	2	21.64	23.40
20	20	40	2	22.97	12.00
60	20	20	2	21.74	18.00
60	20	20	20	21.56	17.00
60	60	20	2	20.55	37.80
60	60	20	20	20.04	35.60
40	20	20	2	22.63	14.40

normal stresses of 45 psi. The soil for Series B3 was also individually mixed with water but the specimens were immediately compacted and tested with no time allowed for curing. The specimens for Series B1 to B3 inclusive were consolidated in general until at least 100% of primary consolidation was complete and about 3 hours were allowed for changing the temperature. Thus the total time of test varied depending on the test temperatures. The principal direct shear tests on clay are contained in Series B4 and B5. For these tests the soil was batch mixed and stored in a loose condition. The normal loads were 60 psi during both phases of the tests. Tables 5 to 9 inclusive give values of Pc, Tc, Ts, $\dot{\delta}$, w and the resulting value of τm. The time allowed for consolidation (about 10 hours for Series B4 and 19 hours for Series B5) was the main difference in these series.

The 129 creep tests performed on the silt were divided into two series (C1 and C2) and the data are given in Tables 10 and 11 respectively. The soil for these tests was mixed in a batch and stored in a humid room for several weeks prior to testing as discussed under Soil preparation. The table gives the values of all the controlled variables, water content and the resulting strain rate.

Series D1 consists of 24 direct shear tests on the silt and the data for these tests are given in Table 12. The soil was prepared the same as for the creep tests and the table gives the values of controlled variables, water content and resulting shear strength.

Due to the large number of tests performed, it was not feasible to include all primary data in either tabular or graphical form. Data for the creep tests were obtained as continuous recorder plots of deformation against time along with independently recorded values of deformation taken as a check. Six curves each as examples for both clay and silt respectively are shown in Figures 7 and 8. These curves show the effect of different shear stresses at fixed levels of the other variables. They are essentially the same as those published by others for soils and other materials. In the direct shear tests, shear stress was recorded at regular intervals of deformation. Plots of these values gave typical curves as illustrated in Figure 1 where curve 'a' shows the relationship obtained for the clay and curve 'b' that for silt.

For the creep tests on both soils, the rate of deformation was obtained by drawing tangents at regular intervals to the curves from the recorder plots of deformation versus time. The rate of deformation, $\dot{\delta}$, was then plotted against the corresponding deformation, δ. Examples of these plots are given for the clay and silt in Figures 9 and 10 respectively which correspond to the deformation time curves of Figures 7 and 8. Figures 11 through 26 show the results of analyses of creep test data.

Results of the direct shear tests on clay, Series B2, B4 and B5 are shown as plots of τm versus $\frac{1}{Ts}$ in Figures 27, 28 and 29 respectively. Each value of Tc gives a different line having a slope of $\frac{\Delta H}{\beta k}$ in accordance with Equation 18. Results of direct shear tests on silt are shown as τm versus $\frac{1}{Ts}$ in Figure 30 and as τm versus Pc and Pns in Figure 31. This latter plot gives strength envelopes for two values of Ts.

Typical time curves for consolidation of clay and silt showing the effect of temperature on the rate of consolidation are shown in Figure 33. Consolidation void ratio-log pressure curves for both soils are shown in Figure 34 and indicate the effect of temperature on compressibility.

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 previously illustrated in Figure 2 and shown in Figure 8. Schoeck (1957) stated that the basic equation for creep (Equation 12) 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

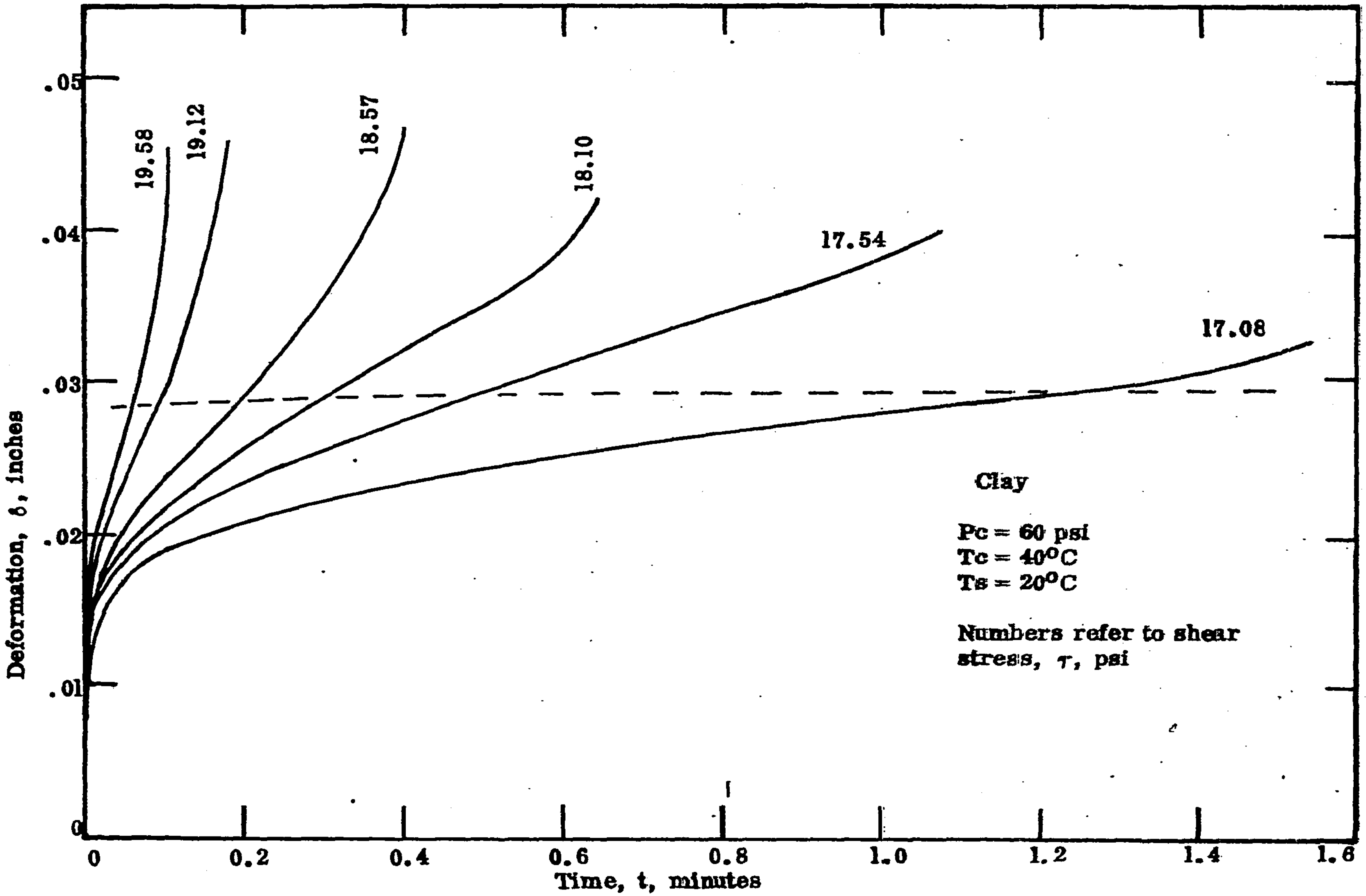

Figure 7. Sample deformation-time curves for clay

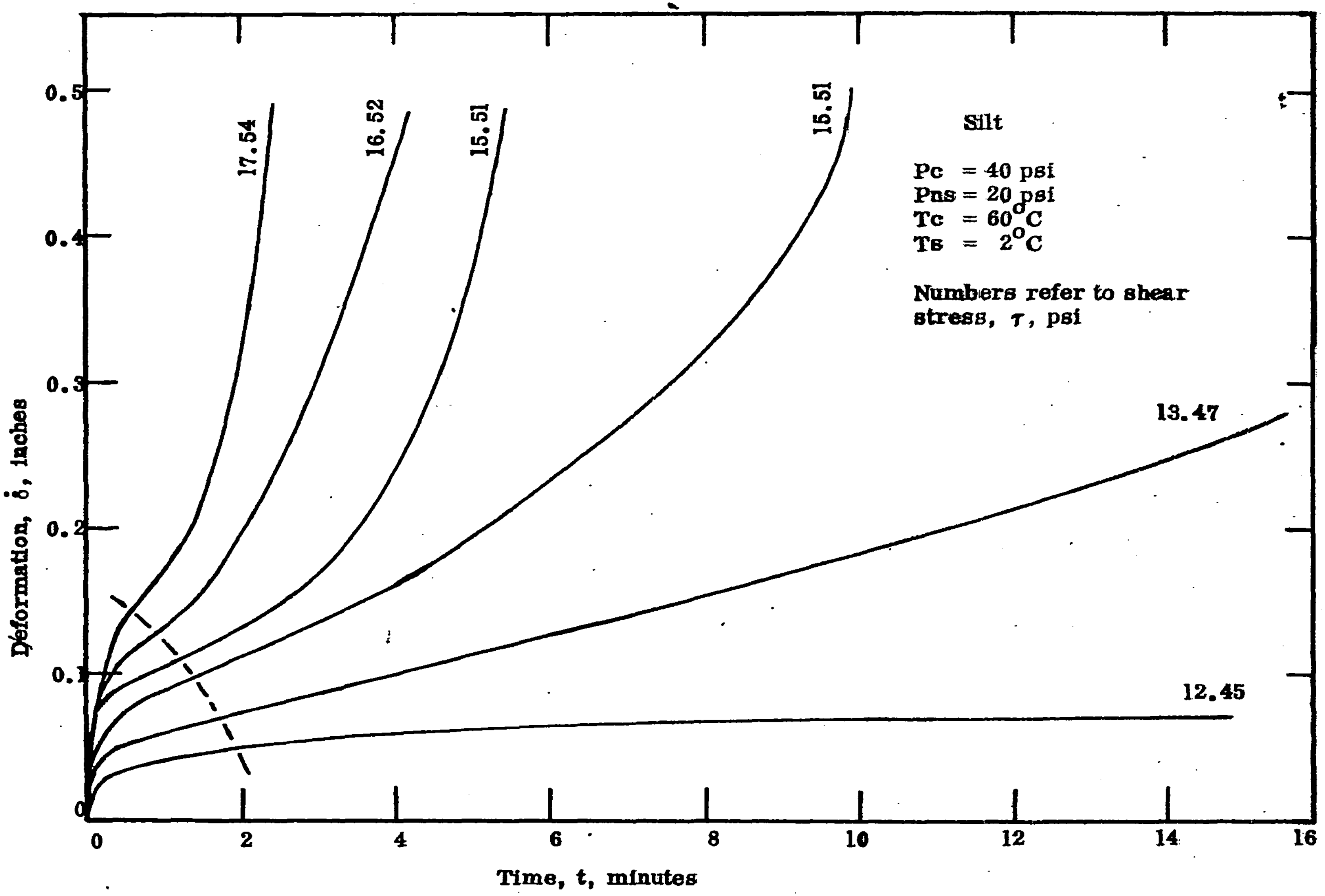

Figure 8. Sample deformation-time curves for silt

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 as previously discussed. An inspection of the basis of Equation 12, i.e. the rate of passage of flow units over an energy barrier, would indicate 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 should be the same for each specimen. For a given pre-test history (consolidation temperature, stress, etc.), the structure of the soil at the beginning of the tests should be essentially constant. Variations during the tests are probably related to deformation, shear stress and temperature, with deformation having the primary influence. Singh and Mitchell (1968) worked entirely with type 'd' curves (Figure 1) 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 Equation 12.

In this study, the minimum points of the rate-of-deformation versus deformation curves, which are equivalent to points of $\ddot{\delta}$ equal to zero, (based on type 'c' deformation), were joined by a straight line which was then extrapolated to intersect the type 'd' curves to satisfy the above two criteria. Examples of these lines are shown in Figures 9 and 10 for the clay and silt respectively. The points of intersection were transferred to the deformation-time curves (Figures 7 and 8) to show the relationship with these curves. For all the tests on clay the lines either were approximately vertical, indicating constant deformation to develop a minimum rate (illustrated by Figure 9), or had a large negative slope, indicating slightly greater deformation required at lower stresses. For all the tests on silt the intersecting line had a positive slope in the order of 3:1 as illustrated in Figure 10. This slope may be related to dilational energy.

The points of intersection obtained from the plots of deformation rate versus deformation (Figures 9 and 10) give the values of minimum deformation rate reported in Tables 3, 4, 10 and 11. These values for all creep tests were then plotted against shear stress and are shown in Figures 11 to 16 inclusive for clay and Figures 17 to 22 inclusive 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 and at a given level of shear stress gives the value of $\frac{\Delta H}{k}$. These latter relationships are shown plotted in Figures 23 to 26 inclusive.

The deformation rate-shear stress plots 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 initial water content could not be easily determined but may be reflected to some degree in the water content at the end of the test, which was determined for almost all specimens. For given temperatures of consolidation and shear and a given normal stress,

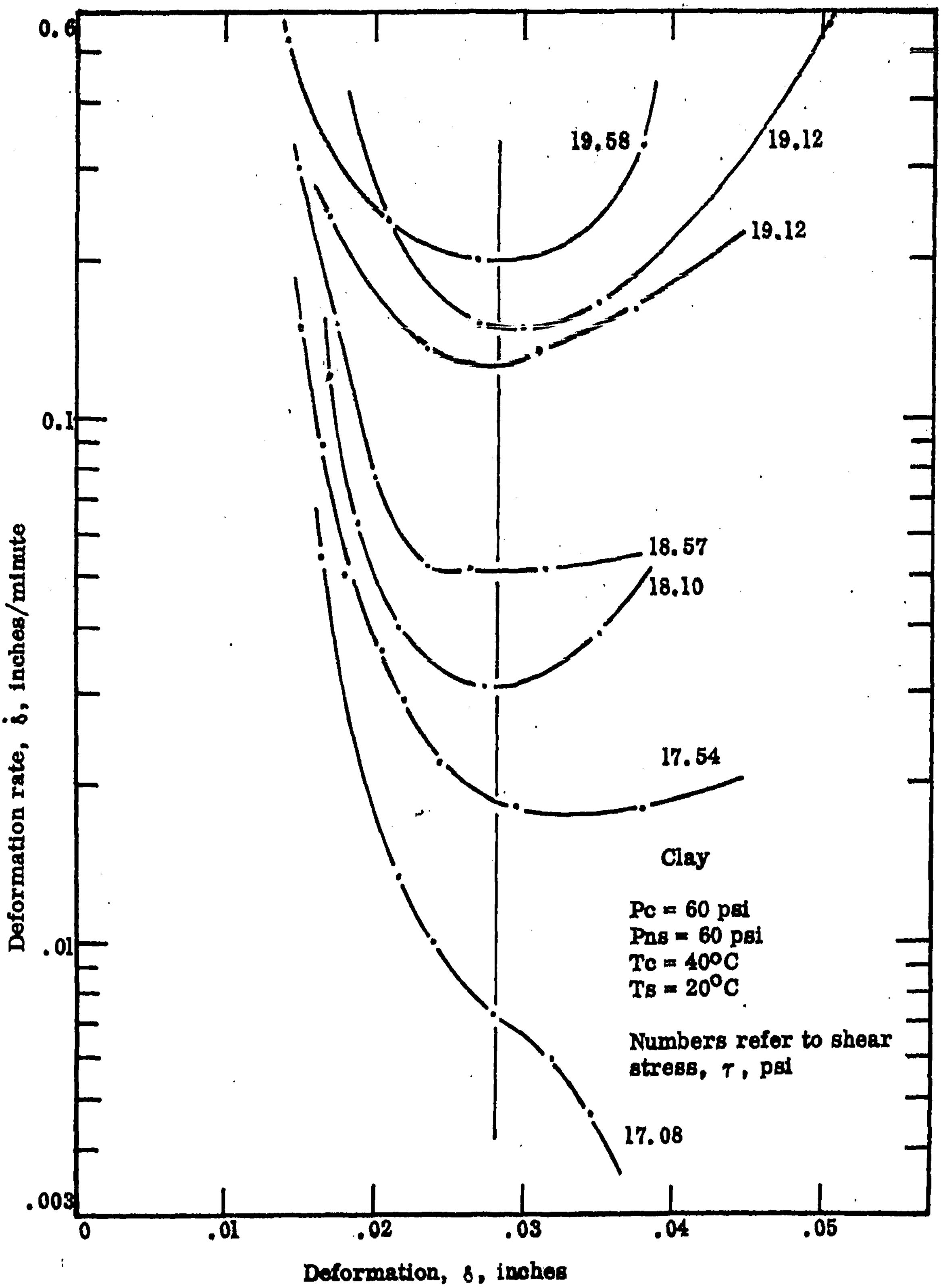

Figure 9. Sample deformation rate curves for clay

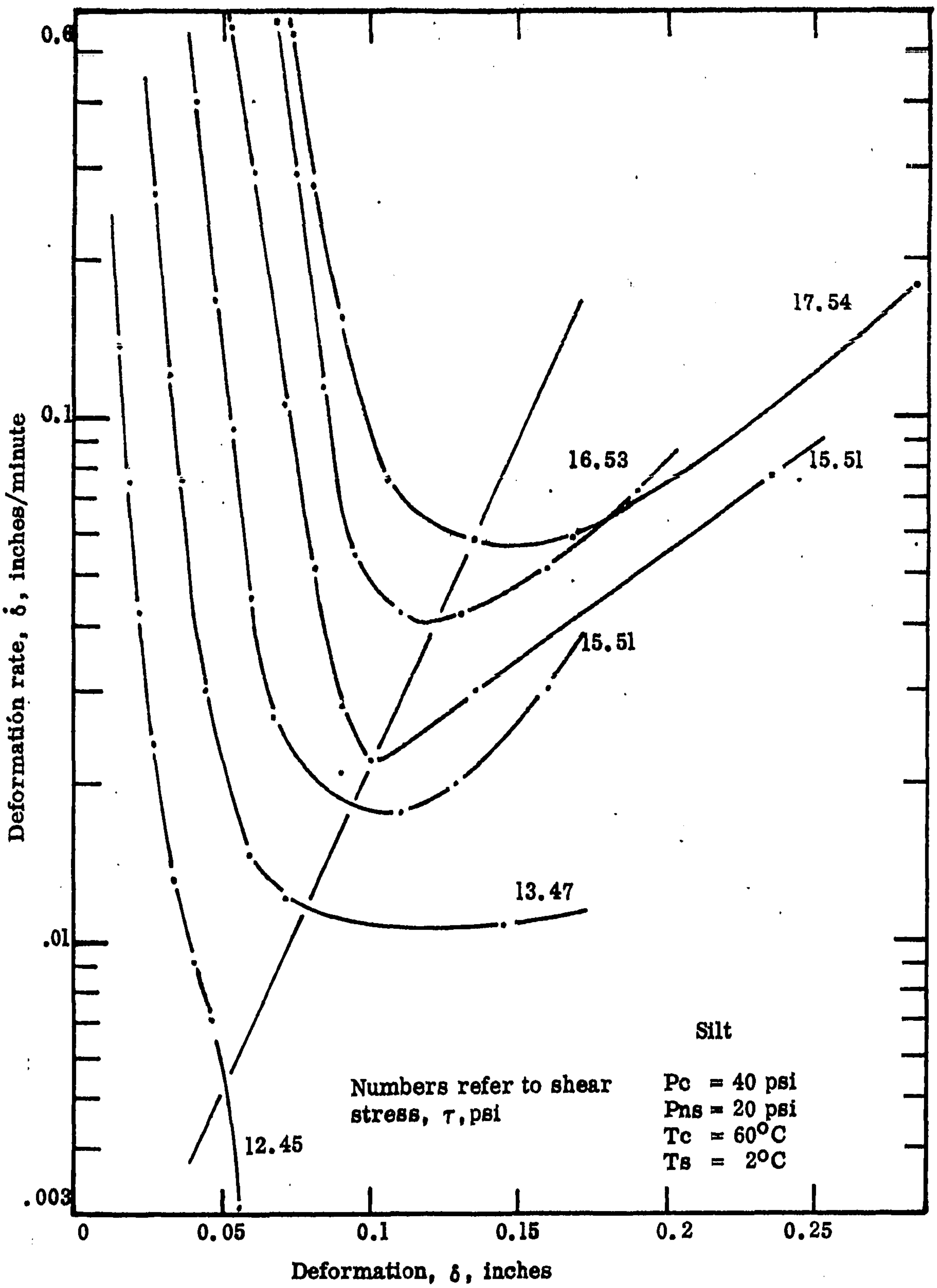

Figure 10. Sample deformation rate curves for silt

the variation of water content was generally less than ½% of the dry weight of soil but variations of over 1% occasionally occurred. However, differences in temperature of consolidation and, most particularly, of shear resulted in water content differences of over 2% for the clay. A part of the variation may be attributed to non-uniform 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 equations (Equation 18 and 19) had all terms linear in the variables which were also assumed to be independent, and there were a reasonable number of data points for most test series, the advantages of treating the data by least squares fitting was obvious. The facilities of the Computation Center, Iowa State University of Science and Technology including a multiple regression program and an I.B.M. 360 computer were used to treat all experimental data.

Regression analysis

The dependent variables (τm in Equation 18 and $\ln \dot{\epsilon}$ in Equation 19) were regressed on the applicable independent variables in order to obtain values for the linear coefficients in the equations. The test results were treated by considering various groups of tests in which some of the parameters were constant and the regression was carried out on the remaining variables. Further, all similar tests on a given soil could be treated as one group and the regression carried out on all the variables in the equations. This leads to a large number of possible combinations;

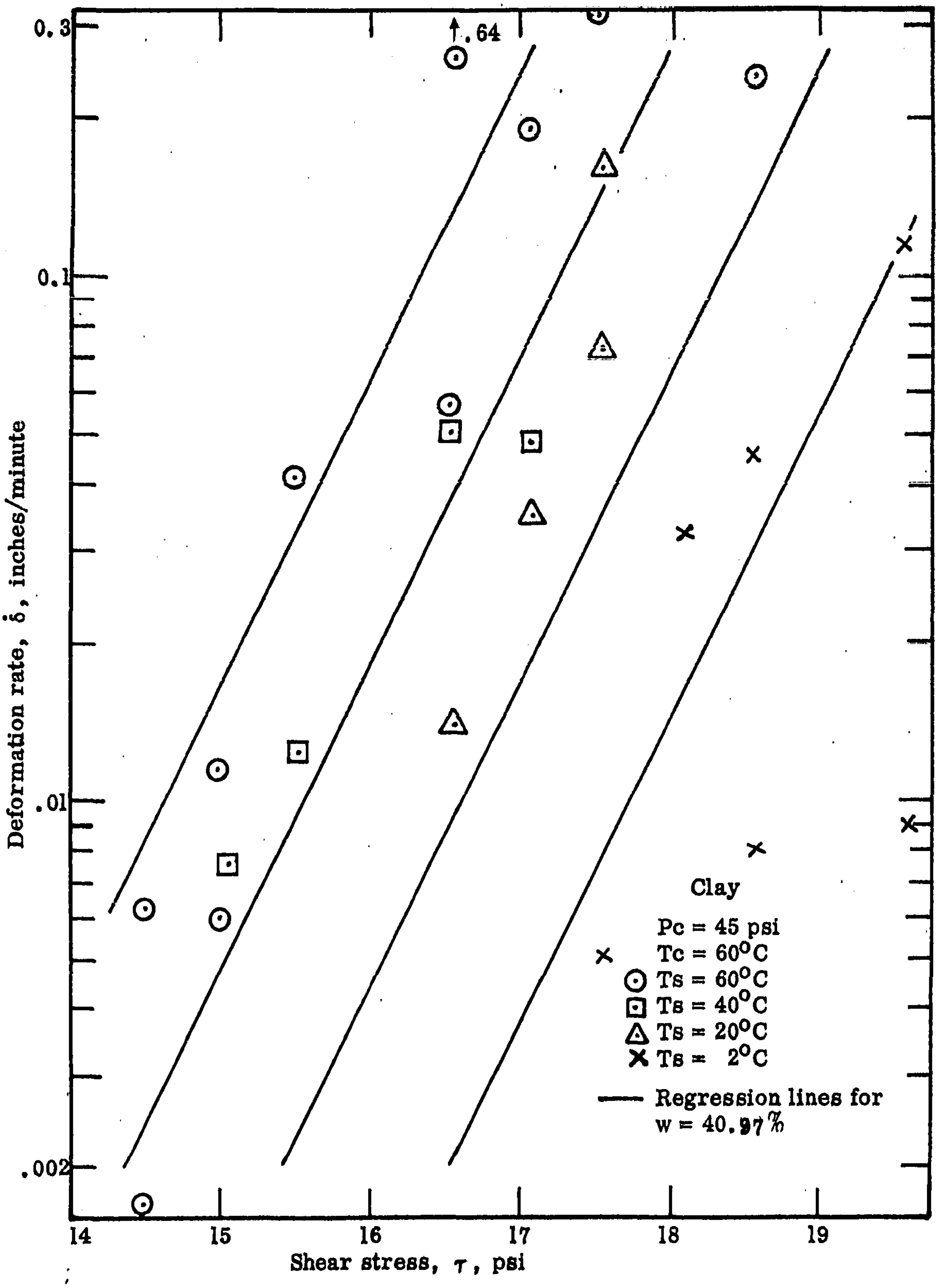

Figure 11. Deformation rate variation with shear stress for clay with Pc = 45 psi and Tc = 60°C

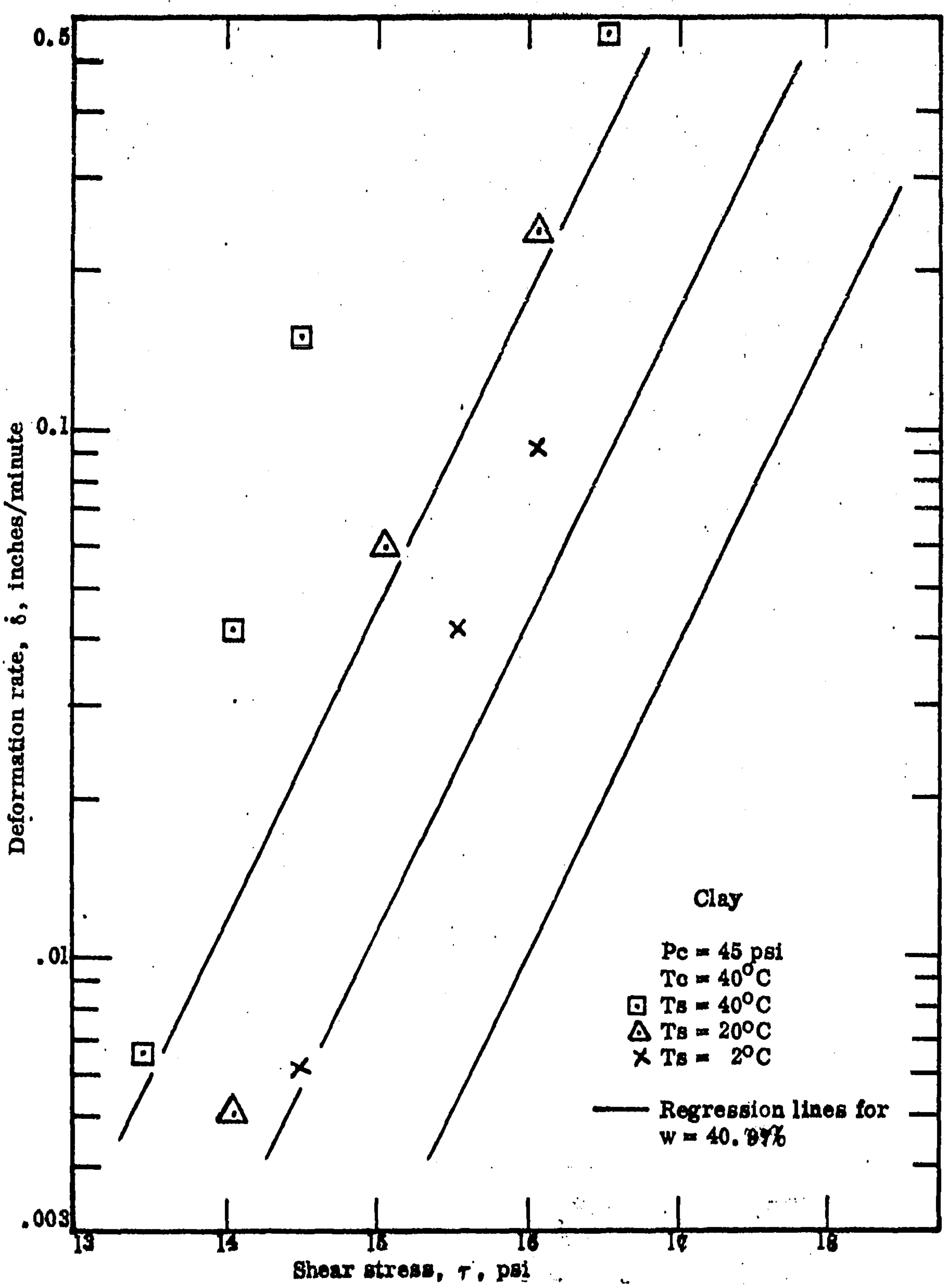

Figure 12. Deformation rate variation with shear stress for clay with Pc = 45 psi and Tc = 40°C

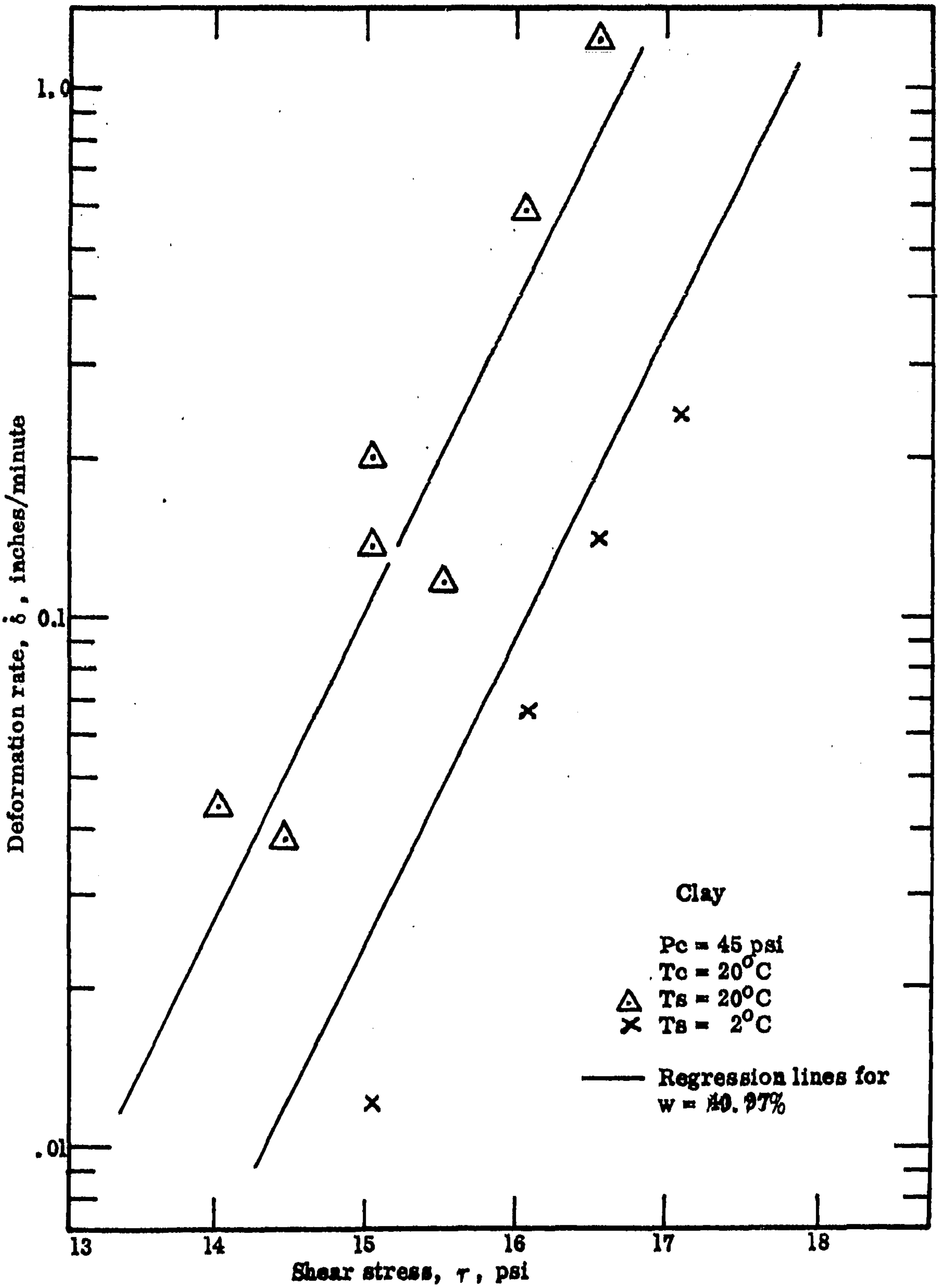

Figure 13. Deformation rate variation with shear stress for clay with Pc = 45 psi and Tc = 20°C

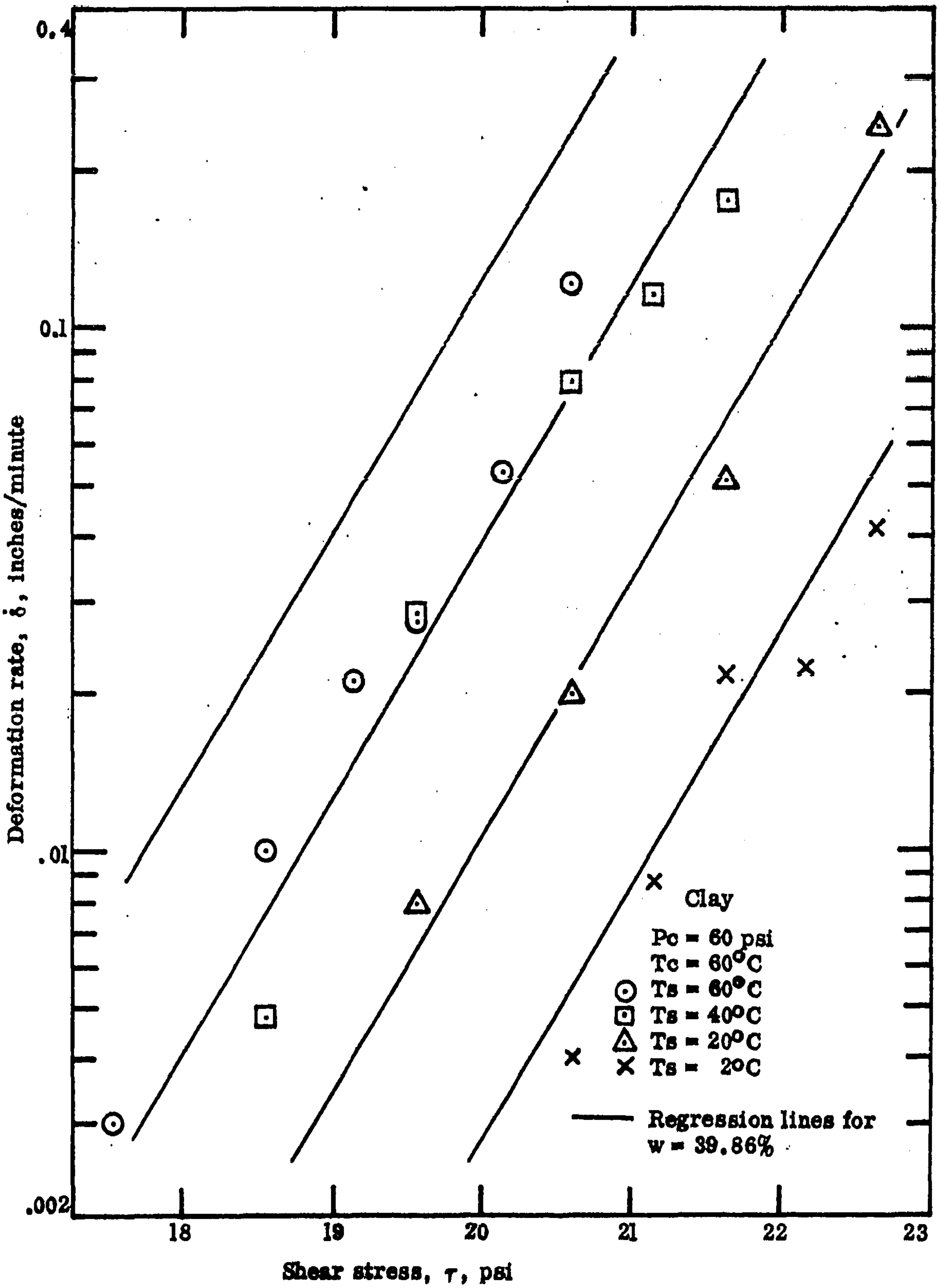

Figure 14. Deformation rate variation with shear stress for clay with Pc = 60 psi and Tc = 60°C

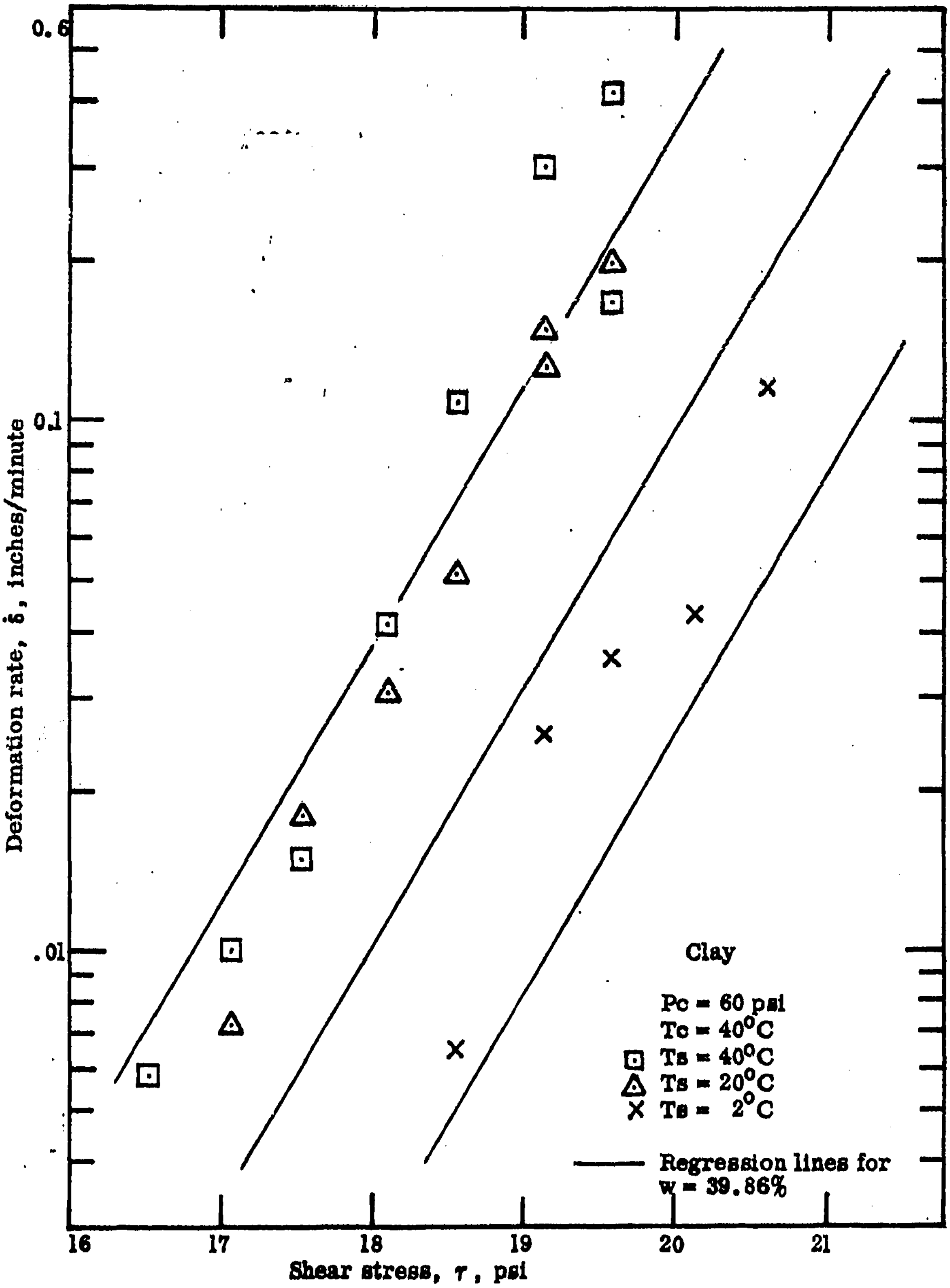

Figure 15. Deformation rate variation with shear stress for clay with Pc = 60 psi and Tc = 40°C

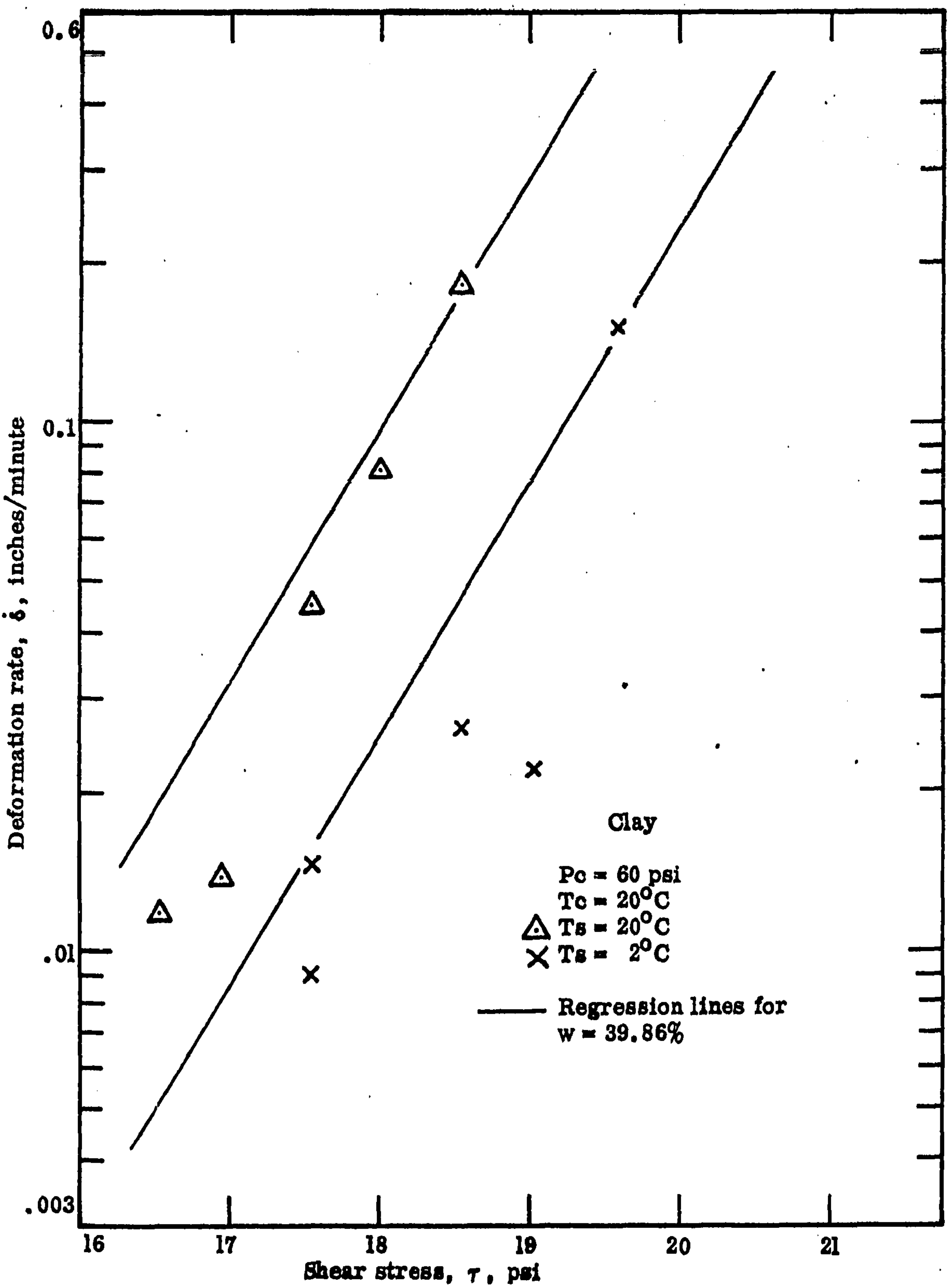

Figure 16. Deformation rate variation with shear stress for clay with Pc = 60 psi and Tc = 20°C

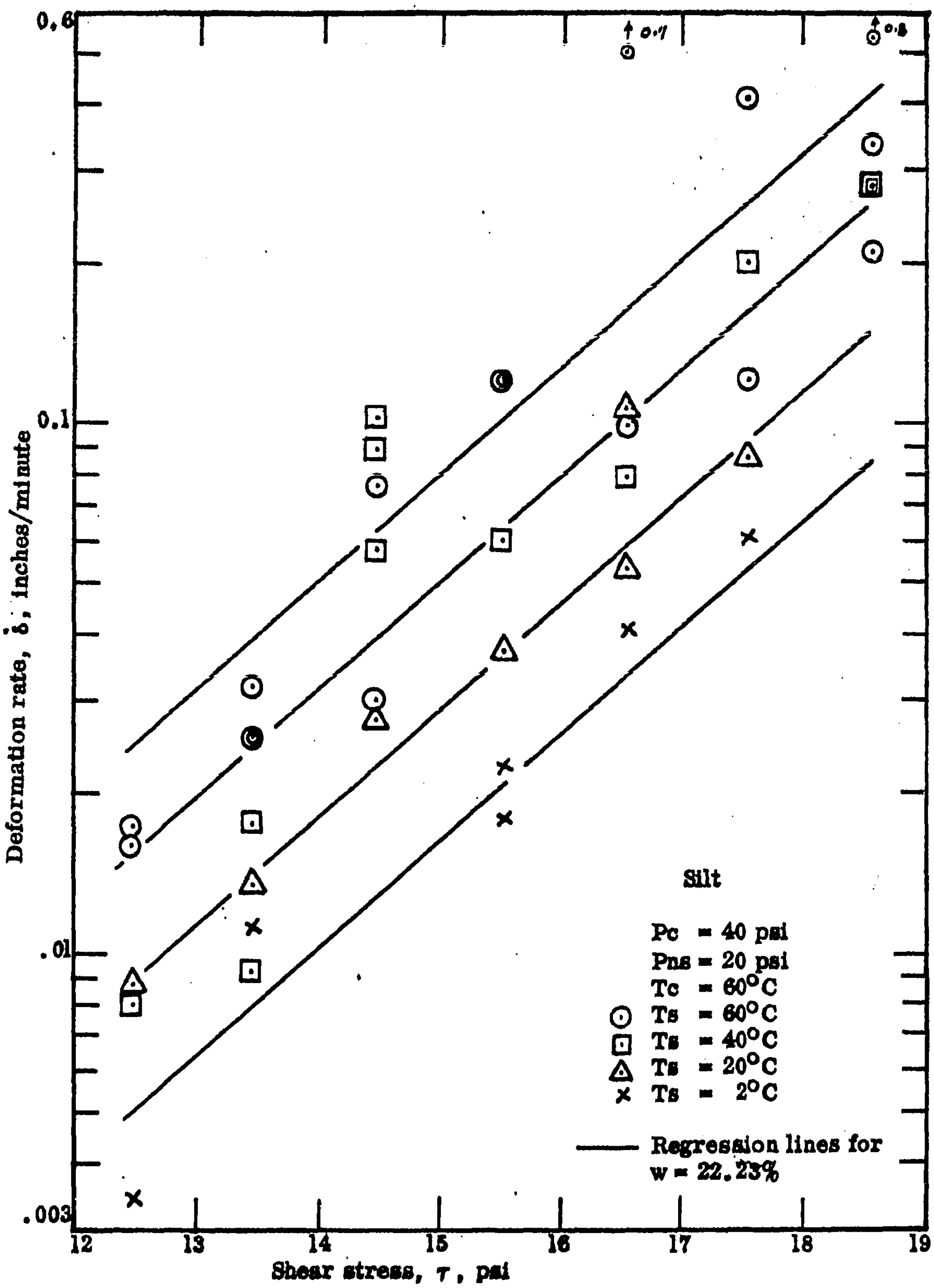

Figure 17. Deformation rate variation with shear stress for silt with Pc = 40 psi, Pns = 20 psi and Tc = 60°C

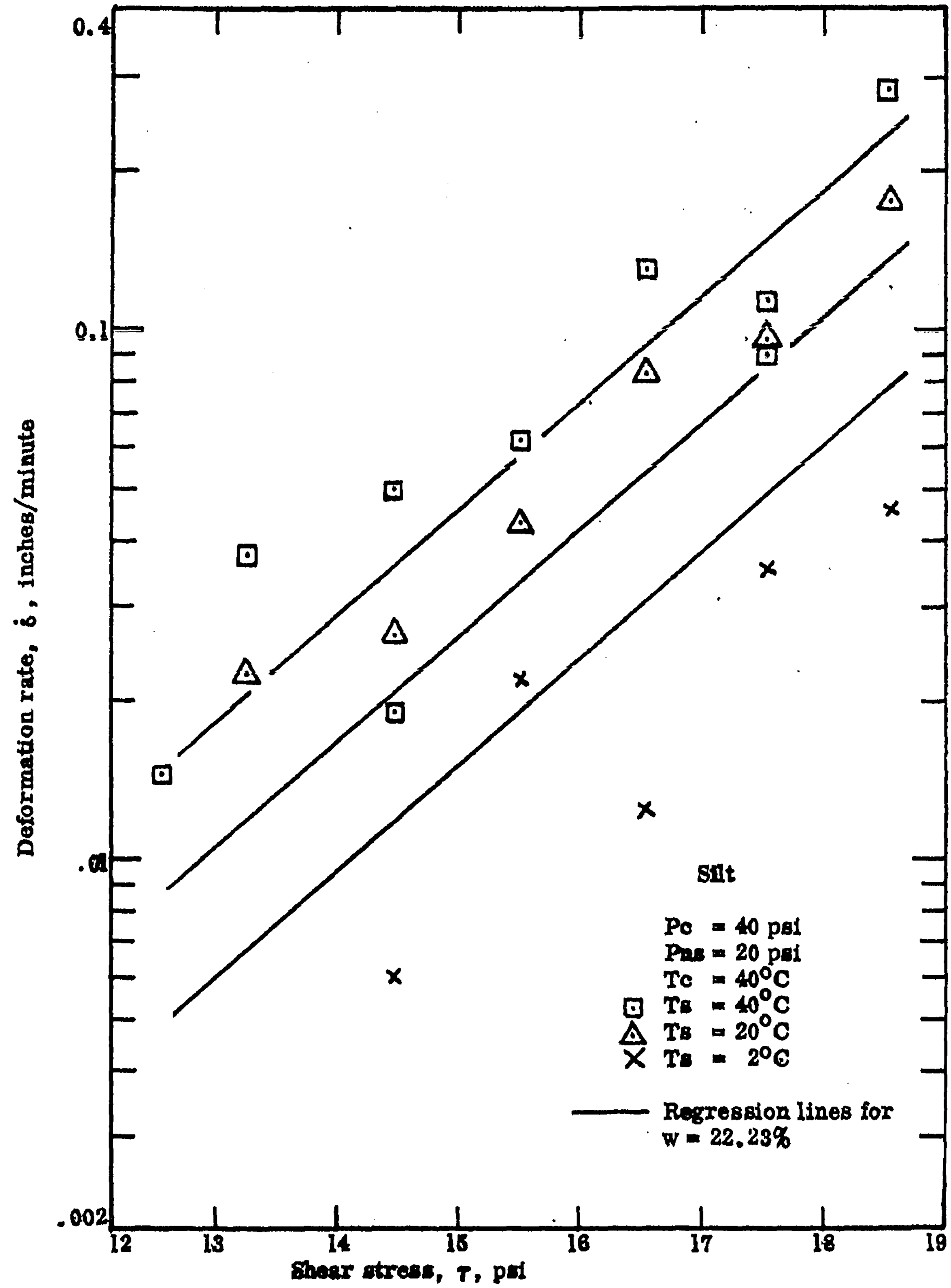

Figure 18. Deformation rate variation with shear stress for silt with Pc = 40 psi, Pas = 20 psi and Tc = 40°C

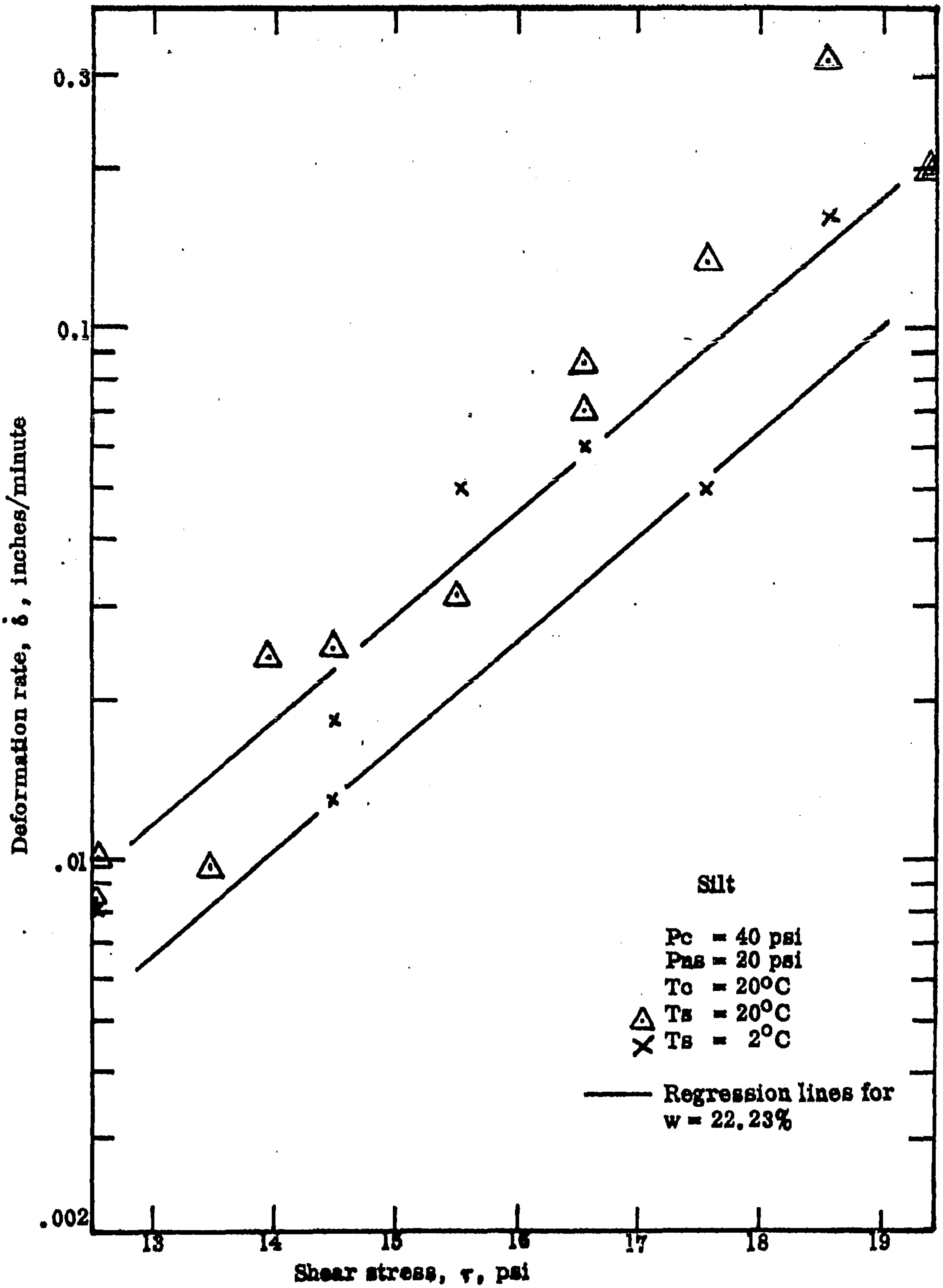

Figure 19. Deformation rate variation with shear stress for silt with Po = 40 psi, Pas = 20 psi and To = 20°C

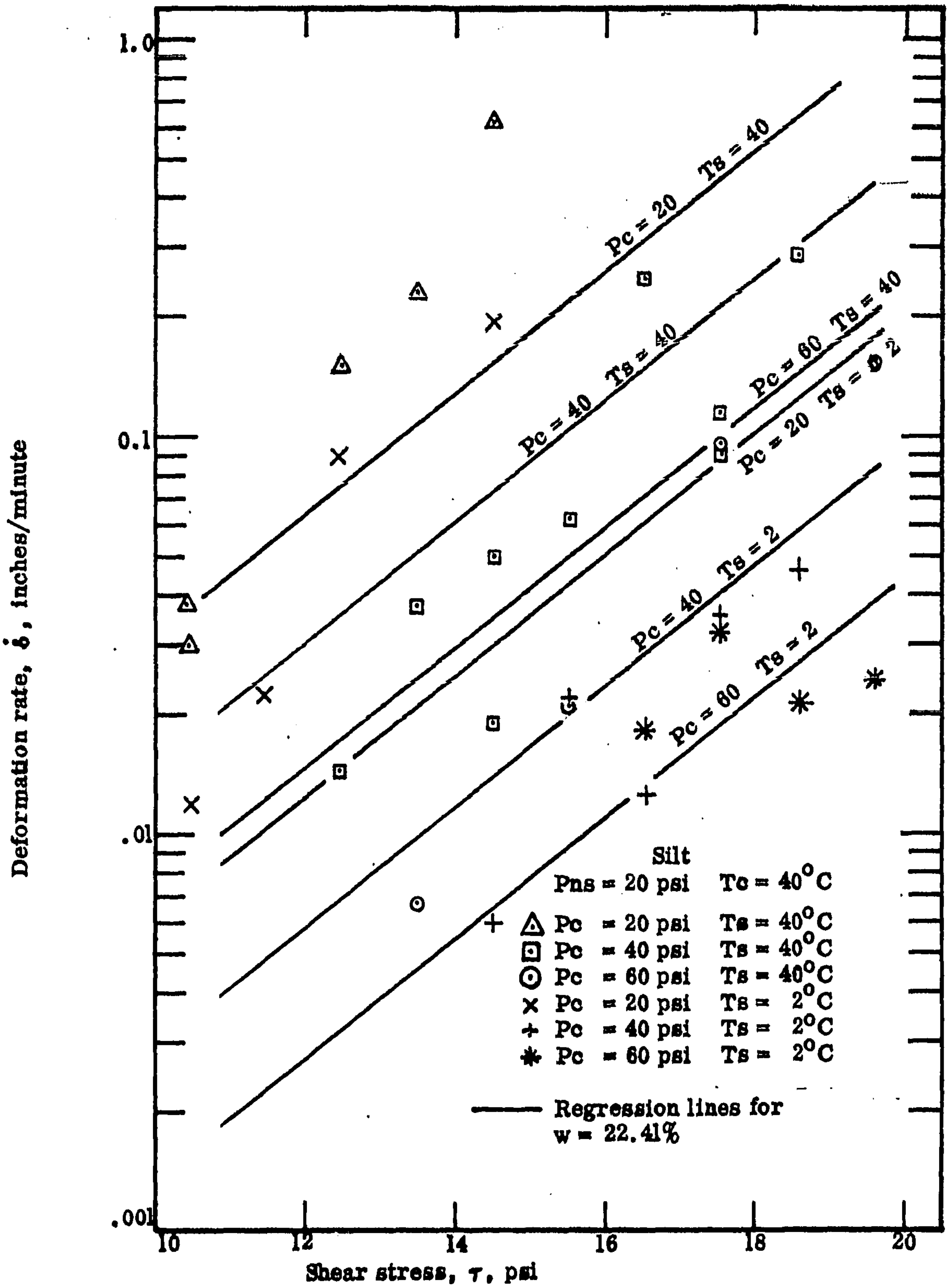

Figure 20: Deformation rate variation with shear stress for silt with Pns = 20 psi and Tc = 40°C

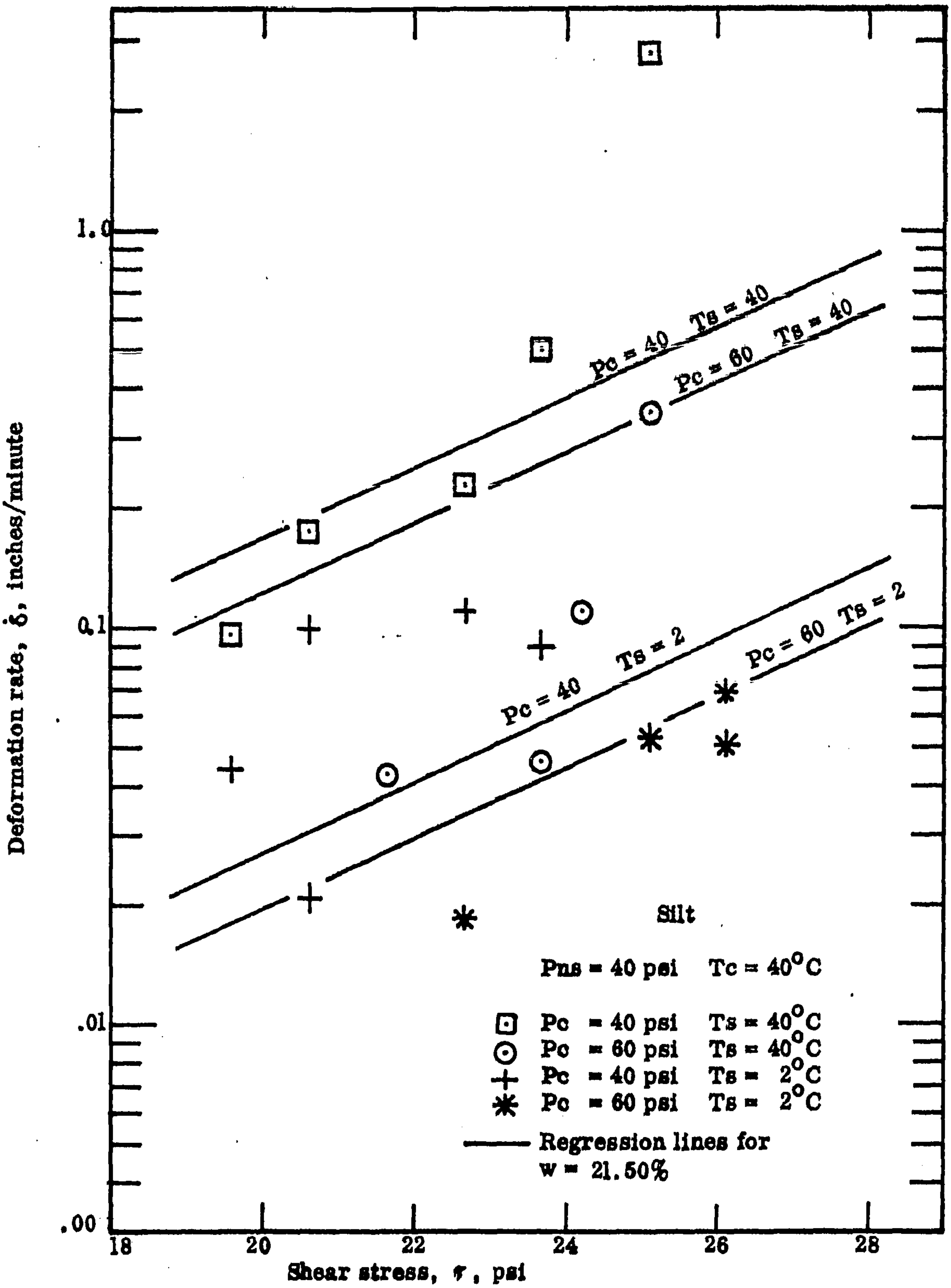

Figure 21. Deformation rate variation with shear stress for silt with Pns = 40 psi and Tc = 40°C

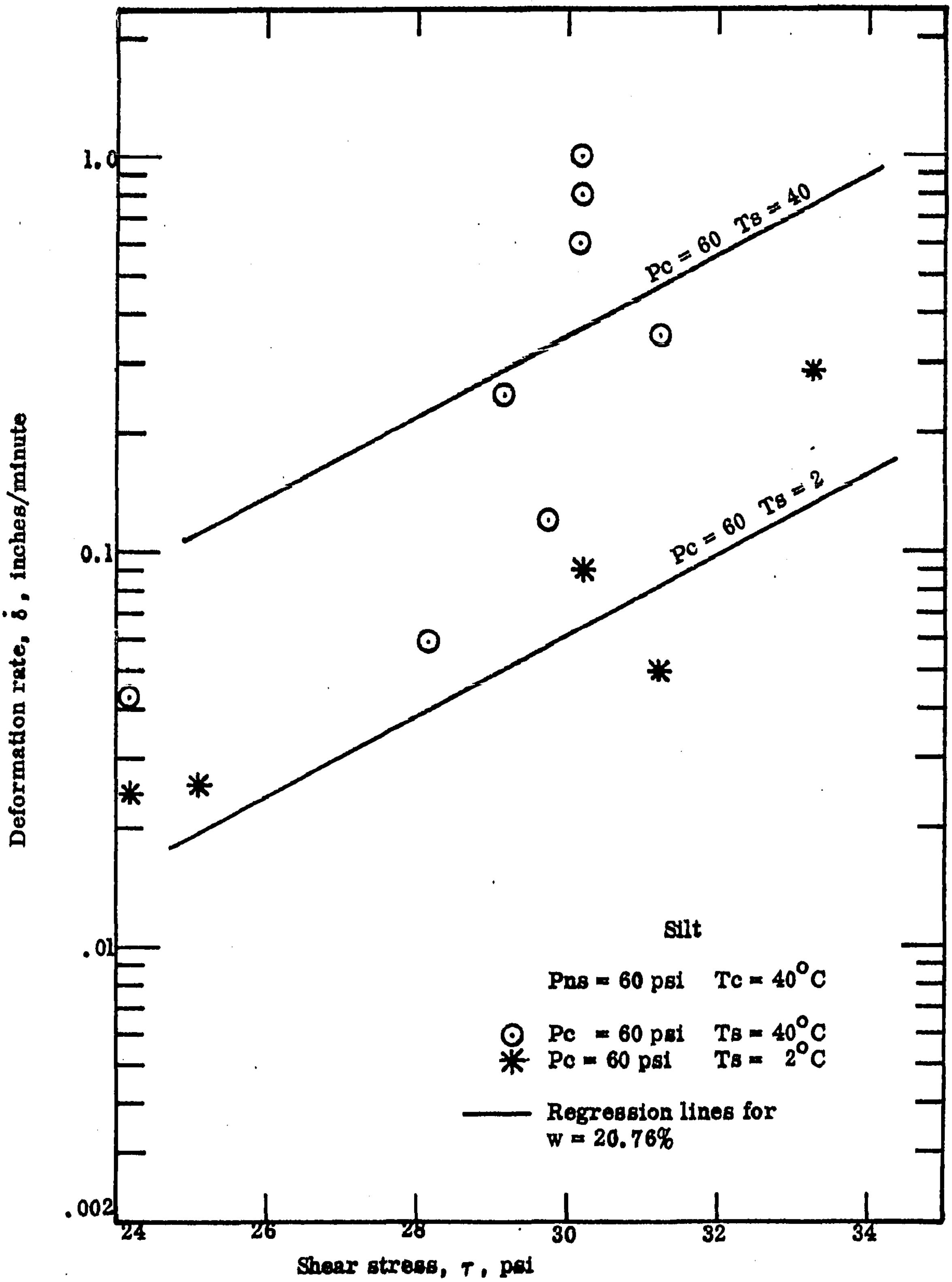

Figure 22. Deformation rate variation with shear stress for silt with Pns = 60 psi and Tc = 40°C

the results of the most significant of these are included in Tables 13 to 21 inclusive.

The statement on page 418 of Snedecor (1966) is pertinent to the interpretation of regression results where different variables are used. He states, "...both estimates and tests depend on the independent variables included in the regression. In any one regression the estimates and tests are correlated; the whole complex changes if independent variables are added or deleted. In this sense, statements made about the predictive value of a variable are not unique; they depend upon the other variables being used in the regression". The effect of this is evident in the regression results presented, where it can be seen that the values of the coefficients change as more variables are considered. In most cases, however, the inclusion of more independent variables in the analysis involved an increase in the number of tests considered and this alone would tend to change the values of the coefficients.

All experimental results were treated by multiple regression and the results of these analyses are reported in Tables 13 to 21 inclusive. The tables give the test series; values of variables held constant for each particular analysis; values of the square of the multiple correlation coefficient, R^2; standard errors for the regression, Se, (given in the tables below R^2); degrees of freedom for residual sum of squares, d.f.; values of the coefficients for the variables and the intercept; standard errors for the coefficients (given in the tables below the values of the coefficients). Average values of water content, w, and mass specific gravity, Gm, for the group of tests considered are also included in the tables for comparison. The standard errors give measures of the variation of predicted values and of the coefficients. The intercept includes the constant values of variables not included in the particular regression analysis or, in the case of a series where all variables are considered, the intercept should be the value of lnM in Equations 18 and 19. Creep tests on clay were first considered as individual groups within the series, and were separated according to Tc and Ts. The $\ln \dot{\delta}$ was regressed on τ and w. This grouping corresponds to the individual sets of points for a given temperature shown in Figures 11 to 16 inclusive. The results of these regression analyses are given in Table 13 for Series A1 and Table 14 for Series A2. Although these results were not plotted, the analysis was carried out to determine whether there were any trends or relationships between the variables. Specifically for these series, any variation of β with Ts would be distinguishable. As can be seen by considering the values of β and their standard errors in Tables 13 and 14, there are no significant trends with Ts. Also within these series, the coefficient of water content, $\beta\gamma$, does not show any significant trends and, in fact, the values are not significant in most cases. Due to the small number of samples involved (as reflected in the residual degrees of freedom which vary from 0 to 5), small trends in the coefficients would not be distinguishable.

All tests in a given series having the same value of Tc were next combined and $\ln \dot{\delta}$ was regressed on $\frac{1}{Ts}$, τ and w. The results of these analyses for clay are shown in Table 15 where lines 1, 2 and 3 are the results for Series A1 and lines 5, 6 and 7 are the results for Series A2.

Table 13. Regression results for creep tests on clay Series A1 (Pc = 45psi)

Line	Tc °C	Ts °C	R^2 S_e	d.f.	Intercept	β	$\beta\gamma$	Av w %	Av Gm
1	60	60	.9300		-71.022	1.442	1.118	39.60	1.276
			.6522	5	27.375	0.259	.744		
2	60	40	.9205		-29.874	1.153	0.185	40.58	1.285
			.5168	1	196.567	1.346	4.310		
3	60	20	.9224		-95.032	1.770	1.498	41.12	1.288
			.4790	1	93.449	0.627	2.369		
4	60	2	.9784		-96.746	0.296	2.110	40.52	1.311
			.3217	1	29.635	0.506	0.909		
5	40	40	.8899		-116.993	1.761	2.092	42.26	1.272
			1.221	1	96.223	0.637	2.148		
6	40	20	1.000		-66.609	1.835	0.852	41.99	1.273
			0	0	0	0	0		
7	40	2	1.000		-14.010	1.761	-0.392	42.30	1.274
			0	0	0	0	0		
8	20	20	.9024		-38.321	1.434	0.360	40.67	1.307
			.4693	5	19.633	0.211	0.469		
9	20	2	.9986		-4.080	1.561	-0.592	40.39	1.320
			.083	1	13.573	0.080	0.358		

Table 14. Regression results for creep tests on clay Series A2 (Pc = 60psi)

Line	Tc °C	Ts °C	R^2 Se	d.f.	Intercept	β	$\beta\gamma$	Av w %	Av Gm
1	60	60	.9936		-37.010	1.170	0.275	38.81	1.327
			.1332	3	9.197	0.055	0.230		
2	60	40	.9755		-26.178	1.184	-1.338	39.84	1.317
			.3270	2	52.043	0.133	1.307		
3	60	20	.9982		30.496	1.202	-1.478	40.03	1.324
			.1065	1	18.584	0.059	0.483		
4	60	2	.9624		-51.206	1.113	0.573	39.96	1.334
			.2508	2	19.091	0.156	0.463		
5	40	40	.9527		-42.566	1.451	0.329	39.95	1.326
			.4157	5	12.732	0.157	0.277		
6	40	20	.9918		-20.169	1.381	0.207	40.22	1.324
			.1362	4	4.460	0.066	0.119		
7	40	2	.9120		-35.164	1.233	0.186	40.41	1.328
			.4354	2	68.787	0.273	1.712		
8	20	20	.9932		-50.005	1.360	0.580	39.62	1.317
			.1356	2	10.094	0.087	0.263		
9	20	2	.9168		-62.098	0.931	1.033	39.93	1.316
			.4348	2	21.132	0.247	0.542		

Table 15. Coefficients of Equation 19 obtained by regression for creep tests on clay using combined groups of tests

Line	Tc °C	Ts °C	Pc psi	R^2 Se	d.f.	Intercept	$\frac{\Delta H}{k}$	α	β	$\beta\rho$	$\beta\gamma$	Av w %	Av Gm
1	60	60											
		40	45	.8671		-46.116	7.939		1.317		1.143	40.59	1.284
		20		.6039	16	11.842	1.265		0.156		0.381		
		2											
2	40	40	45	.9128		-59.956	4.509		1.660		1.120	42.19	1.273
		20		.6127	6	26.667	1.119		0.214		0.616		
		2											
3	20	20	45	.9345		-3.245	9.625		1.427		0.308	40.57	1.312
		2		.3798	8	16.532	1.320		0.142		0.359		
4	60	60											
	40	40	45	.8689		-30.640	6.558	-0.114	1.354		0.805	40.97	1.287
	20	20		.6064	37	4.487	0.689	0.008	0.096		0.123		
		2											
5	60	60											
		40	60	.9690		-30.158	5.883		1.138		0.570	39.60	1.326
		20		.2416	16	5.155	0.378		0.054		0.152		
		2											
6	40	40		.8663		-13.658	4.832		1.246		0.091	40.16	1.326
		20	60	.5228	16	9.603	0.818		0.126		0.248		
7	20	20	60	.9356		-31.907	7.735		1.117		0.896	39.78	1.317
		2		.3345	6	12.608	1.139		0.142		0.351		
8	60	60											
	40	40	60	.8292		-28.274	5.949	-0.111	1.118		0.711	39.86	1.324
	20	20		.5386	45	5.871	0.540	0.009	0.079		0.161		
		2											
9	60	60											
	40	40	45	.8423		-17.814	6.137	-0.112	1.206	0.219	0.754	40.36	1.307
	20	20	60	.5772	86	3.748	0.428	0.006	0.060	0.016	0.095		
		2											

Table 16. Coefficients of Equation 18 obtained by regression for direct shear tests on clay

Series	Tc °C	Ts °C	Pc psi	R^2 Se	df	Intercept	$\frac{\Delta H}{\beta k}$	$\frac{\alpha}{\beta}$	$\frac{1}{\beta}$	ρ	γ	Av w %	Av Gm
B1	14.0-	14.0-	15-	.9505		-15.901	12.288	0.138	8.062	0.144	0.733	35.85	1.407
	59.5	58.8	120	1.507	32	9.273	2.082	0.021	1.165	0.012	0.152		
B2	65	1.5-	45	.7747		-18.324	7.977				0.520	38.69	1.373
		65		.9699	51	6.666	0.617				0.196		
B1 &	14-	2-	15-	.8906		-5.022	7.361	0.074	1.173	0.148	0.467	38.21	1.370
B3	60	60	120	2.411	57	6.786	1.933	0.025	0.544	0.017	0.129		
B4	2-	2-	60	.9318		-16.258	5.303	0.098			0.501	39.26	1.346
	60	60		.4785	31	8.066	0.547	0.006			0.228		
B5	20-	2-	60	.8738		-30.714	6.366	0.094			0.936	39.26	1.340
	60	60		.5843	33	5.987	0.901	0.008			0.197		
B2, 4	2-	1.5-	45-	.9404		-27.402	7.239	0.103	0.920	0.111	0.680	39.00	1.355
& 5	65	65	60	.8109	126	4.525	0.420	0.007	0.202	0.027	0.128		
B1-B5	2-	1.5-	15-	.8596		-11.898	8.267	0.143	1.934	0.112	0.654	38.76	1.360
Inc.	65	65	120	1.788	183	3.326	0.666	0.009	0.184	0.010	0.076		

Table 17. Comparison of coefficients of Equation 11 obtained by regression for creep and direct shear tests on clay

Series	Tc °C	Ts °C	Pc psi	Intercept	$\frac{\Delta H}{\beta k}$	$\frac{a}{r}$	$\frac{1}{\beta}$	ρ	γ	Av w %	Av Gm
					Expected coefficients from creep tests						
A1	60 40 20	60 40 20 2	45	-22.629	4.843	0.084	0.739		0.595	40.97	1.287
A2	60 40 20	60 40 20 2	60	-25.290	5.321	0.099	0.894		0.636	39.86	1.324
A1 & A2	60 40 20	60 40 20 2	45 60	-14.771	5.089	0.093	0.829	0.182	0.625	40.36	1.307
					Experimental coefficients from direct shear tests						
B1	14-59.5	14-58.8	15-120	-15.901	12.288	0.138	8.062	0.144	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	0.074	1.173	0.148	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
B2, B4 & B5	2-65	1.5-65	45 60	-27.402	7.239	0.103	0.920	0.111	0.680	39.00	1.355
B1-B5 Inc.	2-65	1.5-65	15-120	-11.898	8.267	0.143	1.934	0.112	0.654	38.76	1.360

Table 18. Regression results for creep tests on silt Series C1 (Pc = 40, Pns = 20)

Line	Tc	Ts	R^2 Se	Residual d.f.	Intercept	β	$\beta\gamma$	Av w %	Av Gm
	60	60	.8346		-23.212	0.514	0.579	22.19	1.682
1			.5653	13	7.761	0.068	0.353		
	60	40	.9315		-41.223	0.460	1.425	22.00	1.677
2			.3677	8	7.840	0.058	0.367		
	60	20	.9810		-33.989	0.478	1.058	22.07	1.689
3			.1558	4	7.323	0.035	0.333		
	60	2	.9548		-21.419	0.518	0.428	22.32	1.686
4			.2795	3	13.625	0.066	0.613		
	40	40	.9793		-50.852	0.387	1.888	22.25	1.700
5			.1577	6	6.457	0.027	0.293		
	40	20	.9766		-12.366	0.398	0.139	22.54	1.697
6			.1570	3	6.213	0.046	0.294		
	40	2	.9825		-35.731	0.404	1.115	22.53	1.706
7			.1537	2	5.022	0.048	0.227		
	20	20	.9593		-20.523	0.472	0.452	22.11	1.695
8			.2882	9	4.938	0.034	0.227		
	20	2	.9139		-26.436	0.482	0.691	22.41	1.694
9			.3730	4	15.242	0.074	0.671		

Table 19. Coefficients of Equation 19 obtained by regression for creep tests on silt (Series Cl, Pc = 40psi, Pns = 20psi) using combined groups of tests

Line	Tc °C	Ts °C	R^2 Se	d.f.	Intercept	$\frac{\Delta H}{k}$	α	β	$\beta\gamma$	Av w %	Av Gm
1		60									
	60	40	.8948		-18.348	2.278		0.503	0.679	22.14	1.682
		20	.4346	36	4.507	0.305		0.036	0.205		
		2									
2		40									
	40	20	.9258		-22.013	3.971		0.369	1.178	22.41	1.701
		2	.2874	16	5.309	0.398		0.038	0.261		
3	20	20	.9480		-15.086	1.822		0.473	0.487	22.22	1.694
		2	.2967	15	4.283	0.698		0.030	0.212		
4	60	60									
	40	40	.8780		-13.529	2.598	0.004	0.462	0.544	22.23	1.690
	20	20	.4239	74	3.111	0.253	0.003	0.024	0.143		
		2									
5	60	60									
	40	40	.8759		-14.257	2.460		0.464	0.547	22.23	1.690
	20	20	.4246	75	3.046	0.221		0.024	0.144		
		2									

Table 20. Regression results for creep tests on silt Series C2 (Tc = 40; Ts = 40, 2)

Line	Pc psi	Pns psi	R^2 Se	Residual d.f.	Intercept	$\frac{\Delta H}{k}$	β	$\beta\gamma$	Av w %	Av Gm
	60	60	.7769		-13.354	3.975	0.236	0.864	20.76	1.648
1			.7044	9	9.001	1.133	0.082	0.556		
	60	40	.8249		-39.464	3.387	0.153	2.102	21.27	1.660
2			.4332	5	11.571	0.815	0.122	0.656		
	60	20	.8979		-16.542	3.355	0.427	0.775	22.00	1.644
3			.4531	4	20.275	0.852	0.127	1.035		
	40	40	.9214		-29.474	4.970	0.170	1.879	21.71	1.650
4			.4005	6	13.653	0.934	0.093	0.788		
	40	20	.9786		-26.652	4.301	0.395	1.413	22.35	1.690
5			.1740	10	4.100	0.254	0.026	0.200		
	20	20	.9840		-17.572	2.682	0.647	0.714	22.83	1.602
6			.2406	6	6.738	0.542	0.068	0.383		

Table 21. Coefficients of Equation 19 obtained by regression for creep tests on silt using combined groups of tests

Series	Tc °C	Ts °C	Pc psi	Pns psi	R^2 Se	d.f.	Intercept	$\frac{\Delta H}{k}$	β	$\beta\rho$	$\beta\mu$	$\beta\gamma$	Av w %	Av Gm
C2	40	40	60	60	.7769		-13.354	3.975	0.236			0.864	20.76	1.684
		2			.7044	9	9.001	1.133	0.082			0.556		
C2	40	40	60	40	.8754		-32.120	4.152	0.206	0.016		1.866	21.50	1.650
		2	40		.4259	14	9.618	0.594	0.070	0.020		0.504		
C2			60											
	40	40	40	20	.8524		-35.088	3.752	0.348	0.038		1.827	22.41	1.644
		2	20		.5375	27	7.914	0.524	0.057	0.014		0.387		
C2			60	60	.8204		-22.867	3.555	0.308	0.043	0.003	1.294	21.81	1.659
	40	40	40	40	.5721	58	5.567	0.389	0.038	0.010	0.017	0.262		
		2	20	20										
C1	60	60	60	60										
&	40	40	40	40	.8354		-13.465	2.663	0.407	0.067	0.047	0.749	22.00	1.641
C2	20	20	20	20	.5240	123	3.150	0.222	0.023	0.007	0.010	0.143		
		2												

Table 22. Comparison of coefficients of Equation 18 obtained by regression for creep and direct shear tests on silt

Series	Tc °C	Ts °C	Pc psi	Pns psi	R^3 Se	d.f.	Intercept	$\frac{\Delta H}{\beta k}$	$\frac{a}{\beta}$	$\frac{1}{\beta}$	ρ	μ	γ	Av w %	Av Gm
							Expected coefficients from creep tests								
C1&	60	60	60	60											
C2	40	40	40	40		123	-33.084	6.543		2.457	0.165	0.115	1.840	22.00	1.641
	20	20	20	20											
		2													
							Experimental coefficients from direct shear tests								
D1	60	60	60	60											
	40	40	40	40	.9785		-30.346	4.832	.0005	1.240	0.123	0.359	1.769	21.90	1.659
	20	20	20	20	1.312	19	20.816	1.481	.027	0.860	0.028	0.043	0.914		
		2													
D1	60	60	60	60											
	40	40	40	40	.9785		-30.276	4.828		1.231	0.123	0.359	1.770	21.90	1.659
	20	20	20	20	1.279	20	19.977	1.413		.705	0.027	0.042	0.891		
		2													

This analysis would reveal any relationship between Tc and $\frac{\Delta H}{k}$, β or $\beta\gamma$. Except for the fact that $\frac{\Delta H}{k}$ in both series increases with Tc in the order Tc = 40, Tc = 60, Tc = 20 (for which there is no apparent reasonable explanation), there does not appear to be any consistent trend in the values of the coefficients.

All the tests in a given series were then treated as one group with Tc as an additional variable. The coefficients obtained from these two analyses, given for Series A1 in line 4 of Table 15 and for Series A2 in line 8 of Table 15, are considered to be the most meaningful for creep tests on clay and are shown plotted in Figures 11 to 16 inclusive. In both cases the average water content for the series (40.97% for Series A1 and 39.86% for Series A2) was used to obtain the regression lines. Thus, points having water contents greater than the average would tend to fall above their corresponding regression lines while those having water contents less than the average would tend to fall below. It should be noted that the coefficient of Tc, viz α, in both cases is based only on three levels of Tc.

Finally all creep tests on clay were combined and regression was carried out on all the variables including Pc. The results of this analysis are shown in the last line of Table 15. In this analysis the coefficient of Pc, viz $\beta\rho$, was based only on two levels of Pc. Further, Student t tests (calculations are illustrated in the Appendix) comparing the coefficients in the previous analyses (lines 4 and 8 in Table 15) indicated significant differences in the values of β but there were no significant differences in the values of $\frac{\Delta H}{k}$ or $\beta\gamma$. Thus this

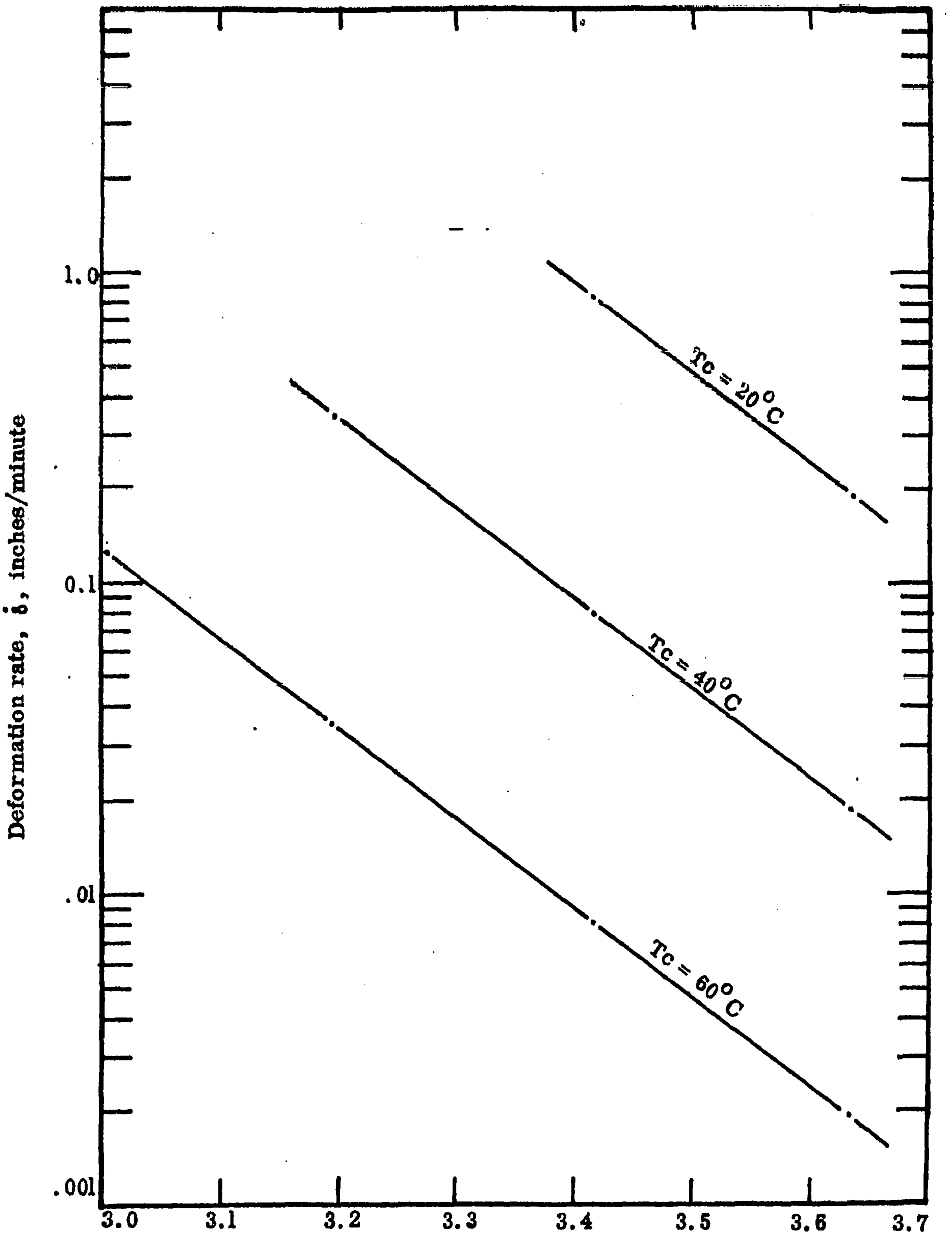

Figure 23. Deformation rate variation with shear temperature for clay with Pc = 45 psi and τ = 16.5 psi

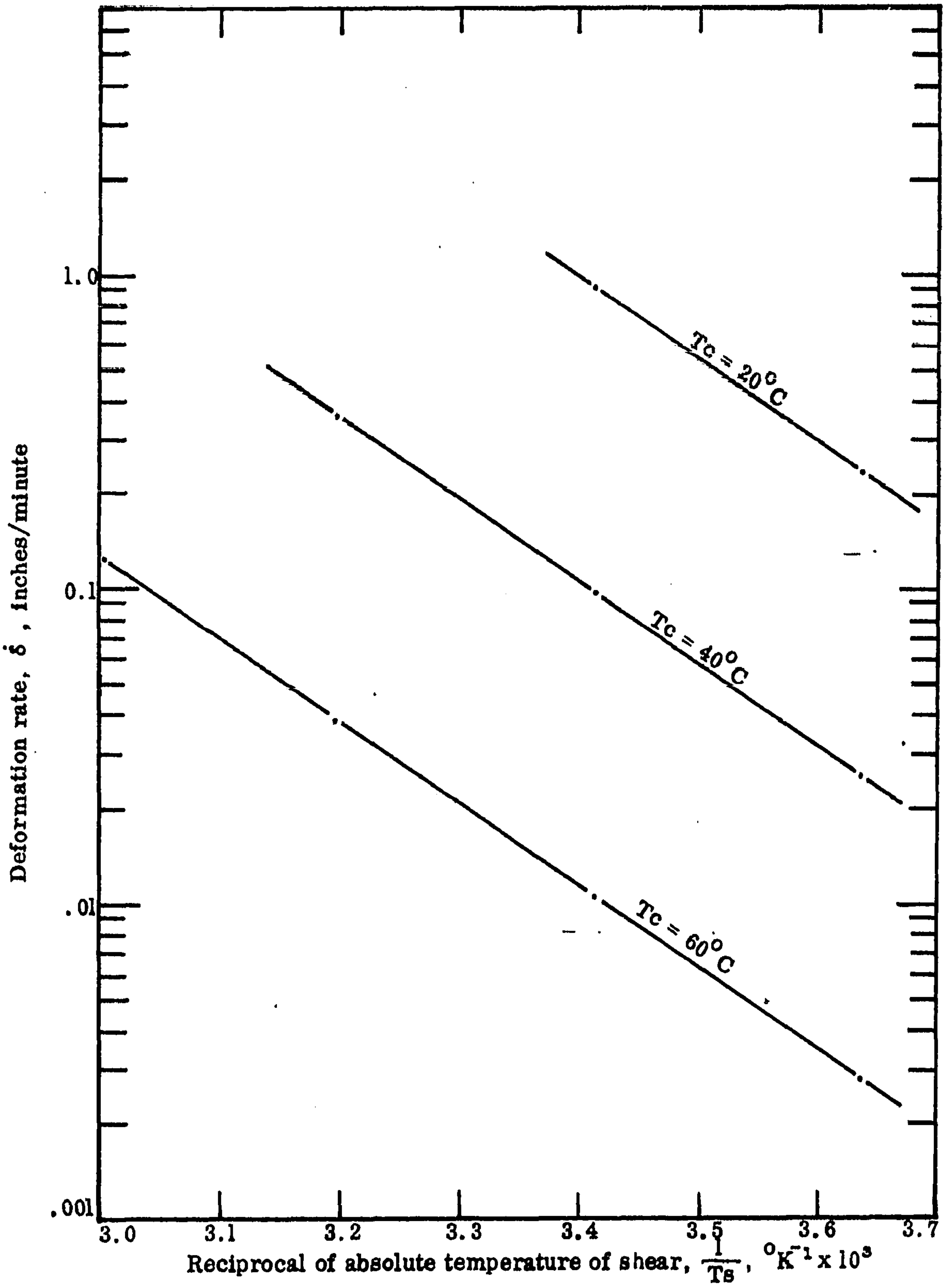

Figure 24. Deformation rate variation with shear temperature for clay with Pc = 60 psi and τ = 20 psi

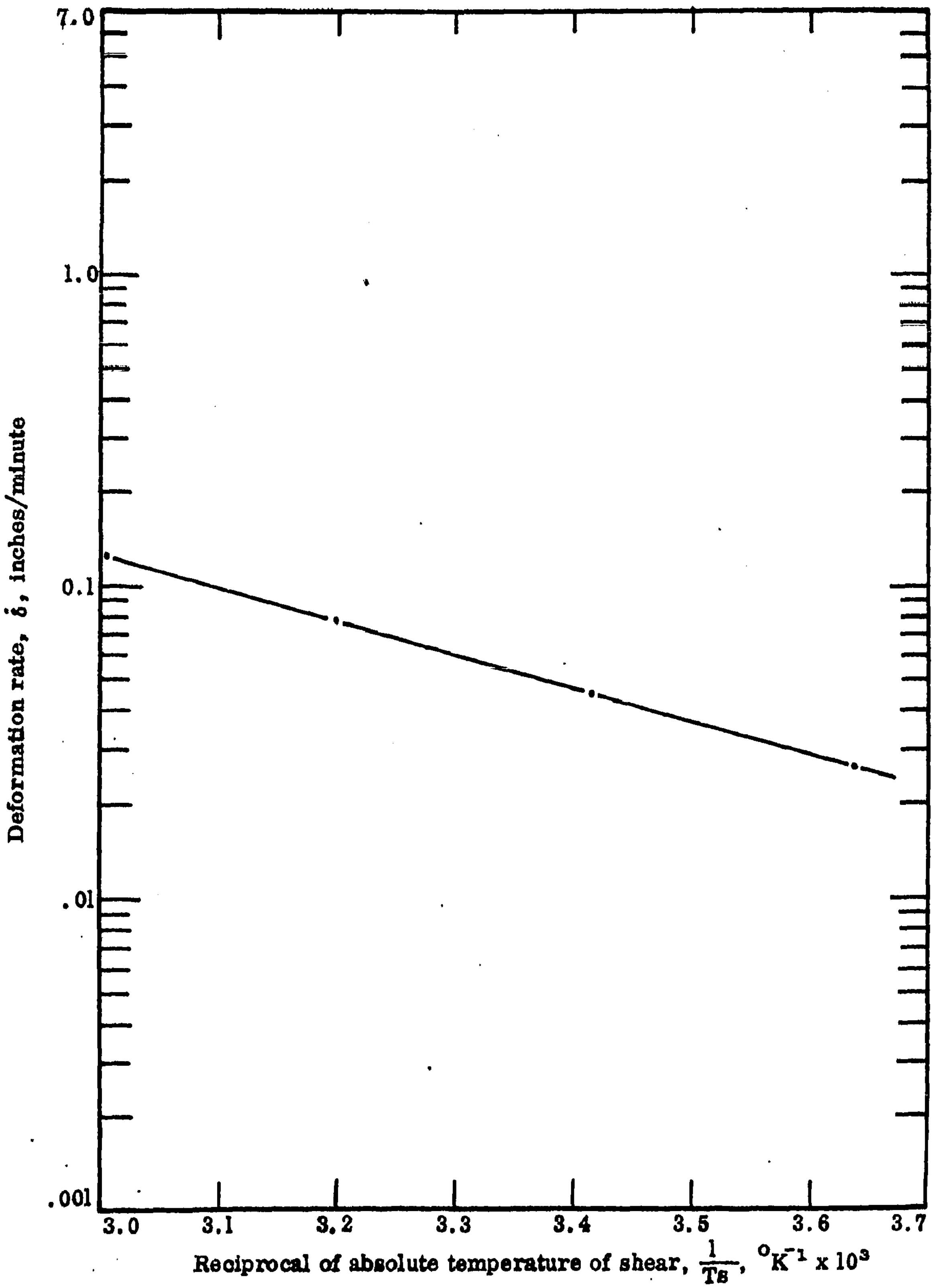

Figure 25. Deformation rate variation with shear temperature for silt with Pc = 40, Pns = 20, τ = 16 psi

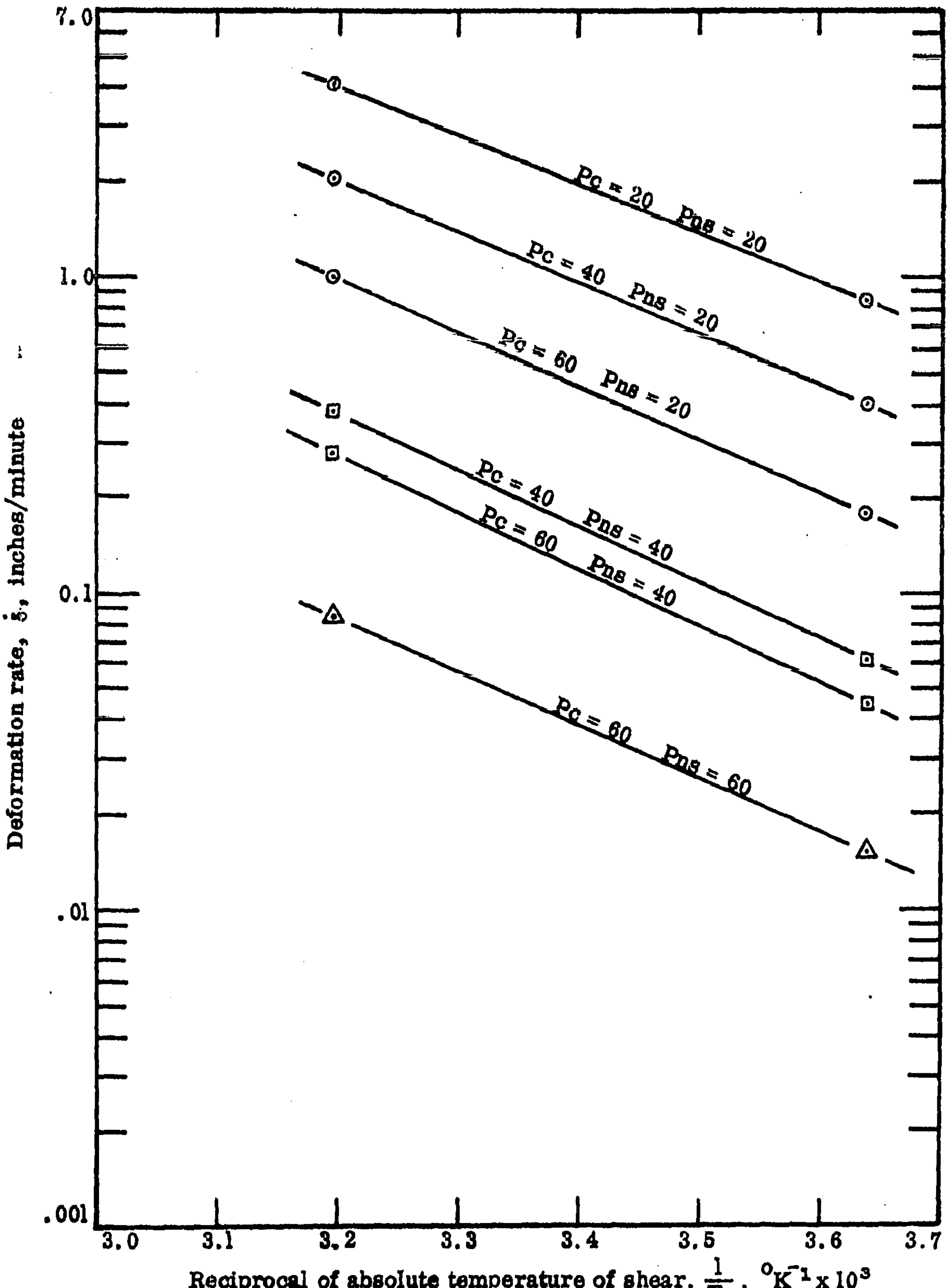

Figure 26. Deformation rate variation with shear temperature for silt with Tc = 40, τ = 24 psi

analysis, based on all the tests, may not be very significant. The implications of the differences in the values of β is discussed later.

Regression analyses were run on the direct shear test results for clay according to the five series of tests and various combinations of the series. The results of these analyses are given in Table 16 which essentially gives the same information as Tables 13, 14 and 15 with the exception that the coefficients in Table 16 are those applicable to Equation 18 rather than Equation 19. A comparison of the coefficients in Table 16 shows some significant differences particularly in the case of $\frac{\Delta H}{\beta k}$. These differences are probably related to the differences among the series due to soil preparation and procedures as previously described.

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 Equation 18 obtained from direct shear tests with those of Equation 19 obtained from creep tests. The comparison can be made by multiplying the direct shear coefficients or 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 17 which shows the converted coefficients for two series of creep tests which were run at different normal stresses, and the coefficients for these two series combined. The lower part of the table simply summarizes the coefficients given in Table

16 for the direct shear tests.

The results of regression analyses of the creep tests on silt are given in Tables 18, 19 and 20. For Series C1, in which the normal stresses were held constant, the tests were treated exactly the same as the creep tests on clay (Series A1 and A2). Table 18 gives the results of small groups of tests having the same values of all variables except τ and w. As for the clay, no significant trends in the values of β or $\beta\gamma$ with Ts were found and in fact the values of β were very constant throughout the temperature range tested.

The results for Series C1 of combinations based on Tc are given in the first three lines of Table 19 which show differences in the values of the coefficients. The values of $\frac{\Delta H}{k}$ and $\beta\gamma$ are high while the value of β is low for Tc = 40°C. This may be related to the fact that both w and Gm are high for this group and probably reflects some experimental variation.

Results obtained by considering all tests in the series are given in lines 4 and 5. The value obtained for the coefficient $\bar{a}$ (line 4) was not significant and the coefficients obtained by omitting Tc from the regression are given in line 5. These latter coefficients and the average water content for the series (w = 22.23%) were used to plot the regression lines shown in Figures 17, 18 and 19.

Creep tests on the silt (Series C2) were also performed at various levels of Pc and Pns with Tc = 40°C and Ts = 40°C and Ts = 2°C. These tests were first analyzed by considering groups having the same values of Pc and Pns and regressing $\ln\dot{\delta}$ on Ts, τ and w. The results of these analyses, given in Table 20, would permit variations of $\frac{\Delta H}{k}$, β and $\beta\gamma$ with

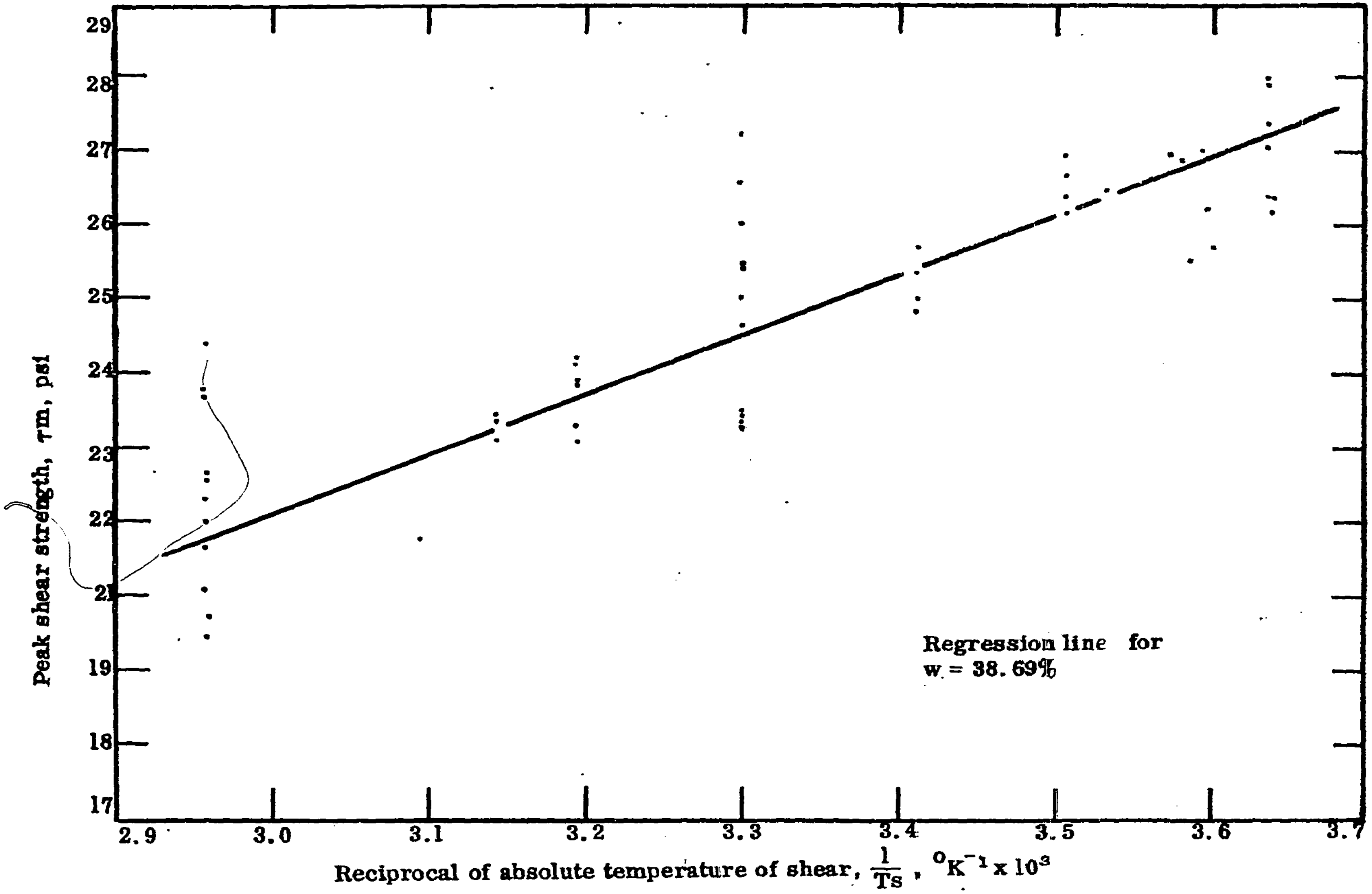

Figure 27. Variation in shear strength of clay with temperature of shear for Series B2

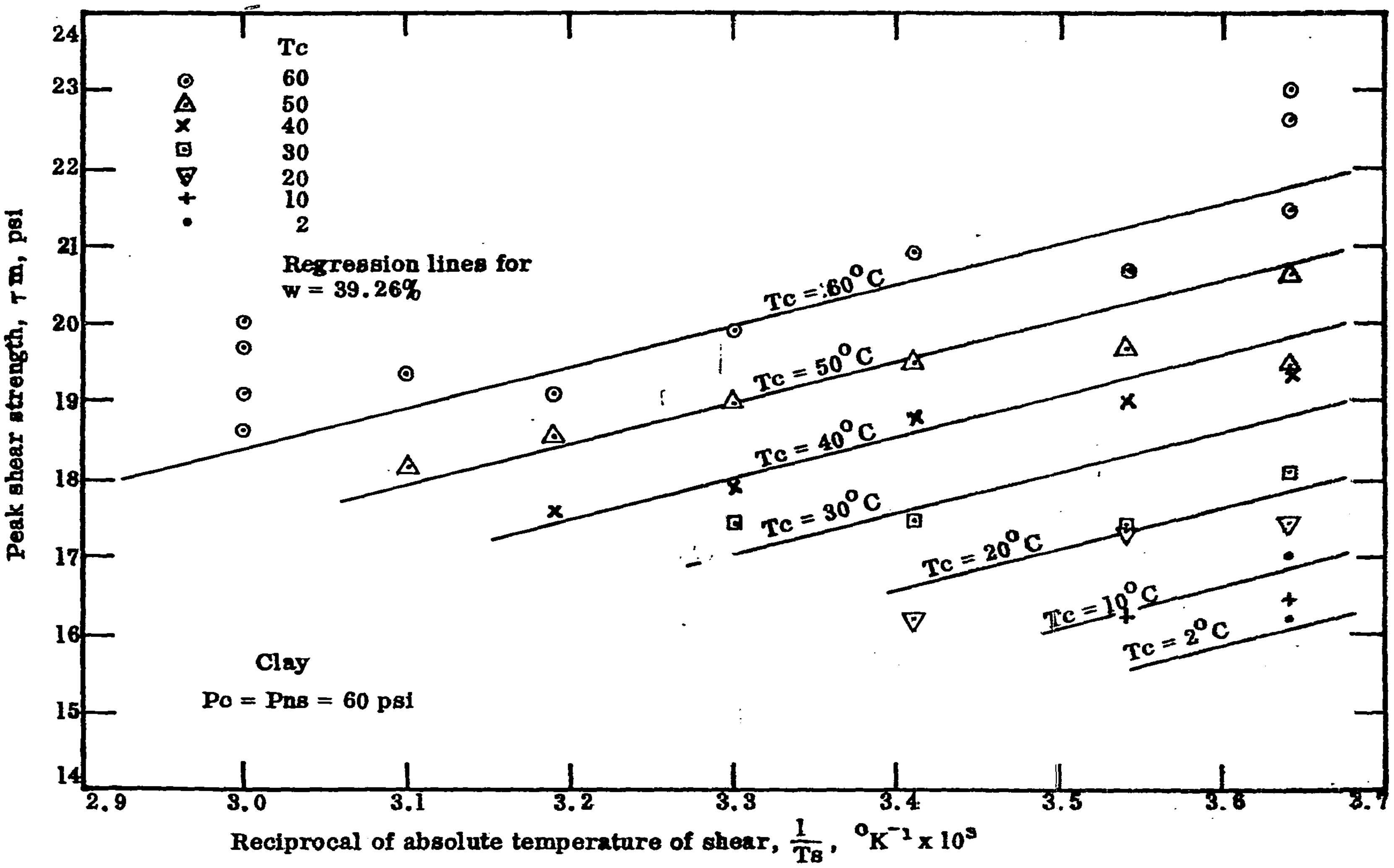

Figure 28. Variation in shear strength of clay with temperature of consolidation and shear for Series B4.

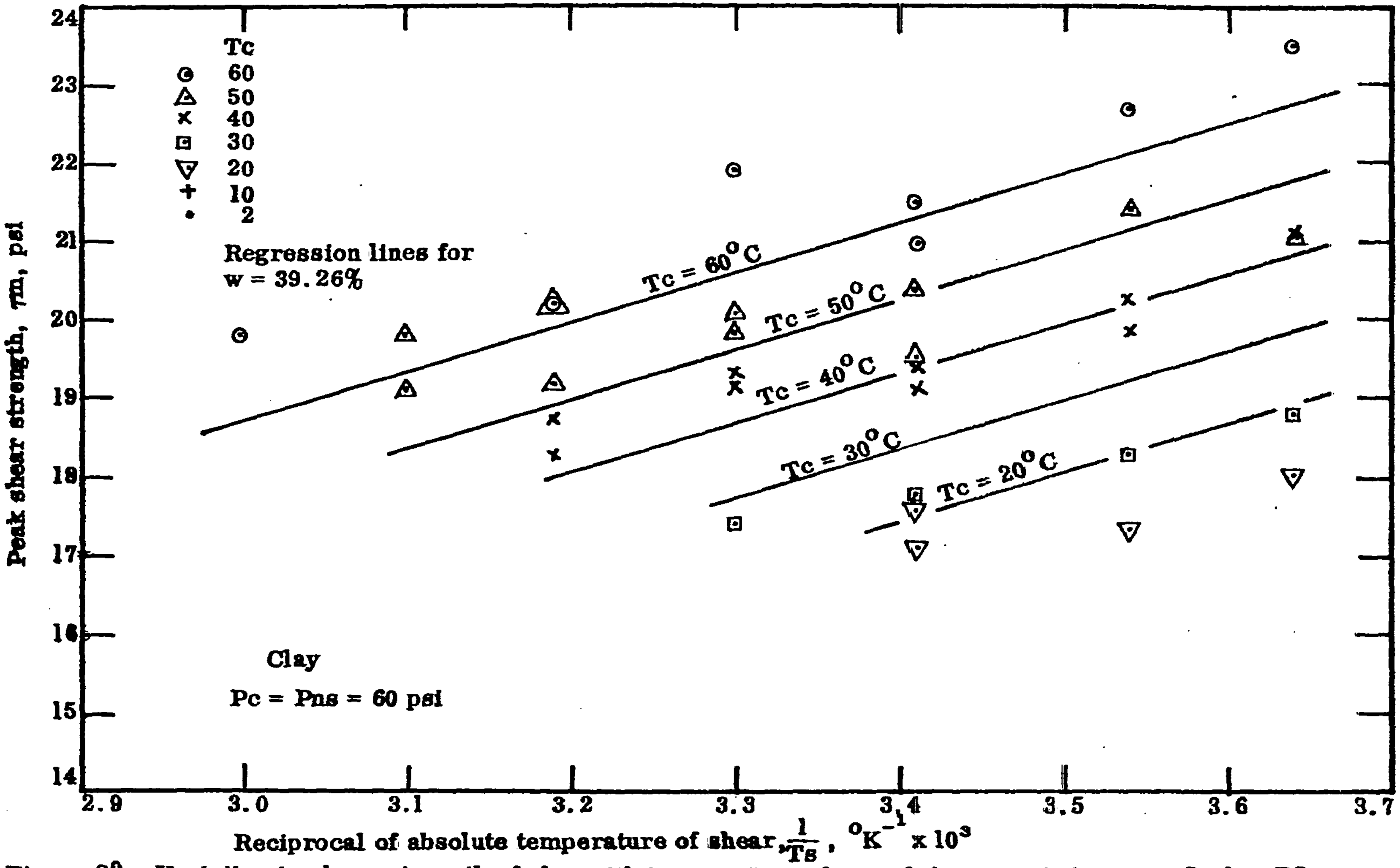

Figure 29. Variation in shear strength of clay with temperature of consolidation and shear for Series B5

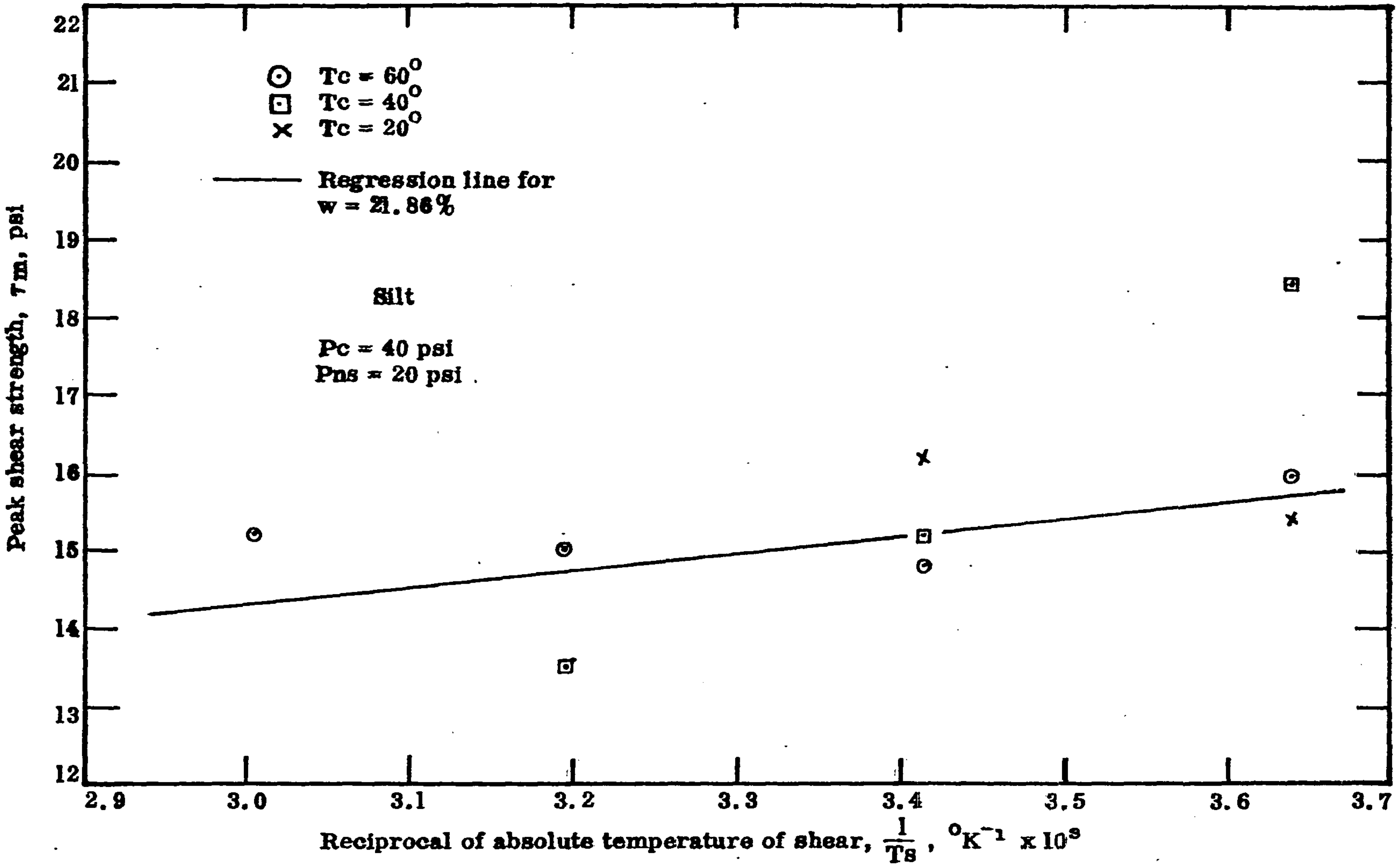

Figure 30. Variation in shear strength of silt with temperature of consolidation and shear

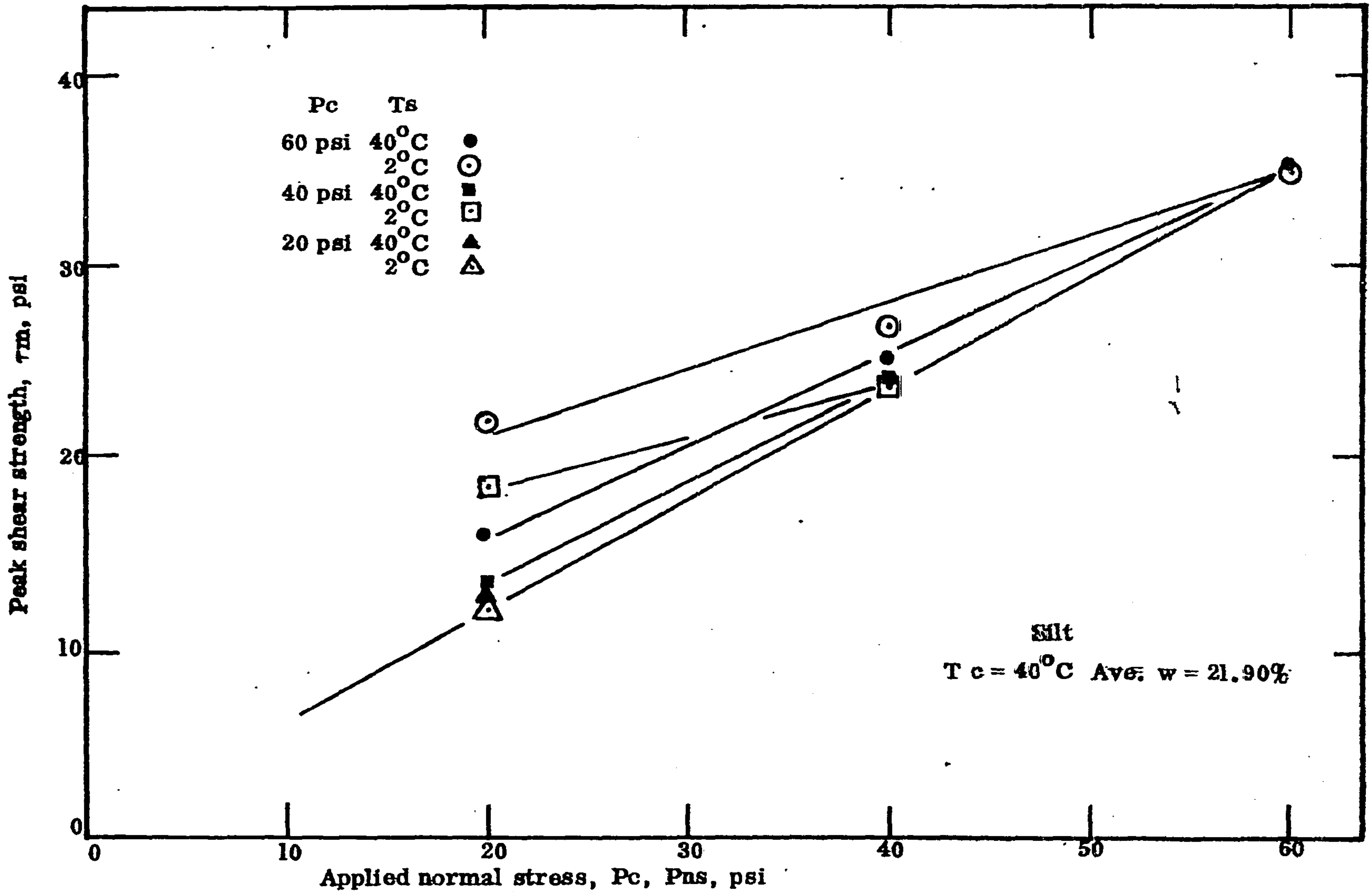

Figure 31. Strength envelopes for silt sheared at two temperatures

Pc and Pns to be distinguished. It can be seen that β increases significantly with decreasing values of Pns while $\frac{\Delta H}{k}$ and $\beta\gamma$ do not show any trend. The implication of the variation in β with Pns is discussed later.

These tests were next grouped according to common values of Pns and the results are given in Table 21 lines 1, 2 and 3. These coefficients and the average water contents for the respective groups were used to plot the regression lines in Figures 20, 21 and 22. The fourth line of Table 21 gives the results obtained by considering all tests in Series C2 and regressing ln $\dot{\delta}$ on all variables of Equation 19. Similar results are given in line 5 for all tests of Series C1 and C2 combined.

The 26 direct shear tests on silt were treated as one group and τ_m was regressed on all the other variables in Equation 18. The coefficient of Tc was found to be insignificant (as it was for the creep tests on silt) and a second regression was run with Tc omitted. The results of these analyses are given in Table 22 along with the expected coefficients from the creep tests which were obtained in the same way as for clay.

It may be noted here that water content (measured at the end of the test) and consolidation pressure are not entirely independent variables but, at the same time, they cannot be treated as a single variable. For given conditions prior to compaction and during consolidation and shear, the water content at the end of the test should be uniquely determined. However, as previously noted, differences in water content prior to compaction probably results in differences in compacted structure and thus variations in water content at the end of the tests even though all other conditions were held constant. Further, water content is partially dependent on temperatures of consolidation and shear due to the additional consolidation which takes place at higher temperature and the difference in density of water at different temperatures.

DISCUSSION AND CONCLUSIONS

Discussion

From the regression summaries for the creep tests it is apparent that the coefficient β, although somewhat variable for the individual smallest groups of tests, 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 creep tests performed on the clay at different normal stresses and in the values obtained at different normal stresses on silt. For both clay and silt the values of β were lower at the higher normal stresses. These results indicate a relationship between the coefficient β and the normal stress during creep. 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 and this is a problem which should be investigated by further research. 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 $\frac{1}{4}$ of that for clay. The value of β' for clay under

60 psi normal stress is calculated, as shown in the Appendix, to be 0.6711×10^{-18} cm^3 which would be equivalent to the volume of a cube 87.6Å on a side while, for the 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 in the order of size of a clay particle while for the silt the volume is considerably smaller, inferring 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 Equation 12 that β is independent of temperature. 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 Equation 15.

Values of the coefficient $\frac{H}{k}$ or $\frac{H}{\beta k}$ (and hence activation enthalpy) determined from the regression analyses appear to be of a reasonable magnitude compared with other reported values as shown in Table 1. Values of $\frac{H}{k}$ for the two series of creep tests on clay were shown to be not significantly different statistically. However, the higher values of Pc gave the lower value of $\frac{H}{k}$. This may be due to greater particle orientation under the higher normal stress or due to differences in initial water content. This latter suggestion is speculative since the average water content at the end of the tests was lower for the higher value of Pc as would be expected but the coefficient of water content, $\beta\gamma$, was higher for 45 psi than for 60 psi. From Tables 3 and 4 it can also be seen that the variation in w was greater in Series A1 than in Series A2 which may also have influenced the temperature coefficient. Mitchell et al. (1968) 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 values of $\frac{\Delta H}{\beta k}$ varied from 5.3 to 12.3 in the five series of direct shear tests on clay. Some of the reasons for these variations include differences in specimen preparation, consolidation time, water content and experimental variations. The soil of Series B1, having the highest value of activation energy, was stored as a ball and thus had time to develop bonds which probably were not entirely broken during placement in the rings and compaction. Further, the water content was the lowest and density highest for this series, which may also be related to the high value of activation energy. A difference between Series B4 and B5, which is perhaps significant, is in time of consolidation which for Series B5 was about twice as long as for Series B4. The activation energy found for Series B4 was lower than for Series B5 which tends to confirm an assumption stated earlier that time of consolidation would affect the results. It may be noted also that in test Series B1 to B3 inclusive, the consolidation phase was stopped as soon as 100% primary consolidation was reached whereas, in Series B4 and B5 and the creep tests, total time was maintained constant. This may be a reason for the higher activation energies of the former series compared to the latter series and the creep tests.

Activation enthalpies, ΔH, obtained from the coefficients of $\frac{1}{Ts}$ were found from creep tests on clay to be about 12 kilocalories per mole and from direct shear tests on clay they were found to range from 12.7 to 29.3 kilocalories per mole. The values for direct shear tests were obtained by using the average value of β from the creep tests. Creep tests on silt gave values from 4.9 to 7.1 kilocalories per mole while the direct shear tests on silt gave 3.9 kilocalories per mole using the average β value from the creep tests. These values are somewhat lower than the experimental values of Mitchell et al. (1968) which ranged from 31 to 40 kilocalories per mole. A large part of this difference is probably related to experimental procedure. Mitchell et al. rapidly increased the temperature during the course of a creep test and measured the increased rate of deformation which occurred. They reported that cell pressure, deviator stress and back pressure were kept constant during the temperature change. Unless pore water pressure decrease (consolidation rate) was very rapid for the soil tested (this was not reported), the excess pore water pressures built up during the temperature increase would not be entirely dissipated in spite of the constant back pressure and drained conditions. This would result in a higher rate of deformation due to the decreased effective stress. Had the pore water pressures remained constant as suggested by the authors, then, in view of the effect of temperature on consolidation found by Campanella and Mitchell (1968) and others as well as in this study, the deformation rate would be expected to decrease due to the additional consolidation at the higher temperature. In this study, this latter effect may have tended to reduce the values somewhat in view of the fact that the specimen thickness was small and drainage could occur quite rapidly. Thus, at the higher shear temperatures, pore water pressures built up due to deformation would dissipate more rapidly than at lower temperatures. However, the duration of tests on the clay was very short compared to the time of even 50% consolidation and the effect should have been negligible. Although time for consolidation of silt was short, the majority of tests were performed on specimens having an overconsolidation ratio of two and, in this case, developed pore pressures were negative. Therefore, if differences in drainage due to temperature were to have an effect, it would be to give activation energies which were too large for the silt.

The lower activation enthalpies found for the silt give 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 suggesting that the bonds are not strongly influenced by the surface energy of the particles.

It should be noted that the activation energy was determined from the regression results rather than directly from the experimental data.

A plot of deformation rate versus 1/T could be obtained from the deformation rate-stress plots by ignoring water content variation. However, water content decreased with increasing temperature of shear and thus gave a curved line which suggested that activation energy was a function of temperature. Although this may be possible as Dorn (1954) found for aluminum, the definition of activation energy tends to preclude this possibility. This illustrates an advantage and a disadvantage of using regression analyses. A random variable, such as water content, can be taken into account and the data made to better fit the model but at the same time, other functional or non-linear relationships may be overlooked. To avoid this error, regression analyses were carried out on smaller groups of data to determine whether there were any such relationships. As previously discussed, no definite trends were in evidence but this does not eliminate the possibility of activation energy varying with temperature. To further investigate this possibility, tests should be run at a wide range of water contents to determine independently the effect of this variable and its proper functional relationship.

Under a normal stress, 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. (1966) 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 (1967) and others and are illustrated for clay and silt in Figure 33 which shows the decreased time of consolidation at 60°C as compared to 2°C. These curves were obtained from standard consolidation tests performed in the shear rings at the two temperatures. Consolidation at a higher temperature, as shown in Figure 34, gives a reduced void ratio at any load although the compressibility remains essentially the same. Thus, for a given normal load, a higher temperature of consolidation should give a reduced rate of deformation as illustrated for clay in Figures 23 and 24 or an increased shear strength as illustrated for clay in Figures 28 and 29. Although a reduced void ratio at the higher consolidation temperature is also indicated for silt in Figure 34, no significant effect was measured in either direct shear or creep tests. This behavior further supports the suggestion of a weak influence of particle surface energy in the case of silt.

The coefficient $\beta\gamma$, related to the effect of water content, varied quite erratically particularly among the smallest groups of tests and even changed sign in a few cases. This is not surprising in view of the small and random variation in water content, 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 the creep tests on silt was 0.75 giving an average value of γ of 1.8 which was the value obtained as an average from the 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 which is well recognized in the field of Soil Mechanics and which may be explained on the basis of the large differences in specific surface area between these materials.

As previously discussed, the coefficient β may be related to the normal stress during shear. However this variation was of insufficient magnitude to account for the increased strength with increased normal stress. In the linear equation proposed, additional relationships with Pc and Pns were obtained. These relationships for silt are shown in Figure 31 which gives the results of direct shear tests performed at several levels of normal stresses. This shows typical strength envelopes which may be formulated in terms of the Krey-Tiedemann criterion (Equation 3). The coefficients ρ and μ obtained from regression based on Equation 18 are essentially tan ϕ_c and tan ϕ_r respectively of Equation 3. It is interesting to note that these two terms alone almost account for the shear strength while the remaining five terms of Equation 18 tend to cancel each other. In Equation 3, the effect of water content is contained in tan ϕ_c while the effects of temperature and deformation rate are contained in $\tan \phi_r$. The effect of shear temperature on $\tan \phi_r$, is well illustrated in Figure 31 and indicates that the temperature of shear primarily affects the cohesion component of shear strength.

In view of the foregoing discussion of the various coefficients, it becomes evident that the model equations (Equations 18 and 19) may require modification as further evidence becomes available. A complete and correct model equation should account for all the variables involved in the behavior being studied. The intercept, $\ln M$, of Equation 19 ($\frac{1}{\beta}\ln M$, of Equation 18) contains the combined effects of those variables not specifically considered, as well as a dimensional length term which accounts for the fact that deformation rate is used rather than strain rate, and a frequency factor arising from the Arrhenius equation. A part of the intercept value may also be accounted for by the apparent dependencies of β on Pns and of water content on Pc as well as the probable nonlinearity of the relationships between water content and shear strength. Among the variables not specifically considered are effects of dilatency and electrolyte concentration on structure. Although the mechanism of deformation has been considered only from the standpoint of bonds at interparticle contacts, there is no doubt that dilatency plays a major role in soil resistance to deformation, particularly in more granular soils such as silts and sands. Dilatency may be considered as "macro-dilatency" which is caused by the lifting of one particle over another and "micro-dilatency" which is caused by irregularities on the surface of the particles (Rosenqvist, 1959). These components of dilatency are related to the normal stresses and are not readily separated as unique components of strength or energy terms. Dilatency may be considered as a part of the activation free energy but, as it is probably not temperature dependent, it would not be determined as a part of ΔH but rather would be related to the entropy term and thus normal stresses, in keeping with the method of derivation of Equations 18 and 19. However, regardless of the possible modifications to the model equations with further investigations, the equations have been shown to fit the experimental results quite well and future investigations may show that some of the differences in coefficients found here were simply due to experimental variations.

An interesting and important fact discovered in this study and apparently not previously reported, is the relationship between creep response and stress-deformation relationships of direct shear tests. From Figures 9 and 10 it can be seen that, if the intersections between a line of constant deformation rate and the curves for each shear stress are plotted against the corresponding deformations, a stress-deformation curve results which is of the same shape as the curves obtained from the direct shear tests. This has been done for tests on each of silt and clay using Figures 9 and 10 and the curves are compared in Figure 32 with stress-deformation curves from direct shear tests performed at the same temperatures and normal loads. The agreement between the curves is quite good and the peak shear stress obtained from the creep test plots was within 10% of the peak shear strength found from the direct shear tests run at the same rate of deformation.

The above relationship and the general agreement between the values

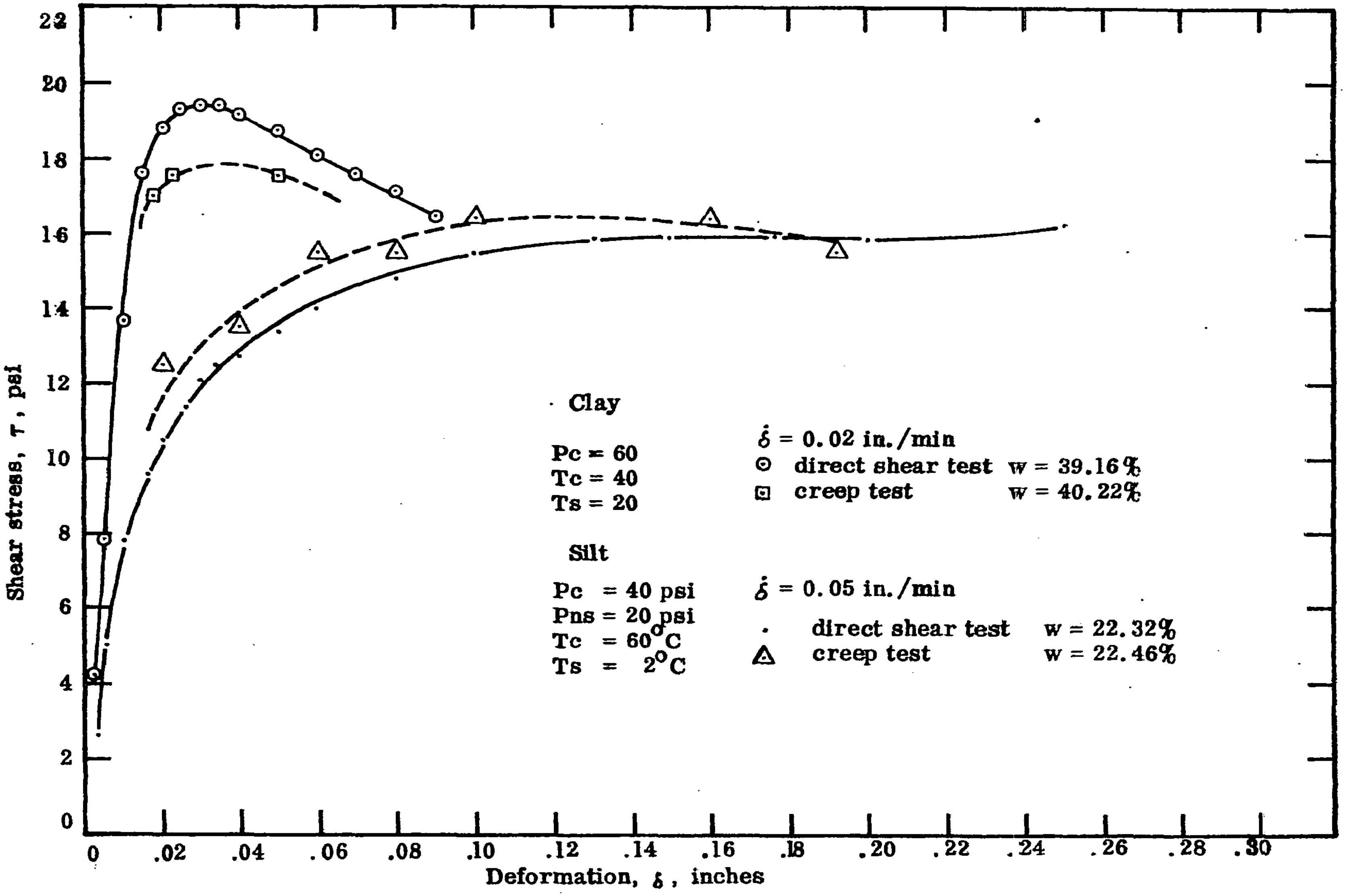

Figure 32. Sample stress-deformation curves for clay and silt from direct shear and creep tests

of the coefficients obtained from creep and direct shear tests tend to confirm the hypothesis that there is one consistent deformation mechanism for soils which is independent of method of load application. These confirmations were obtained as a result of performing the creep tests at stresses which were in 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 on the deformation rate curves. In most reported creep studies (e.g. Tan, 1959; Singh and Mitchell, 1968), the applied stress is generally low enough that all tests go to terminal creep (i.e. $\dot{\delta} \rightarrow 0$) and, in this case, a criterion for comparing with strength behavior becomes difficult. This would account for many of the rheological models proposed to explain soil behavior. If the soil structure is related to deformation rather than time as proposed herein, then the behavior of terminal creep curves should be compared only in the initial portion where a common value of deformation can be obtained. It is interesting to note, however, that for the silt, the line of minimum deformation rates plotted on the creep curves in Figure 8 approached a condition of constant time as proposed by Singh and Mitchell (1968) as a criterion for comparing behavior. The criterion may be influenced by the test methods and further investigation of this point should be carried out.

One of the main weaknesses of the experimental program was the variability of the soil specimens. This variability was probably due to variations in water content within the batches of stored soil and would result in variable structure in the compacted specimens. This problem may be overcome in future studies by mixing and de-airing the soil in a slurry consistency and then slowly consolidating it to a consistency suitable for testing. This procedure would take a great deal more time than that used in this study but would hopefully give more consistent results resulting in fewer required tests. Further, if the model proposed herein is used as a basis for further study, a few carefully selected and precise tests would establish the suggested relationships.

Suggestions for further research

Based on the observations of this study, the following suggestions for further work have been listed.

1) The criterion, used to extrapolate minimums of deformation rate-deformation curves to curves showing no minimum, needs confirmation based on relationships of structure to deformation for different test methods.

2) The correct functional relationship of water content to strength and creep behavior and its dependence on normal stresses needs further investigation.

3) The relation of the coefficient β to normal stress needs clarification.

4) Consolidation effects on structure and the resulting effects on strength is not clear and should be studied with the goal of explaining the decreasing rate of deformation which occurs at low stresses.

5) Measurement and control of pore water pressures would be desirable in any tests of soil behavior. The pore pressure measuring needle developed for this study would be satisfactory providing the soil was saturated and drainage could be controlled. This is not possible with

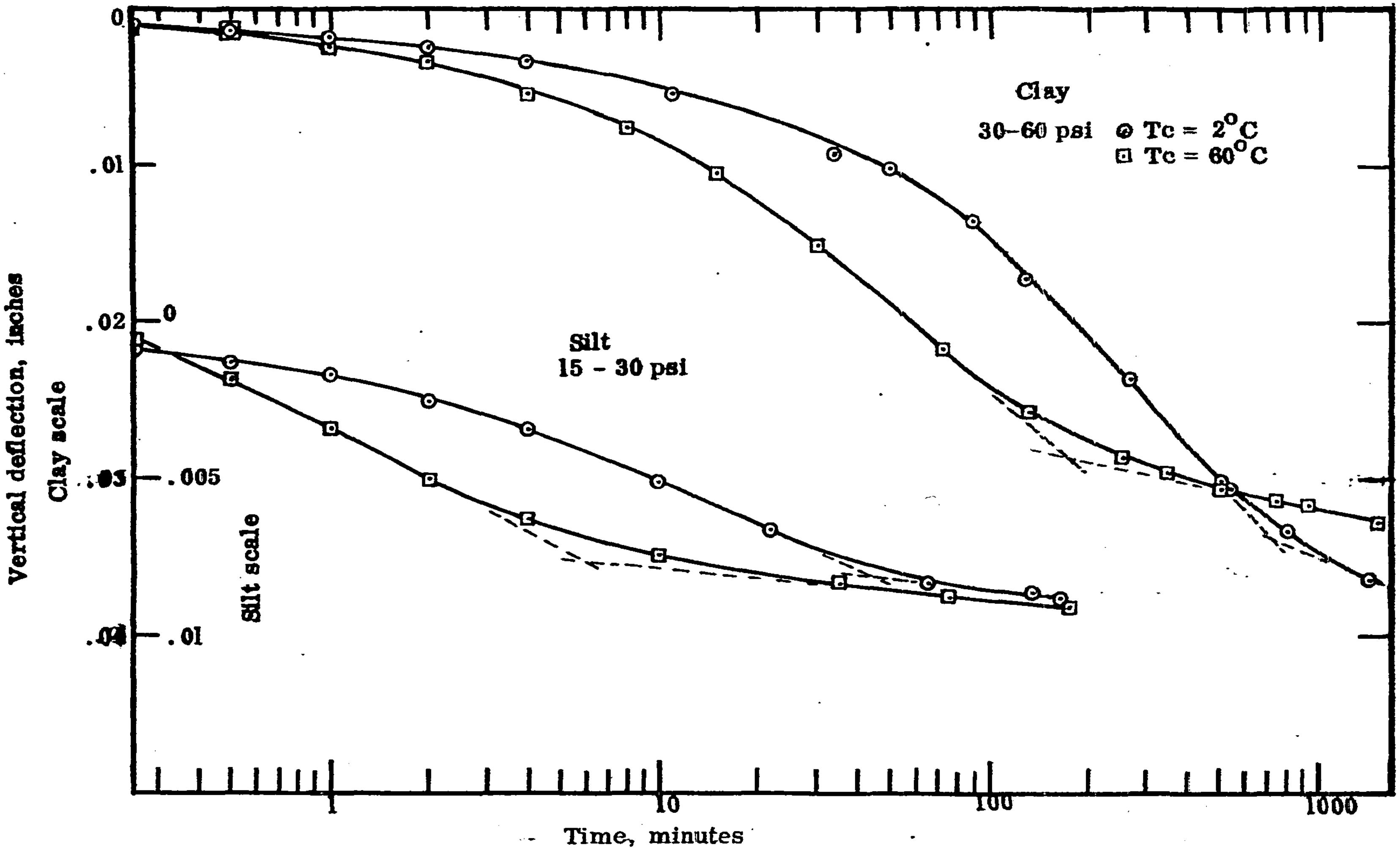

Figure 33. Effect of temperature on consolidation rate of clay and silt

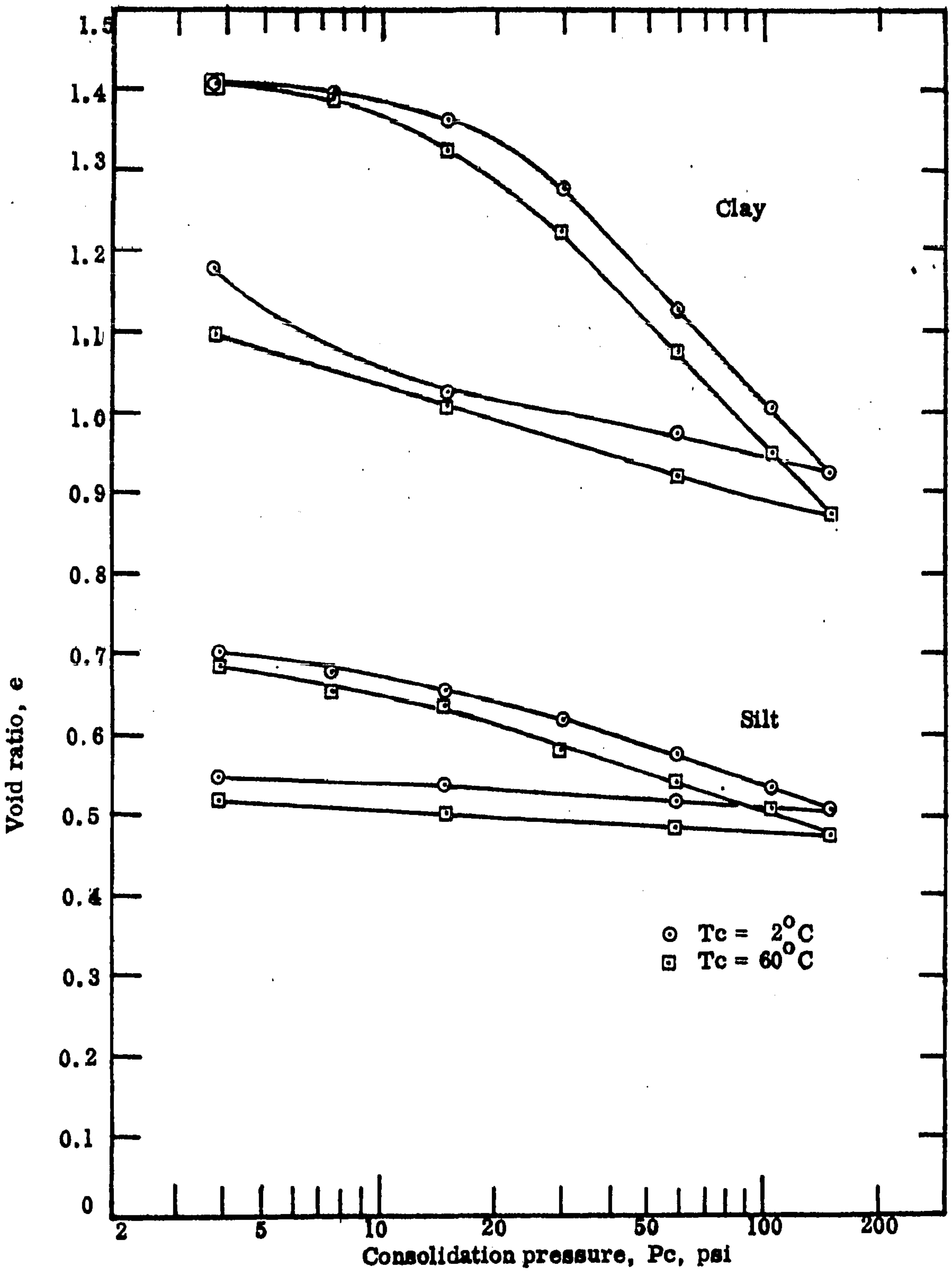

Figure 34. Effect of temperature on compressibility of clay and silt

most direct shear devices. It is believed that a simple shear device could be designed to give the required drainage control and at the same time would eliminate many of the problems inherent in direct shear testing such as the indeterminancy of principal stress directions and the narrow shear zone which did not permit measurements over large deformations.

Conclusions

A simple model of cohesive soil is proposed. This model is based on bonds formed at interparticle contacts due to a structure in the adsorbed water layers adjacent to clay particles. The high energy of adsorption of water to mineral surfaces precludes any mineral-to-mineral contact but, when two particles are in close proximity, their adsorbed water layers intermix and the water forms a continuous structure in this zone causing the particles to be bonded. Stresses applied to the system are transferred through these bonds and deformation of the system occurs by distortion or breaking of the bonds. The resistance of the water structure to deformation or rupture imposes an energy barrier to the deformation of the soil mass. The energy barrier may be surmounted by bonds having sufficient thermal or shear energy.

Based on this bond model, an equation was developed starting with the Arrhenius equation of chemical kinetics and including those factors known to affect soil behavior. This equation was shown to be basically valid for both direct shear tests and creep tests on a highly plastic clay and a clayey silt. In direct shear tests, the soil is subjected to a given rate of deformation and the resulting shear resistance reflects the energy required to overcome the bonds between the particles. In creep tests, the soil is subjected to a given shear stress and the resulting rate of deformation reflects the rate at which the bonds are broken. In both cases the breaking of bonds involves the flow unit acquiring sufficient energy to surmount the energy barrier. The kinetic energy of a flow unit is related to its temperature and thus the rate at which the energy barrier is crossed is temperature dependent. An increase in temperature, therefore, gives an increased rate of deformation for a given level of stress in a creep test and a decreased shear resistance at a given rate of deformation in a shear test. The proposed equation, written in terms of shear strength and deformation rate respectively takes the forms:

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

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

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

1) Creep tests yield a linear relationship between the logarithm of deformation rate and shear stress which is independent of temperature. The proportionality coefficient, which may be a function of normal stress during shear, can be used to obtain a measure of the volume of the flow unit. This volume was found to be of the order of the particle size for the clay but considerably smaller than particle size for the silt.

2) Activation enthalpies, obtained from the coefficients of the reciprocal of the absolute temperature of shear were found to be approximately the same for creep and direct shear tests. From creep tests on clay, ΔH was found to be about 12 kilocalories per mole while from direct shear tests it ranged from about 13 to 29 kilocalories per mole. Creep tests on the silt gave values from about 5 to 7 kilocalories per mole while direct shear tests gave about 4 kilocalories per mole. The lower values for the silt indicate less influence of particle surface energy and a bond strength only slightly greater than that of water.

3) An increase in temperature of consolidation was found to increase the strength of clay but had little effect on the strength of silt. The lack of effect in the case of silt indicates a relatively thin adsorbed water layer and thus little effect due to surface energy.

4) The coefficients relating deformation rate and shear strength to water content were also found to agree fairly well between creep and direct shear tests. 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 coefficients were rather poorly defined and the relationship was assumed to be linear.

5) The deformation rate-deformation curves from creep tests were shown to be related to stress-deformation curves from direct shear tests.

6) The agreement between the coefficients obtained from the two test procedures and the observed relationship between the curves of the two tests confirm the hypothesis of a single deformation mechanism which

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 which reasonably characterizes the deformation and strength behavior of the soils. However, modifications of the model equation may be required as further investigation may reveal interdependencies of some of the variables and specifically to account for the functional relationship of water content. The test results were analyzed by means of multiple regression which aided in interpreting the variability due to differences in water content. Further studies, based on the model proposed herein, would appear to be desirable since the model explains the strength and deformation behavior of soils in terms of fundamental parameters which could be used as a rational basis for studies of the effects of mineralogical, physico-chemical and structural properties.

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APPENDIX

Calculations for Student t test

To make valid t tests it is necessary to assume that the errors are normally distributed. This assumption would seem valid for the results reported herein.

The following data are taken from Table 15.

Series	Pc	Res. d.f.	$\frac{\Delta H}{k}$	Se (for $\frac{\Delta H}{k}$)	Se
A1 (line 4)	45	37	6.558	0.689	.6064
A2 (line 8)	60	45	5.949	0.540	.5386

It is desired to determine whether there is any significant difference in the two values of $\frac{\Delta H}{k}$. This is done by first pooling the standard errors to get a pooled estimator of variance according to the formula:

$$Sp^2 = (C_{i1} + C_{i2}) \frac{(Q_1^2 + Q_2^2)}{df_i + df_2}$$

where C_i is the diagonal matrix element and may be found from $Ci = (Sei/Se)^2$ and Q^2 is the residual sum of squares which may be found from $Q^2 = df \times Se^2$.

For the above data this gives:

$$Sp^2 = (1.0045 + 1.2900) \frac{(13.6064 + 13.0532)}{82} = 0.7460 .$$

Then t is found from:

$$t = \frac{(\frac{\Delta H}{k})_1 - (\frac{\Delta H}{k})_2}{Sp}$$

This may be compared with the appropriate value from t tables at the desired level of significance. For the example this is:

$$t = \frac{6.559 - 5.949}{0.7460} = .7061$$

The value of t with 82 degrees of freedom at the 90% confidence level is found from tables (e.g. Snedecor, 1966 p. 46) to be 1.66. Thus, since the calculated value of t is less than this, it can be stated that the value of $(\frac{\Delta H}{k})_1$ is not significantly greater than $(\frac{\Delta H}{k})_2$. Other pairs of coefficients can be similarly compared.

Calculations of flow unit volumes

Example calculation for clay tested at 60 psi.

Average β from Table 15 = 1.118 $in.^2$/psi

$k = 1.3805 \times 10^{-16}$ dyne cm. $^{o}k^{-1}$ $T = 300^{o}k$

$$\frac{1.118 \times 1.3805 \times 10^{-16} \times 300}{6.895 \times 10^{4}} = 0.6711 \times 10^{-18}\ cm^3$$

$(0.6711 \times 10^{-18})^{1/3} = 0.8755 \times 10^{-6}$ cm or 87.55Å

Example calculation for silt tested at 60 psi.

Average β from Table 20 = 0.236 $in.^2$/lb

$$\frac{0.236 \times 1.3805 \times 10^{-16} \times 300}{6.895 \times 10^{4}} = 0.142 \times 10^{-18}\ cm^3$$

$(0.1417 \times 10^{-18})^{1/3} = 0.521 \times 10^{-6}$ cm or 52.1Å.