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Bulletin 245

Water Tensions; Swelling Mechanisms; Strength of Compacted Soil

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Water Tensions; Swelling Mechanisms; Strength of Compacted Soil

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Contents

A Device for Measuring Tensions in Water

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The measurement of negative pore water pressures in soil systems is important in both research and design studies on precompressed saturated clays and partially saturated compacted clays. To date, no equipment that can measure water tensions below absolute zero pressure under conditions of changing external stress has been available. This paper describes such equipment.

In the device described here, the sensing element contacting the soil is a fine-grained ceramic stone, which is connected by a closed water system to a flexible steel membrane. Tension is controlled by externally adjusting the membrane deflection. The absence of a free water surface prevents cavitation.

Calibration data demonstrate that the pore pilot can indeed measure water tensions below absolute zero pressure.

● THIS PAPER describes a tensiometer capable of measuring water tensions of large magnitudes. The absence of a free water surface within the system prevents cavitation of the water at pressures below absolute zero pressure. The device was calibrated by application of pressures and vacuums of known magnitudes. Computed osmotic pressure differences were used for calibration in the high tension range. The calibration results demonstrate that the device can indeed accurately measure water tensions as large as 90 psi (all pressures referred to in this paper are gage pressures).

This device was developed at the Soil Engineering Division, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, as the first phase of a doctoral research program on the properties of compacted clays. The research program is under the supervision of Dr. T. William Lambe, Head of the Soil Engineering Division.

THE IMPORTANCE OF WATER TENSION MEASUREMENTS IN SOILS

The importance of "effective" or "intergranular" stresses in the determination of the properties of soils has long been recognized in soil mechanics. The effective stress concept was first formulated by Terzaghi (12). The validity of this concept for saturated soil systems has been demonstrated experimentally (1, 2).

In saturated clays appreciable negative pore water pressures can be developed: During the undrained shear of over-consolidated clays; upon removal of overburden and desiccation; and during sampling. Hansen and Gibson (7) have presented a detailed theoretical treatment on the effects upon shear strength caused by the change of stress a clay undergoes during sampling; however, lack of instrumentation for the measurement of the induced negative pore water pressures allowed only qualitative conclusions.

The measurement of water tensions is of prime importance in the study of partially saturated, compacted clays. It is felt by many that the strength of compacted clays is due primarily to the negative pore water pressures developed during compaction. Hilf (8) measured a water tension of 13.8 psi in a clayey silt compacted at optimum water content by Standard Proctor compaction. It cannot be assumed that water tensions in compacted soils are equivalent to negative pore water pressures as used in the effective stress concept, as this concept applies only to saturated systems. The degree of discrepancy will depend upon many factors, the two main factors being the degree of saturation and the compressibility of the soil. Much experimental work remains to be done in this area.

REQUIREMENTS FOR WATER TENSION MEASUREMENTS IN SOILS

Needed for research on compacted clays is a device that can measure moderately large water tensions in both saturated and partially saturated soils systems under conditions of changing external stress. The equipment should require no moisture transfer between soil and device and should have a negligible time lag. Such a device has been developed and is described herein.

Table 1 summarizes available methods for determining water tensions in soils together with the tension range for which the methods are suited (4, 5, 11).

As normally performed, all of the above techniques involve moisture transfer with a concomitant time lag to equilibrium. The tensiometer and pressure membrane methods can be modified so that they are "no flow" devices. Hilf's device (8) for measuring pore pressures in compacted clays is basically a combined tensiometer-pressure membrane system modified to operate as a "no flow" device. However, his device is still limited to a tension of 0.8 atm, and the air pressure applied in the pressure membrane method precludes other possible changes in external stress.

DESCRIPTION OF DEVICE

Basic Principles

The device described below is basically a tensiometer so modified as to operate under "no flow" conditions, with the tension range greatly extended beyond 0.8 atm. Tensions below absolute zero pressure are possible because there are no free vapor-liquid surfaces present in the system. A schematic of the device is shown in Figure 1. It measures the pressure or tension necessary to prevent the flow of water out of or into the soil mass. The sensing element which is in contact with the soil consists of a porous ceramic cylinder and is connected by a continuous closed water system to a chamber above a steel membrane. The membrane is initially prestressed from below by air pressure. Water movement toward or away from the ceramic stone is indicated by movement of a mercury null slug in a capillary bore section of lucite tube between the stone and the steel membrane. The change in air pressure necessary to prevent water movement out of or into the soil can be interpreted as a tension or a pressure in the pore water.

TABLE 1

METHODS FOR DETERMINING SOIL WATER TENSIONS

After Aitcheson (1)

Method	Tension Range, Atmospheres
Tensiometer	0 - 0.8
Tension Plate	0 - 0.8
Pressure Membrane	0.1 - 20
Gypsum Block	0.5 - 50
Sorption (Vacuum Desiccation)	10 - 10.000

It can be shown by thermodynamics that a liquid, when sealed in a closed system without a free surface, may be tensioned well below its vapor pressure without boiling. The fact that water can withstand considerable tension has also been demonstrated experimentally. Green (6) has summarized the work of Berthelot (1850), Dixon (1909), Meyer (1911), Vincent and Simmonds (1943), and Temperley and Chambers (1946). In these experiments, the researchers sealed a section of capillary tube in which a small bubble of air was purposely entrapped in the water-filled tube. The tube was gently heated until the bubble disappeared and then was allowed to cool. In all cases the temperature at which the bubble reappeared was considerably below the disappearance temperature. Computation of the contractions during cooling led to the value of induced water tension. There was general agreement that water containing air in solutiong could withstand a tension of 40 atm in a closed vessel. In 1951 Green (6) performed a modified Berthelot experiment. He vacuum-distilled his water, completely de-airing it. A bubble of water vapor was entrapped instead of an air bubble. Water tensions were obtained upon cooling of 190 atm. These experiments clearly demonstrate that in the absence of a free surface water can withstand considerable tension without cavitation.

Design Details

The device was designed for a maximum range of approximately 10 atm of tension. A schematic of the device is shown in Figure 1. The membrane chamber was constructed of stainless steel. All other fittings are soft brass, joined by $^1/_8$-in. copper tubing. The null section consists of a $^3/_4$-in. lucite tube with a 0.040-in. bore, screwed by means of $^3/_8$-in. pipe threads into brass fittings. Pipe threads were used throughout. The copper tubing is connected to other fittings by means of standard compression sleeve fittings. The five valves used in the apparatus were specially constructed piston type plug valves similar to those described by Bishop and Henkel (3). Sealing between the piston and bore is accomplished by "O" rings.

The steel membrane design was a compromise between flexibility for adequate displacement sensitivity at the null section and rigidity for adequate strength under the prestress air pressure. The membrane was treated as a circular plate with clamped edges, (13), and designed for a prestress air pressure of 200 psi. The final design dimensions are shown in Figure 2. The critical seal between the top and membrane sections was accomplished by use of an inset lead gasket.

Temperature changes cause unequal expansion (or contraction) of the steel chamber and the enclosed water. Calculations show that the pressure-temperature response of the device is + 1.1 psi per deg F. As ambient temperature can be controlled within a degree during the duration of most tests and the precision of the pressure gage used is ± 1 psi, it is felt that this effect is minor.

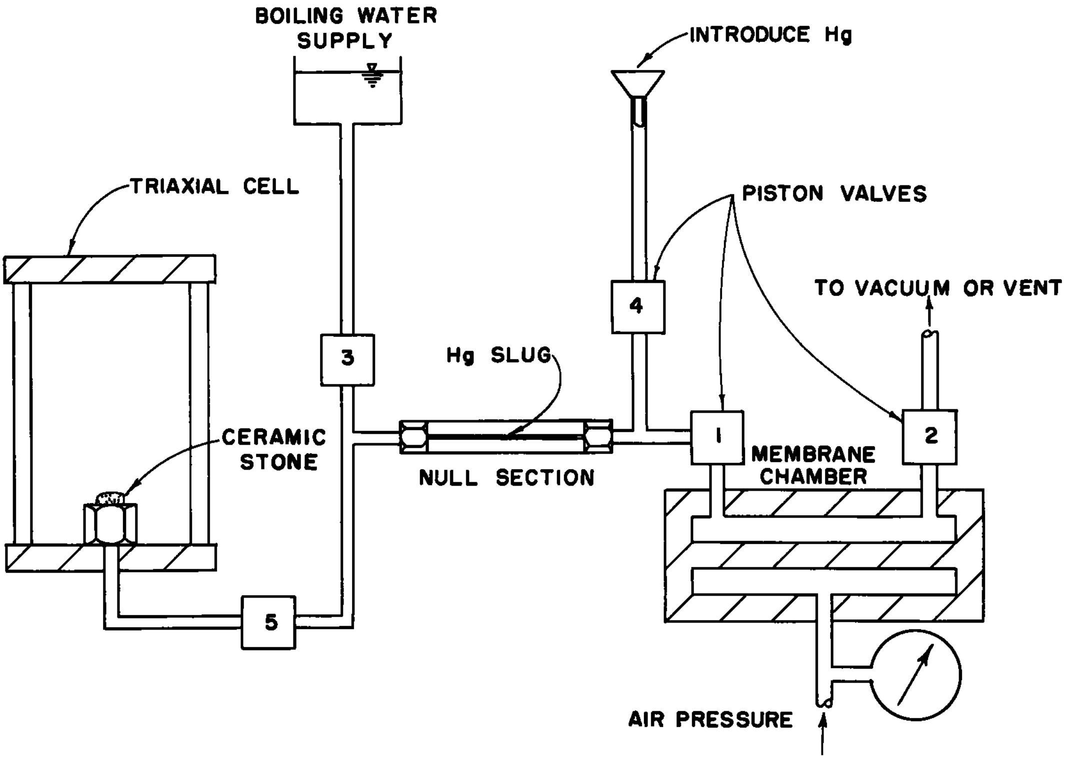

Figure 1. Schematic of device.

The ceramic stone used is a 0.5 in. diameter by 0.9 in. long cylinder of Linde alumina, fired at 1,000 C. The stone was prepared by the Ceramics Division, Department of Metallurgy, Massachusetts Institute of Technology. The stone is attached to the system by means of a "Flex" fitting, consisting of a compression fitting acting on a rubber compression sleeve.

A major construction problem to be overcome was the possible entrance of air into the system through valves, fittings, construction joints, and the pores of the ceramic stone. Such entrance of air would immediately cause cavitation in the tensioned water system.

All joints are water-filled during preparation for testing. Tight joints will act as two-dimensional capillaries. They will not drain in the presence of air until the pressure differences across them exceed their capillary head. Assuming the surface tension of water as 0.075 gm/cm, it can be shown that the joints will exclude air under a water tension of 10 atm if the joint width is less than 6 micro-in. The same reasoning applies to the maximum permissible radius of continuous pores in the ceramic stone. These dimensions cannot be measured; satisfactory joint design can only be proven during calibration of the equipment. Nonetheless, joints of maximum possible tightness remained as a prime requirement.

Preparation of the Device for Testing

It is essential that the device be completely filled with de-aired water. Because the presence of the smallest air or water vapor bubble will allow cavitation to occur, extreme care must be taken in filling the device. A step-by-step preparation procedure is outlined in Table 2. Valve numbers refer to Figure 1.

CALIBRATION

If the water and the device itself were perfectly incompressible, the induced tension in the water would be exactly equal to the reduction in the air pressure acting upon the membrane. However, because volume changes occur in the device with changes in pressure, the membrane will deflect somewhat, even though the mercury slug is maintained stationary. As the steel membrane is very stiff, this small deflection due to volume changes results in a water tension smaller than the drop in air pressure. If a 1 to 1 correspondence between air pressure drop and induced tension is defined as an

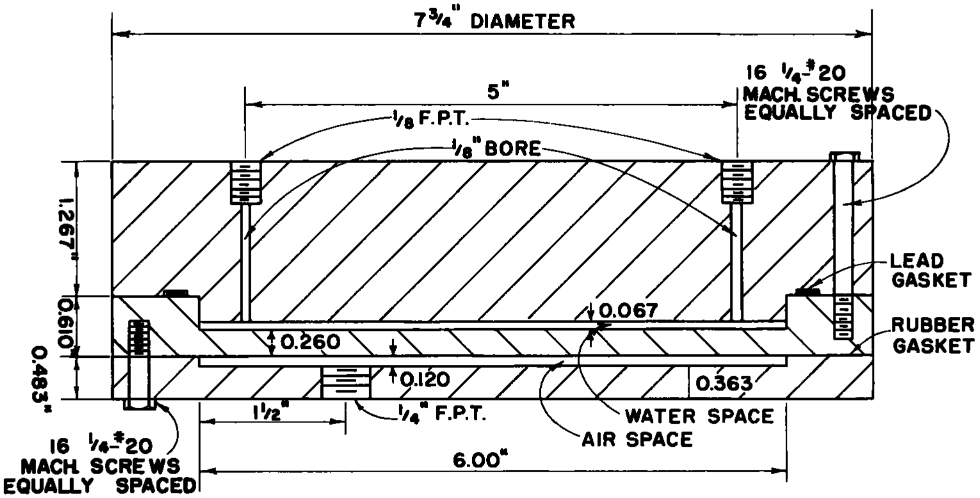

Figure 2. Steel membrane details.

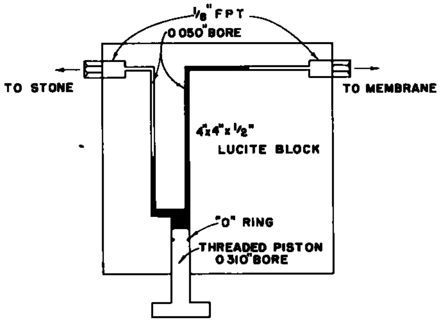

Figure 3. Modified null section.

efficiency of 100 percent, calculations show that the compressibility of the water alone will reduce the efficiency to 95 percent. Compression of fittings, valves, gaskets, etc., will reduce the efficiency still more. Just how much must be determined by calibration tests.

The deflection behavior of the steel membrane was investigated to determine whether it was operating within its elastic range. If not, fatigue and plastic deformations would preclude any consistent calibration of the device. The device was prepared in the usual manner except that the stone was left out and valve 5 remained closed. Before air pressure was applied to the membrane, valves 1 and 3 were opened; all other valves remained closed. The displacement of the mercury null versus applied air pressure was measured up to a maximum air pressure of 200 psi. The air pressure was decreased by steps to zero. This process was repeated several times. It was found that the null movement was perfectly reversible, so that the membrane operates within its elastic range. The flexibility constant of the membrane was found to be 3.7 x 10^{-4} in.^{3}/psi, which compares favorably with the theoretical design value of 3.1 x 10^{-4} in.^{3}/psi.

Because of the compressibility of the system between the null section and the ceramic stone, the system is not truly a "no flow" device when the mercury slug is maintained in a constant position in the null section. The resulting time lag to equilibrium pres-

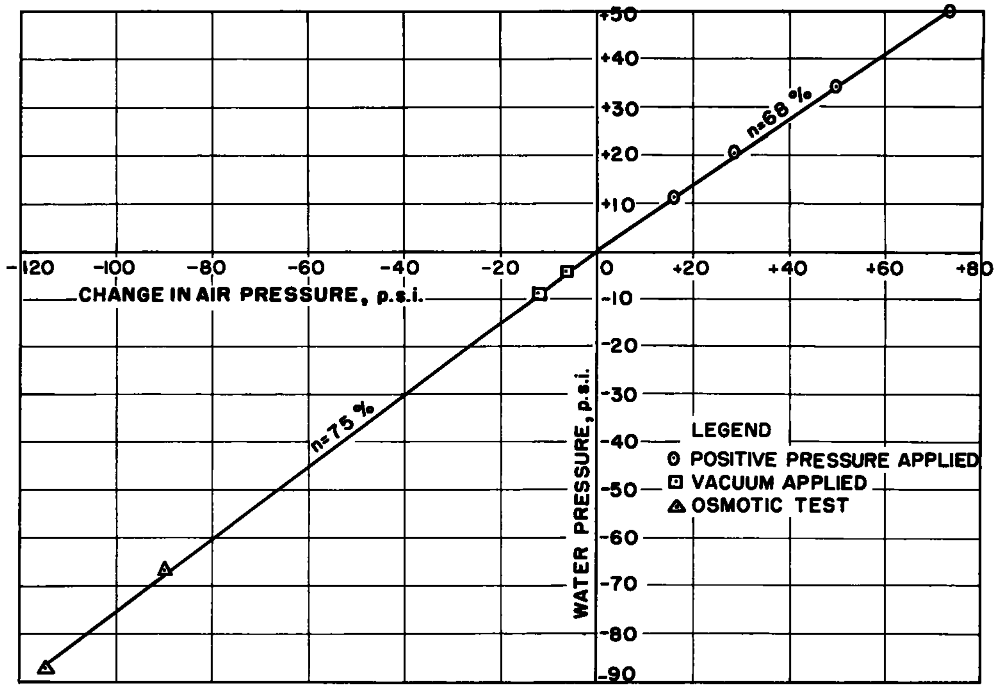

Figure 4. Calibration data.

sure depends upon the system compressibility and the conductivity of the stone—the ease with which the stone allows water movement under pressure. The stone conductivity was measured by preparing the device in the usual manner described in Table 2. The porous stone assembly was inserted in a triaxial chamber filled with water. An air pressure was applied to the triaxial chamber, and valve 1 closed and valves 4 and 5 opened. The time-displacement of the mercury null slug was observed (this is basically a constant head permeability test). The conductivity of the stone was 0.0051 cc/atm-min. This relatively low conductivity value led to a noticeable time lag to equilibrium pressure in the over-all calibration tests described later. A modified null section that will eliminate this undesirable time lag is now under construction (Fig. 3). While the mercury slug will be maintained in a fixed position as before, an adjustable piston will allow control of the volume of the system between the fixed mercury slug and the ceramic stone, thereby compensating for the compressibility of that part of the system.

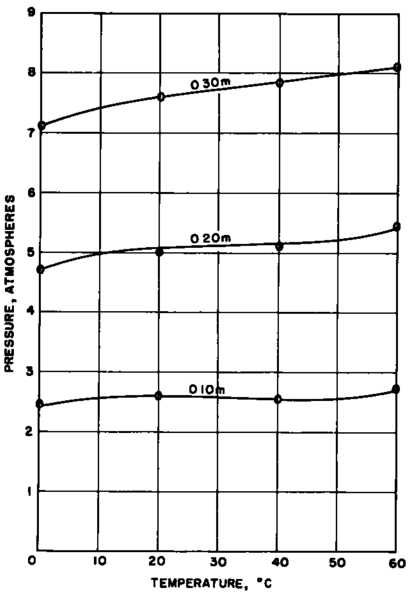

Figure 5. Osmotic pressures (after Berkeley, Hartley, Morse, and Frazer).

The air-entrance pressure of the ceramic stone—the air pressure applied to the water-saturated stone at which air will displace water from the pores—is of critical importance when measuring water tensions in partially saturated soils. When the

TABLE 2

PREPARATION PROCEDURE

1. All valves closed.
2. Open valves 1 and 2; evacuate system with vacuum pump attached at valve 2.
3. Attach boiling distilled water supply at valve 3.
4. Close valve 1; open valve 3; after several minutes open valve 5.
5. When porous stone compression fitting is overflowing with water, insert stone (previously boiled in distilled water) and screw on compression fitting. Close valve 5.
6. Open valve 1. After water flow through the system has occurred for 10 minutes, close valve 2 and remove vacuum pump.
7. Allow 10 minutes for the hot water under positive head to fill system, collapsing any vapor pockets left by the vacuum.
8. Open valve 4, filling hose up to funnel. Close valves 4 and 3, remove water supply hose at 3.
9. Allow system to cool to room temperature.
10. Introduce slug of mercury through funnel, open valve 4.
11. Partially open valve 3, letting out sufficient water until mercury slug appears in lucite null section.
12. Close valve 3, then 4.
13. Open valve 2 and apply desired prestress air pressure to bottom of membrane.
14. Close valve 2.

The system is now ready for operation.

measured tension exceeds the air-entrance pressure, air can enter the system through the stone and cause cavitation. The air-entrance pressure of the stone was measured by assembling the device as described in the above paragraph and then applying air pressure directly to the saturated stone (a pressure membrane technique). The air pressure was increased in 5 psi increments. There was no noticeable movement of the mercury null until an air pressure of 35 psi was reached. At that point the air overcame the capillary head of the stone, and the mercury null moved at a slow, constant rate away from the stone.

The air-entrance pressure of the ceramic stone is small compared with the design range of the device. However, there are ceramic stones available with much larger air-entrance pressures. Kemper (9) has experimentally determined air-entrance pressures of 9 atm in some commercially produced ceramic stones.

The over-all calibration of the device was a relatively simple matter in the pressure range from -12 to +80 psi. Under test conditions the mercury null is held in a constant position by adjusting the membrane air pressure. Therefore, it is only the compressibility of the system from the null slug back towards the membrane that influences the calibration of the system. The ceramic stone was removed and a pressure (or vacuum) line was attached at valve 5 (Fig. 1). Different magnitudes of pressure and vacuum were applied at valve 5, and the membrane air pressure change necessary to prevent movement of the mercury null was recorded. These data are plotted in Figure 4.

It was necessary to find some method for calibrating the device in the range of moderately large tensions. Some means of obtaining large tensions of known magnitude was required. The phenomenon of osmosis was finally decided upon (a more detailed description of the phenomenon can be found in (10)).

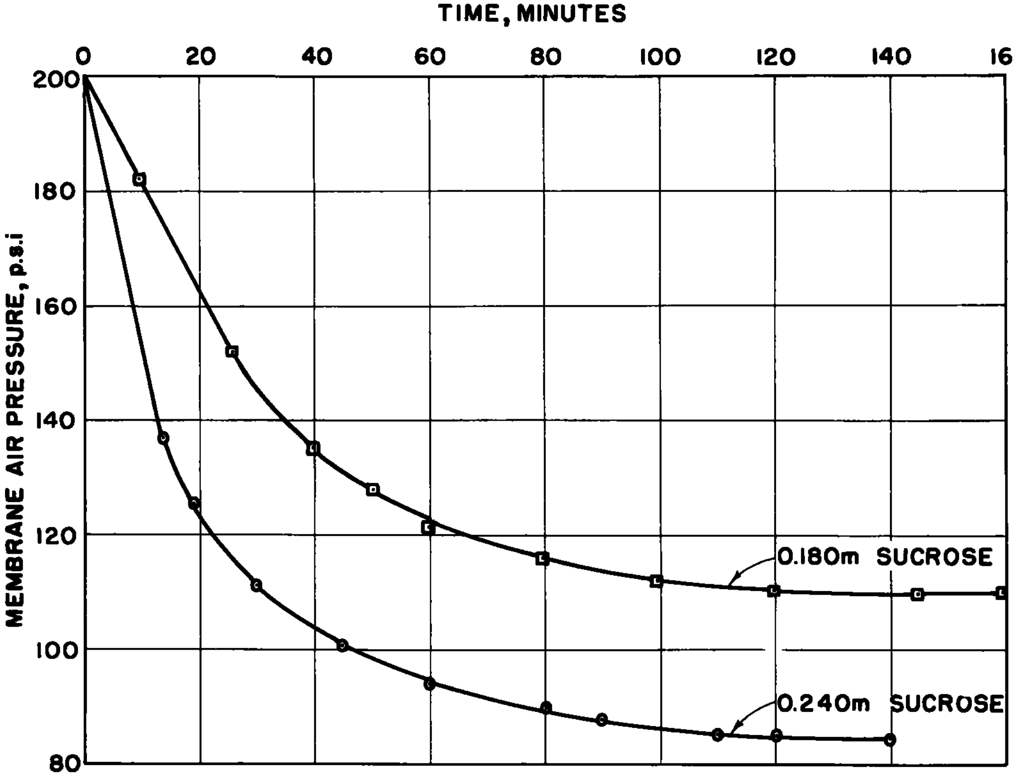

Figure 6. Time lag, osmotic calibration tests.

If a chemical solution is separated from a supply of pure water by a semi-permeable membrane—permeable to water but impervious to the chemical solute—there is a tendency for the solution to dilute itself to a uniform concentration throughout. As the solute cannot pass the membrane, the distilled water tends to flow through the membrane into the solution. If the movement of water is prevented by mechanical means, the resulting pressure P acting across the membrane is referred to as the osmotic pressure.

For dilute solutions the magnitude of osmotic pressure can be predicted accurately by van't Hoff's equation:

$$PV = nRT$$

where P = osmotic pressure
V = volume of solution
n = number of molecules of solute
R = gas content
T = absolute temperature.

For relatively concentrated solutions the van't Hoff equation loses some of its accuracy in the same manner that the ideal gas law ceases to apply at large pressures. Osmotic pressures then become a matter for experimental determination.

Osmotic pressures were first measured with some degree of accuracy by Pfeffer in 1877. More accurate determinations were made between 1901 and 1923 by Berkeley, Hartley, Morse, and Frazer (12). Their results with aqueous sucrose solutions are shown in Figure 5. It can be seen that there is a small temperature dependence, as would be expected from the van't Hoff equation. At 20 C, their results can be expressed empirically as:

$$P = 25.3\ m$$

where P = osmotic pressure, atm.
m = molarity, gram formula wt/1000 g H_2O.

In the above experiments positive pressures were measured; that is, the pressure applied to the solution to prevent water inflow through the semi-permeable membrane. In the calibration of the device, negative pressures were measured—the necessary tension applied to the distilled water to prevent inflow into the semi-permeable membrane.

It must be admitted that the ceramic stone is not a perfect semi-permeable membrane. It was felt, however, that diffusion of sucrose molecules through the stone in the relatively short time to equilibrium would be negligible—this was borne out by determinations of the "down-stream" sucrose concentrations after the tests.

Two osmotic calibration tests were run with aqueous sucrose solutions of 0.180 m and 0.240 m respectively. The system was filled with de-aired distilled water as described previously. The porous stone was inserted into a triaxial chamber which was filled with 2 liters of the sugar solution. The membrane air pressure was regulated to prevent movement of the mercury null towards the stone. A plot of air pressure versus time is shown in Figure 6. There is a pronounced time lag due to the low conductivity of the porous stone and the compressibility of the system. After completion of the tests, samples of the originally distilled water below the stone were tested for sugar content by evaporation. The sugar concentration was less than 0.001 m.

The predicted osmotic pressures based on the data shown in Figure 5 are plotted versus membrane air pressure change in the calibration curve of Figure 4.

CONCLUSIONS

Examination of Figure 4 shows two linear calibration regimes: one for positive pressures corresponding to an efficiency of 68 percent, and one for negative pressures corresponding to an efficiency of 75 percent. Because all of the fittings are tapered pipe threads or tapered copper compression sleeves, it is not surprising that the system is less compressible under water tensions. The fact that the negative pressures determined by the direct application of vacuum are consistent with the negative pressures determined by osmotic calibration is an indication of the validity of the calibration method.

In conclusion, it has been demonstrated that the device described can accurately measure water tensions to a magnitude of 90 psi in saturated systems. The limiting tension of the device when used with partially saturated soils is the air-entrance pressure of the ceramic stone.

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Mechanisms of Swelling by Compacted Clay

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The mechanisms believed to cause swelling in saturated clay-water systems are first reviewed. These concepts, drawn primarily from soil chemistry and soil physics, are extended to compacted natural clays. Test data are presented to show the effects of the ion concentration in the pore fluid on the swelling behavior of a highly plastic clay. These data consist primarily of heaving and fluid pickup measurements on samples molded with pure water and soaked in solutions of varying salt concentrations.

For the clay investigated, it is concluded:

1. For samples compacted wet of optimum water content, swelling can be explained by osmotic repulsive pressures arising from the difference in ion concentration in the double-layer water between interacting clay particles and that in the free pore water.
2. For samples compacted dry of optimum water content, swelling is influenced by factors in additions to osmotic pressures. These other factors may be: the effect of the negative electric and London van der Waals force fields on water, cation hydration and the attraction of the particle surface for water, elastic rebound of particles, a flocculated particle orientation, and the presence of air. The relative importance of these other factors is not known.

● IN SEVERAL areas of the world the differential heaving of foundations due to the swelling of highly plastic clays has resulted in severe damage to buildings. Swelling of clay also changes the engineering properties of strength, compressibility, and permeability. For many earth structures, such as road subgrades and embankments, a loss of strength or an increase in compressibility will be of greater concern to the soil engineer than the heaving per se. As the use of clay, and especially highly plastic clay, for earth structures increases, the need for a better understanding of the swelling phenomenon becomes greater, since the soil engineer must not only know the "as-compacted" properties of clay, but also know how these properties change with time.

The swelling behavior of a compacted clay will be governed primarily (but not solely) by the following factors:

1. Composition of the clay—composition and amount of clay minerals, nature and amount of exchangeable cations, proportions of sand and silt in the clay, and presence of organic matter and cementing agents.
2. Compaction conditions—molded water content, dry density, degree of saturation, and type of compaction.
3. Chemical properties of the pore fluid—both that during compaction and that which is imbibed during swelling.
4. Confining pressure applied during swelling.
5. Time allowed for swelling.

This paper is an effort to explain the swelling phenomenon in compacted clays. This work is part of a fundamental study of the behavior of fine-grained soils that is currently being carried on by the Soil Engineering Division at MIT.

Data are presented to show the effects of salt content in the pore fluid on the swelling characteristics of a compacted natural clay. The data consist primarily of heaving and fluid pick-up measurements on samples molded with pure water and then soaked in solutions of varying salt concentration. The idea for this study came from research in soil physics and soil chemistry by such workers as Bolt (3) and Hemwall and Low (5). These workers ran consolidation-rebound tests on fractionated samples of pure clay minerals and showed that an osmotic repulsive pressure is developed between clay particles that causes rebound or swelling when the effective stress (total pressure minus pore pressure) on the sample is reduced. This repulsive pressure is proportional to the difference in salt content in the water between the clay particles and that in the "free" pore water.

The author's data indicate that osmotic repulsive pressures play an important role in the swelling of a compacted natural clay since an increase in the salt content of the soaking solution was found to reduce the amount of swelling by the soil. It was found, however, that swelling is influenced by factors in addition to osmotic pressures. Other possible swelling mechanisms are discussed.

Since principles of physical and colloidal chemistry and related fields are important to an understanding of swelling, the first portion of the paper summarizes those physiochemical properties of saturated clay-water systems which are thought to influence swelling. These concepts are then extended to compacted natural clays. Only inorganic clays are considered. While the theoretical considerations are somewhat oversimplified and the experimental data and conclusions admittedly incomplete, it is hoped that the reader may at least gain a better insight into the causes of swelling by compacted clay.

THEORETICAL CONSIDERATIONS

Swelling in Saturated Clays

Swelling of saturated clay is considered first, since its behavior is somewhat simpler and better understood than that for a partially saturated soil such as compacted clay. An excellent example of the swelling behavior of the former is the rebound portion of a standard consolidation test on a saturated clay for which any volume increase in the clay sample is accompanied by an equal (neglecting changes in the density of water) increase in the volume of water in the clay.

The Clay Micelle. — Figure 1 represents schematically a clay particle immersed in pure water. Sufficient exchangeable cations (the effect of the hydrogen and hydroxide ions in water are not considered in this paper) surround the particle and are attracted to it by a net negative charge in the clay particle in order that the cations plus the particle constitute an electrically neutral system. This system is designated the clay "micelle." The ions and water within the micelle constitute the "double layer." If the clay particle were immersed in a salt solution instead of pure water, then anions would also be present in the double layer, but the number of cations would be increased accordingly in order that the micelle still remain electrically neutral. In other words, the double layer includes that portion of the water surrounding the particle in which there is a negative electric field requiring an excess of positive charges relative to negative charges.

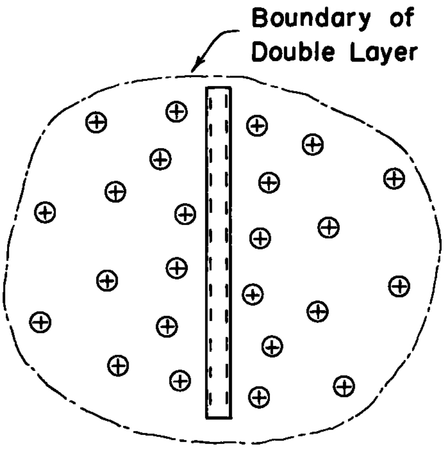


Figure 1. The clay micelle in pure water.

The variation in cation and anion concentration with distance from the clay particle surface for suspensions of clay in water can be calculated from principles of colloidal chemistry, the Gouy-Chapman theory being an example (8). Calculations based on such theories are quantitatively correct for the ideal systems for which the theories were developed; however, most natural soils are far from these ideal systems so that one can only use such theories in a qualitative manner when dealing with soils.

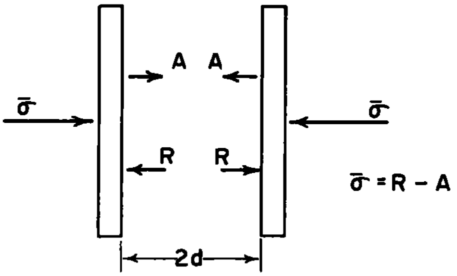


Figure 2. Forces between two clay particles.

Forces Between Clay Particles.—The magnitude of rebound in a standard consolidation test is, of course, directly related to the decrease in effective stress. In turn, the effective stress $\bar{\sigma}$ can be related to physicochemical forces acting between clay particles, and it is these latter forces that are basically responsible for swelling. Lambe (11) represents this relationship between forces by the expression (Fig. 2)

$$\bar{\sigma} = R - A$$

for parallel particles at an equilibrium interparticle spacing of 2d (this relationship and the following discussion of the forces involved apply to interparticle spacings of greater than 10 to 20 Å.) where: (assuming unit area)

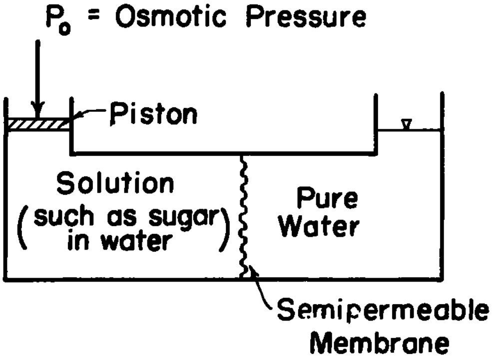


Figure 3. The osmotic pressure of a solution.

1. A = the attractive pressure which is usually considered to be caused by London van der Waals secondary valence forces of attraction between the adjacent clay particles. (This secondary valence force is primarily a function of particle thickness and interparticle spacing and decreases rapidly with increasing interparticle spacing (8).)
2. R = the repulsive pressure which arises from the interaction of the double layers associated with the two clay particles. This pressure will be discussed.

A parallel particle alignment is assumed in order to simplify the concept of interparticle forces. "Edge effects" arising from electrical charges at the edges of the particle are neglected. These edge effects may cause an additional attractive force between particles that can lead to a nonparallel particle orientation (11).

All natural clays will swell or rebound when the effective stress $\bar{\sigma}$ is reduced and many, particularly if remolded (remolding tends to align clay particles into a more nearly parallel orientation and also tends to break attractive bonds that may have existed between particles at points of contact prior to remolding, Lambe (11)), will even slake completely if $\bar{\sigma}$ is reduced to zero (for example, the result of immersing an unsupported chunk of clay in water). Hence, for these clays R must be greater than A for at least some of the particles, and the mechanism of swelling can be studied by investigating the nature of the repulsive pressure R.

It is generally believed, Low and Deming (12), that this repulsive pressure has several components. One of the most important components is thought to be caused by an osmotic pressure (This osmotic pressure is the "electricl repulsion" referred to by other writers such as Lambe (10, 11). The higher ion concentration causing osmotic pressures arises, after all, from the electric field in the double layer.) arising from

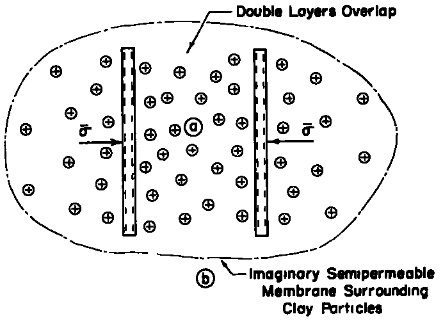


Figure 4. Osmotic pressure between two clay particles immersed in water.

the higher ion concentration in the double-layer water of the clay micelle than in the "free" pore water, that is, that water outside of the micelle. The nature of the other components will be presented after a discussion of osmotic pressures.

Osmotic Repulsive Pressure (Principle). —When an aqueous solution is separated from pure water by a semi-permeable membrane, that is, a membrane that permits the passage of water but not that of the substance dissolved (solute) in the solution, water tends to pass through the membrane into the solution, thereby diluting it. This phenomenon is called "osmosis." The pressure that must be applied to the solution in order to prevent the flow of water into the solution through the semi-permeable membrane is called the "osmotic pressure" of the solution. This is shown diagramatically in Figure 3.

Although a mechanistic picture has not yet been developed to explain fully osmosis, there are formulas for calculating osmotic pressures. One of the simplest of these is the van't Hoff equation which yields

$$p_o = \bar{R}Tc$$

where p_o = osmotic pressure (g per sq cm), $\bar{R}$ = gas constant, T = absolute temperature ($\bar{R}T = 2.5 \times 10^7$ g cm per mole for 20 C), and c = concentration of solute (moles per cc of solution). (The van't Hoff equation as presented is only strictly applicable to very dilute solutions of nonelectrolytes. Measured osmotic pressure values exceed calculated values for most other cases, Prutton and Marion (17).) Osmotic pressures can reach very large magnitudes. For instance, 130 gm of sugar per liter of aqueous solution exerts an osmotic pressure of about 10 tons per sq ft. Osmotic pressures can, of course, be developed between two solutions of unequal concentration so that

$$p_o = \bar{R}T\,(c_a - c_b)$$

where c_a and c_b refer to the solute concentrations on either side of a semi-permeable membrane.

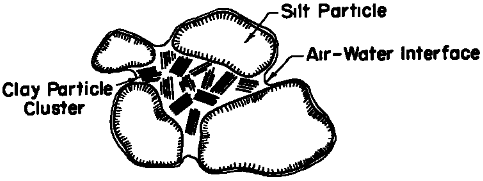
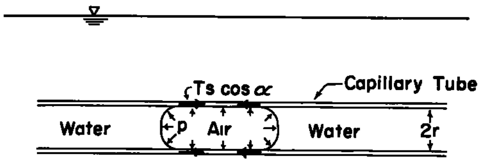


Figure 5. An illustration of pore water tensions in compacted clay.

Osmotic Repulsive Pressure (in Clays). —Osmotic pressures can act in clays since: (a) there exist differences in solute concentrations (in this case ions are the solute), and (b) the electric field around the negatively charged clay particle acts as a semi-permeable membrane. This can be illustrated by picturing two clay particles that have been immersed in a beaker of pure water and pushed under an effective stress $\bar{\sigma}$ to an interparticle spacing of 2d, as shown in Figure 4. (Pure water is used to simplify the illustration, because in this case the only ions in the double layer will be the exchangeable cations.) Because of the exchangeable cations, the concentration of ions in the double-layer water is larger than the concentration of ions in the free water. The exchangeable cations are attracted to the clay particles by the negative electric field arising from the negative charge on the particles. Hence the electric field acts as a semi-permeable membrane in that it will allow water to enter the double layer but will not allow the exchangeable cations to leave the double layer. The dashed line in Figure 4 depicts figuratively this "semi-permeable membrane." One can see that due to the difference in ion concentrations between points such as "a" and "b", water would like to flow from "b" to "a" and that an effective stress $\bar{\sigma}$ (plus the attractive pressure A)



$$p \approx \frac{2Ts}{r} \cos \alpha$$

where Ts = surface tension in water
r = radius of capillary tube
α = contact angle

Figure 6. Air pressure in capillary tube.

is required to prevent an increase in interparticle spacing, that is, swelling. If $\bar{\sigma}$ is reduced, then water flows from "b" to "a", thus decreasing the ion concentration at "a" (in other words, the double layer is expanded) until the correspondingly lower osmotic pressure is again in equilibrium with the effective and attractive pressures.

At this point one might re-examine the meaning of the term "pore water pressure" as used in soil engineering. When the soil engineer measures pore pressures, as for instance with a piezometer, he measures the pressure in the "free pore water" (that water outside of the double layer). For a particle arrangement as shown in Figure 4, the pore pressure u as used in soil engineering is, therefore, the pressure in the water as a point such as "b". The total pressure in the double-layer water (to mean water plus ions) at a point such as "a" midway between clay particles is, however, greater than u by a pressure equal to the developed osmotic pressure (and possibly other pressures to be mentioned) and this pressure increase is counterbalanced by ($\bar{\sigma}$ + A). This assumes that the pore pressure u also acts between particles. For a detailed discussion of variations in water pressures throughout a clay-water system, see Low and Deming (12).

Osmotic Repulsive Pressure (Factors Influencing).—The ion concentration differential that determines the osmotic repulsive pressure between particles is the ion concentration at the midplane between particles (that is, point "a" in Fig. 4) minus the ion concentration in the water outside of the double layer (the free pore water). Thus, based on the van't Hoff equation, the osmotic pressure becomes

$$p_o = \bar{R}T (c_c - c_o)$$

where c_c and c_o refer to the total ion (both cations and anions) concentration (moles per cc) at the midplane and in the free pore water, respectively. (The reader is referred to Bolt (3), Kruyt (8), Low and Deming (12), Hemwall and Low (5) and Lambe (11), for a more detailed presentation and discussion of these and other factors and for additional reference material.)

Whereas the ion concentration in the free pore water c_o can be easily measured, the midplane concentration c_c must be computed from a theory relating ion concentration with distance from the clay particle surface. For certain ideal cases, the Gouy-Chapman theory has been used and the calculated osmotic pressures checked experimentally. Bolt (3) and Warkentin, et al. (22) ran consolidation-rebound tests on samples of fractionated (minus 0.0002 mm) montmorillonite and illite with various exchangeable cations and pore water ion concentrations over a pressure range of from 0.1 to 50 tons per sq ft. Their data show that the compression-swelling curves based on osmotic pressures computed from the Gouy-Chapman theory agreed qualitatively and in some cases almost exactly with the observed curves, and in all cases the observed trends corresponded with theory. (Bolt and Warkentin, et al. (22) hypothesize that unknown variations in particle shape and spacing can account for much of the discrepancy between the computed and measured curves.)

Of main interest, the Gouy-Chapman theory tells us (as does intuition) that for a given clay:

1. For a constant interparticle spacing, p_o decreases with increasing ion concentration in the free pore water.
2. p_o decreases with increasing interparticle spacing.

Furthermore, the data by Bolt and others indicate that variations in osmotic pressures, and hence swelling, as predicted by the Gouy-Chapman theory, should also apply in a qualitative sense to natural clays for interparticle spacings exceeding 10 to 20 Å. A quantitative treatment of natural clays is, of course, impossible because the extreme variations in particle size, shape, composition, orientation, and spacing preclude any realistic computation of interparticle forces.

Other Repulsive Pressures and Swelling Mechanisms.—The previous discussion has shown that an osmotic pressure can reasonably account for a repulsion between clay particles which results in swelling, or in other words, an expansion of the double-layer, when the effective stress on a clay sample is reduced. Other factors may also contribute to swelling. One of these is the effect of secondary valance or London van der Waals forces on the water surrounding clay particles, Hemwall and Low (5) and Low and Deming (12). Another is the effect of the negative electric field on the double-layer water. Both of these force fields are believed to attract water to the soil particles, although the latter force field is not thought to contribute to R for particles at equilibrium, Low and Deming (12). Whereas there are fairly extensive data indicating that osmotic pressures play a very important role in swelling, at least for certain clay-water systems, there are no known data from which definite conclusions can be drawn relative to the magnitude of influence of the above two factors on swelling.

As previously noted, the foregoing discussion has been restricted to interparticle spacings, 2d, greater than 10 to 20 Å. (The double-layer thickness would then be 5 to 10 Å.) For smaller spacings, the nature and interrelationship of $\bar{\sigma}$, R, and A may change radically. (At these small interparticle spacings, such as one might expect in a highly compressed clay, the likelihood of having A greater than R may be increased for many of the interacting particles, Lambe (11), p 1655-20.) For these smaller spacings (among other things):

1. Osmotic pressures may not be developed; in other words, the "normal" double-layer is not formed, MacEwan (13) and Norrish (15).
2. Water adsorption is generally thought to be governed primarily by hydration of the exchangeable cations and the attraction of the clay particle surface for polar molecules like water, Barshad (2), MacKensie (14) and Norrish (15).

Mention should be made of swelling due to elastic rebound and "unbending" of soil particles. It is felt, Lambe (11), that this phenomenon, while important in coarse-grained soils and in soils containing relatively large platey shaped particles (like mica), is of little importance for most clays. Bolt (3) discusses the importance of these "mechanical" effects versus the "physicochemical" effects for natural soils.

Swelling in Compacted Clays

In attempting to utilize the foregoing swelling concepts to explain the swelling behavior of a compacted clay, one is confronted with the following complications. As the molded water content changes (the reader is referred to Holtz and Gibbs (7) for extensive data on the swelling characteristics of compacted clays) for a given compactive effort:

1. The dry density of the soil and thickness of the double-layer water around clay particles vary.
2. The particle orientation varies.
3. The pore water tensions vary.
4. The degree of saturation, and hence the amount of air in the sample, varies. The pressure in the air may also vary.

The above factors are discussed relative to their possible effects on swelling behavior and the mechanisms involved.

Effect of Molded Water Content on Dry Density and Double-Layer Thickness.—The thickness of the double-layer water on compacted clay particles is nearly always less than that which the particles would like to have if given free access to water. (Silt and sand size particles also have "double-layer water." However, these particles have a relatively small specific surface area that is, surface area per unit weight of soil, so that the effect on the over-all volume change of an expansion of the double-layer around these particles is small compared to that for clay size particles where the specific surface area is large.) For instance, many plastic clays under zero confining pressure will imbibe water until they reach a water content approximating the liquid limit, whereas the clays may be compacted at water contents less than the plastic limit. The difference between these two water contents has been termed the "double-layer deficiency" Lambe (11). This deficiency is thought to be caused by the same factors causing swelling, that is, a double-layer expansion, in saturated clays (for example, osmotic pressures, effect of force fields on double-layer water, etc.). If the thickness of double-layer water is less than 5 to 10 Å (corresponding to a 2d spacing of 10 to 20 Å) the double-layer deficiency may in part be due to cation hydration, etc., as previously mentioned. A double-layer thickness of 5 to 10 Å is approximated by the water content of the soil at 99 percent relative humidity. This water content, even for extremely plastic soils, seldom reaches 10 to 15 percent (pure montmorillonite is an exception) and is usually only a few percent for most lean clays. Hence for many clays, the water content used for compaction may be sufficient for a double-layer thickness of greater than 5 to 10 Å.

The thickness of the double-layer is roughly proportional to the molded water content (assuming all water to be in the double-layer, the average thickness of the double-layer is then equal to the water content divided by the specific surface area). Hence the lower the molded water content (other things being equal), the greater is the water uptake required to satisfy the double-layer deficiency; and for a constant molded water content, an increase in dry density would lead to an increase in the amount of swelling.

Effects of Molded Water Content on Particle Orientation. —There is evidence, Pacey (16), to show that the orientation of clay particles changes with molded water content. These data indicate that compaction dry of optimum water content leads to a nonparallel or flocculated orientation while compaction wet of optimum leads to a parallel or dispersed orientation of clay particles. Different methods of compaction may also yield different particle orientations, even at the same density and water content. One might expect different particle orientations to cause a difference in swelling behavior. Seed's (18) data suggest that a flocculated particle orientation will swell more than a dispersed orientation.

Effects of Molded Water Content on Pore Water Tensions and Degree of Saturation. — Pore water tensions (water pressures less than atmospheric) undoubtedly exist in compacted clay, particularly if compacted dry of optimum water content where the initial degree of saturation is well below 100 percent, Aitchison (1), Soil Mechanics for Road Engineers (19) and Hilf (6). Tensions are caused by the double-layer deficiency in the clay micelles. (As previously pointed out, one should also include the larger size particles since they also desire water.) In other words, all the particles in the soil are competing for the limited supply of water in order to expand their double-layers. Capillarity may also enter the picture if the voids in the soil contain both air and "free" water.[1] The desire of the clay micelles to imbibe this free water would be resisted by the surface tension at the air-water interface in the void.[2] An illustration of this phenomenon is shown in Figure 5 which depicts clusters of clay particles compacted between silt particles. The inflow of water to the clay is restrained by the menisci in the pores between the silt particles. This illustration is only one example of the many possible situations that might occur, since a compacted clay will contain a wide variation in the size and shape of particles and voids.

In summary, then, pore water tensions in a compacted clay represent a balance between double-layer deficiencies and surface tensions at air-water boundaries. Another way one might look at pore water tensions is to say that they exert an effective stress

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[1] Capillarity in the same sense as the capillary rise of water in a glass tube. The author, when using this term, is referring in general to relatively large pores in the soil (for example, diameters of several thousand Å or larger) where the amount of double-layer water is relatively small compared to the "free" pore water. Although the fundamental cause of capillarity may originate from the electrical nature of the soil particles, the term capillarity is a widely used concept which seems applicable in this case.

[2] Calculations based on the capillary equation, Taylor (20), relating pressure, surface tension, and pore diameter show that a 3,000 Å diameter pore can resist a tension of about 10 atmospheres. Bolt's (3) data for Na montmorillonite show that a swelling pressure of 10 atmospheres corresponds to a particle spacing of only about 30 Å (double-layer thickness of 15 Å).

on the compacted clay mass which prevents the particles from imbibing water and swelling. This effective stress is more of an "effect" than a "cause."

When a compacted clay sample is put in contact with water, any air-water menisci at the surface of the sample are broken. Water will flow into the clay because of the pore water tensions within the sample, these tensions having been caused by a combination of double-layer deficiencies and capillarity. With time, the pressure in the pore water increases to atmospheric with a resultant lowering of the effective stress within the sample. Concurrently, the clay micelles expand their double-layers and swelling occurs between clay particles, just as for the saturated clays, until the repulsive pressure minus the attractive pressure between particles is in equilibrium with any applied effective stress. Over-all swelling of the sample can, of course, be prevented by applying an effective stress to the soil equal to the "swelling pressure."

The Role of Air in Swelling.—Previous discussion has mentioned that the presence of air can influence the magnitude of the pore water tensions that are developed in compacted clay. Air may also influence the swelling behavior in another manner. Swelling data, Holtz and Gibbs (7), Figures 8, 10 and 11, usually show that the total volume of air in a compacted sample decreases during the soaking process, particularly for samples compacted dry of optimum water content, although the final degree of saturation is still less than 100 percent. Thus, during the soaking process, some of the air initially in the soil voids must either escape from the soil, be dissolved by water, or be compressed by capillary forces. Most likely a combination of these conditions occurs. If a nonspherical pocket of air is compressed in a soil void during the soaking process, the pressure in the air may produce tensile stresses in the soil skeleton forming the void in which the air resides (a long, initially air filled pore having a relatively large diameter of several thousand Å or larger is visualized). These tensile stresses could cause an increase in the volume of the void, and hence an expansion of the soil.

An analogous situation is shown in Figure 6. An initially air filled capillary tube is immersed horizontally in water. As water enters the tube from both ends by capillarity, the air in the tube becomes compressed. An analysis of the stresses on a cross-section of the tube through the air pocket (there must be enough air in the tube so that the air pocket does not become spherical) shows that both axial and hoop tensions act in the tube at this location; in other words, the pressure in the air tends to expand the tube. If the capillary tube were brittle and could not withstand these tensions, it would ultimately break at the center. If the capillary tube were flexible at the center, a bulging would occur. The maximum air pressure that can be attained in a capillary tube of a given radius (assuming a sufficient amount of air) is proportional to the tensile strength of the tube or the surface tension of water, whichever is smaller.

In a similar manner, it is believed that air in a soil void, if under pressure, can cause an enlargement of the void if the soil skeleton cannot resist this pressure. These air pressures could be quite large. For example, theoretically, air could be compressed by capillary forces to a pressure of 6 atmospheres in a void having a diameter of 0.5 microns (5,000 Å). Such a pore size is not unlikely in a compacted clay.

One can probably best visualize this swelling phenomenon occurring in a soil where: (a) There are many interconnected tubular air voids; (b) the air pressure is initially atmospheric, and (c) water enters the soil from all directions during soaking. These conditions (except possibly for the shape of the voids) are met when a dry sample of clay is immersed without confinement in water. The rapid slaking of such a sample is said, Terzaghi and Peck (21), p. 129, to be caused at least in part, by the air pressure that is built up within the sample. Data will be presented which suggest that air pressures may contribute to the swelling of clay if compacted dry of optimum water content.

The above discussion dealt with the presence of air in the larger voids of a compacted clay. The presence of air in the double-layer water between interacting clay particles would also affect swelling. For example, if air were present, it would tend to expand the double layer, since it is displacing water and ions. In other words, for a given interparticle spacing, the presence of air would increase the repulsive pressure; or for a given effective stress, the interparticle spacing would be larger. The magnitude of influence on swelling of this air is, however, not known. Even the amount of air to be expected in the double layer is a matter of conjecture.

Only two "types" of air have been discussed, that is, large, fairly continuous voids of air that extend throughout the clay mass, and minute bubbles of air that exist in the double-layer water. While air undoubtedly exists in many forms in compacted clay, the above conditions are thought to represent two extreme cases which can be used for a consideration of the major effects of air on the swelling behavior of compacted clay.

EXPERIMENTAL DATA

Swelling data are presented in which compacted samples of a clay are soaked in aqueous solutions ranging from pure water to a 5 molar salt solution. According to the previously developed theory, the samples which are soaked in salt solutions should swell less than the samples soaked in pure water, since an increase in ion concentration in the water outside of the double layer reduces the osmotic repulsive pressure between clay particles. In other words, the salt solution will reduce the double-layer deficiency in the clay micelles. Data are also presented relative to the distribution and pressure of air in as-compacted samples of clay.

Description of Soil

The soil, Vicksburg Buckshot clay, was supplied to the MIT Soil Stabilization Laboratory by the Waterways Experiment Station, Vicksburg, Mississippi. The Atterberg limits, specific gravity, grain size distribution, mineralogical composition, and certain other properties are presented in Table 1. Of particular interest are: (a) The highly plastic characteristics of the soil, although only 36 percent of the soil is clay size; (b) the presence of montmorillonite, a clay mineral known to be very expansive;

TABLE 1

PROPERTIES OF VICKSBURG BUCKSHOT CLAY

1. Specific gravity[a] = 2.74
2. Atterberg limits[a]
   - Liquid limit = 63 percent
   - Plastic limit = 25 percent
   - Plasticity index = 38 percent
3. Grain size distribution
   - 94 percent minus 0.074 mm; 36 percent minus 0.002 mm
4. Mineralogical composition in percent by weight[b]

Illite	Interstratified	25 ± 3[c]
Montmorillonite		25 ± 3
Quartz		20 ± 3
Feldspar		20 ± 10
Fe_2O_3		1.1 ± 0.1
Organic matter		1.1 ± 0.1

5. Other properties[b]
   - Soluble salts (meg. NaCl/100 g) = 0.3
   - Cation exchange capacity (meg./100 g) = 30; 52 (minus 0.002 mm)
   - Glycol retention (mg/g) = 65; 135 (minus 0.002 mm)
   - pH = 4.9

Note: Items 4 and 5 obtained by R. T. Martin, Research Associate, MIT Soil Engineering Division.

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[a] On air-dried and ground soil.
[b] Unless otherwise specified, data shown for soil passing 0.074 mm.
[c] ± indicates probable uncertainty in percentages given.

and (c) the high glycol retention by the soil. Multiplying the glycol retention by 3.22 to obtain the specific surface area yields values of 210 and 435 sq m per g of clay, respectively, for the two soil fractions (by comparison Na montmorillonite has a surface area of 800 sq m per g). The exchangeable cations are believed to be predominantly calcium.

Test Procedure

The test procedure employed for the compaction-soaking tests was as follows:

1. The clay was air-dried, ground to pass a No. 20 sieve, and equilibrated for at least two days at the desired water content. Distilled water was used as the molding fluid.
2. The clay was compacted dynamically in 2.75-in. diameter by 0.85-in. consolidometer rings (fixed ring type) at an effort of 28,000 ft-lb per cu ft and trimmed to a height of 0.6 in. A surcharge of 200 lb per sq ft was applied, followed by immersion of the sample in an aqueous solution. The test set-up is shown schematically in Figure 7. Note that the soaking solution enters both ends of the compacted sample.
3. Three different aqueous solutions were used for soaking. These were: (a) Distilled water; (b) a 0.5 molar $CaCl_2$ solution (55 g $CaCl_2$ per 1,000 cc of solution. $CaCl_2$ was used to prevent ion exchange); and (c) a 5.0 molar $CaCl_2$ solution. The solubility of $CaCl_2$ in water at room temperature is approximately 5.2 molar. Hence, a 5 molar $CaCl_2$ solution is very strong.
4. Measurements of the amount of heave were taken over a four day period.
5. At the end of four days the weight of the sample was measured. The weight of solution in the sample was divided by the estimated unit weight of the solution to obtain a volume of solution. The volume of solution was treated as if it were pure water for computing water contents after soaking. (That is, if 10 g of initially dry soil imbibed 1.5 g of a salt solution with a unit weight of 1.5 g per cc, the recorded "water content" change would be 10 percent. For initially moist samples, however, the exact unit weight of solution in the sample after soaking was unknown and had to be estimated.)

TABLE 2

WATER ADSORPTION AND IMBIBITION DATA ON VICKSBURG BUCKSHOT CLAY

A. Water vapor adsorption[a]

Relative Humidity (%)	Equilibrium Water Content (%)
50	6
~ 99	13.5

B. Water imbibition under "Free-Swell" condition[b]

Initial Water Content (%)	Equilibrium Water Content (%)
3.6	64
4.7	68
4.7	67
4.7	60
8.2	55
32.4	53
50.2	51
51.0	54

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[a] On loose samples of initially oven-dried clay.

[b] On 1.5 mm thick samples of clay spread over a porous stone whose top surface was slightly above a water surface which was enclosed in a sealed container. Equilibrium content taken after 24 hr.

Swelling Data

Data on water adsorption and water imbibition under a "free-swell" condition are presented in Table 2. The data indicate that: (a) At water contents in excess of 13.5 percent (the equilibrium water content at 99 percent relative humidity), it is reasonable to assume that the exchangeable cations are hydrated and that the normal double layer is formed; and (b) under zero confining pressure, the double layer will imbibe approximately 60 percent water. (Assuming that all the water is associated with the clay-size fraction of the soil and that the surface area of this fraction is 435 sq m per g, the average thickness of water around the clay particles in Å is approximately one-half of the water content in percent; for example, a water content of 60 percent corresponds roughly to an average water thickness of 30 Å.) The molded water contents used for the compaction soaking data presented below range from 14 to 24 percent with optimum water content at 19 percent.

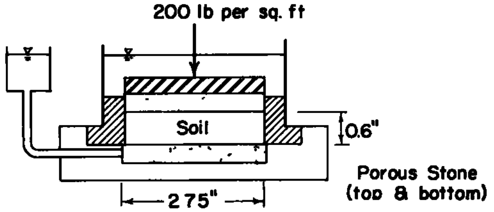

Figure 7. Compaction-soaking test apparatus.

The effects of salt concentration on swelling are shown in Figure 8 in which the molded dry density, the percent heave (change in height divided by initial height times 100), and the "water content" increase are plotted against the molded water content. The effect of salt content on the rate of swelling for samples compacted wet and dry of optimum water content are presented in Figure 9. Dry density and water content curves before and after soaking are plotted in Figure 10 for samples immersed in both pure water and in the 5 molar $CaCl_2$ solution. Figure 11 shows the relationship between volume change (cc) and water pickup (g) for samples soaked in water (since the swelling data were obtained on soil samples of unequal volume prior to soaking—variations of up to 15 percent—the observed data have been adjusted to correspond to initial samples volumes of 100 cc).

The data show that:

1. The amount of heaving and water pickup decreases with increasing molded water content.
2. The degree of saturation after soaking is less than 100 percent. The volume of water pickup also exceeds the volume of expansion, particularly for samples compacted dry of optimum water content.
3. The soaking of compacted samples in salt solutions produces a marked decrease in the amount of fluid pickup and heaving. Figure 8 shows that the absolute magnitude of this reduction in fluid pickup and heaving is fairly uniform, particularly for the strongest salt solution, over most of the molded water content range. Furthermore, the 5 molar salt solution prevents swelling for a sample compacted 2 percent wet of optimum water content.
4. The initial rate of swelling is practically unaffected by the salt concentration in the soaking solution. (A 5 molar $CaCl_2$ solution has a viscosity 10 times and a surface tension 1.3 times that of pure water so that one might expect a net decrease in the rate of swelling due to the large increase in viscosity.)

The data show that the salt content in the pore fluid has a decided effect on the swelling behavior of this compacted clay. Hence it would appear that osmotic repulsive pressures play an important role in swelling, since osmotic pressures depend upon the difference in ion concentration in the water between the clay particles and that in the free pore water. Furthermore, it would seem that the osmotic pressure concept can satisfactorily explain, at least in a qualitative sense, the observed swelling of samples compacted wet of optimum water content, since the addition of salt prevented swelling. This does not necessarily mean, however, that an osmotic pressure is the only component of the repulsive pressure R. For example, even for the sample that did not swell when immersed in the 5 molar $CaCl_2$ solution, there must be a repulsive pressure between particles in order to counterbalance the effective stress (that is, the 200 lb per sq ft surcharge) plus any attractive stresses that may act. If this strong salt solution reduced osmotic pressures to zero, then some other mechanism of repulsion must be operative even though the magnitude of its influence is small.

While it is evident that some of the swelling of samples compacted dry of optimum water content can be explained by osmotic pressures, there are certainly other factors which influence the swelling behavior. One might assume that the reduction in swelling due to the 5 molar $CaCl_2$ solution is approximately equivalent to the "osmotic" swelling

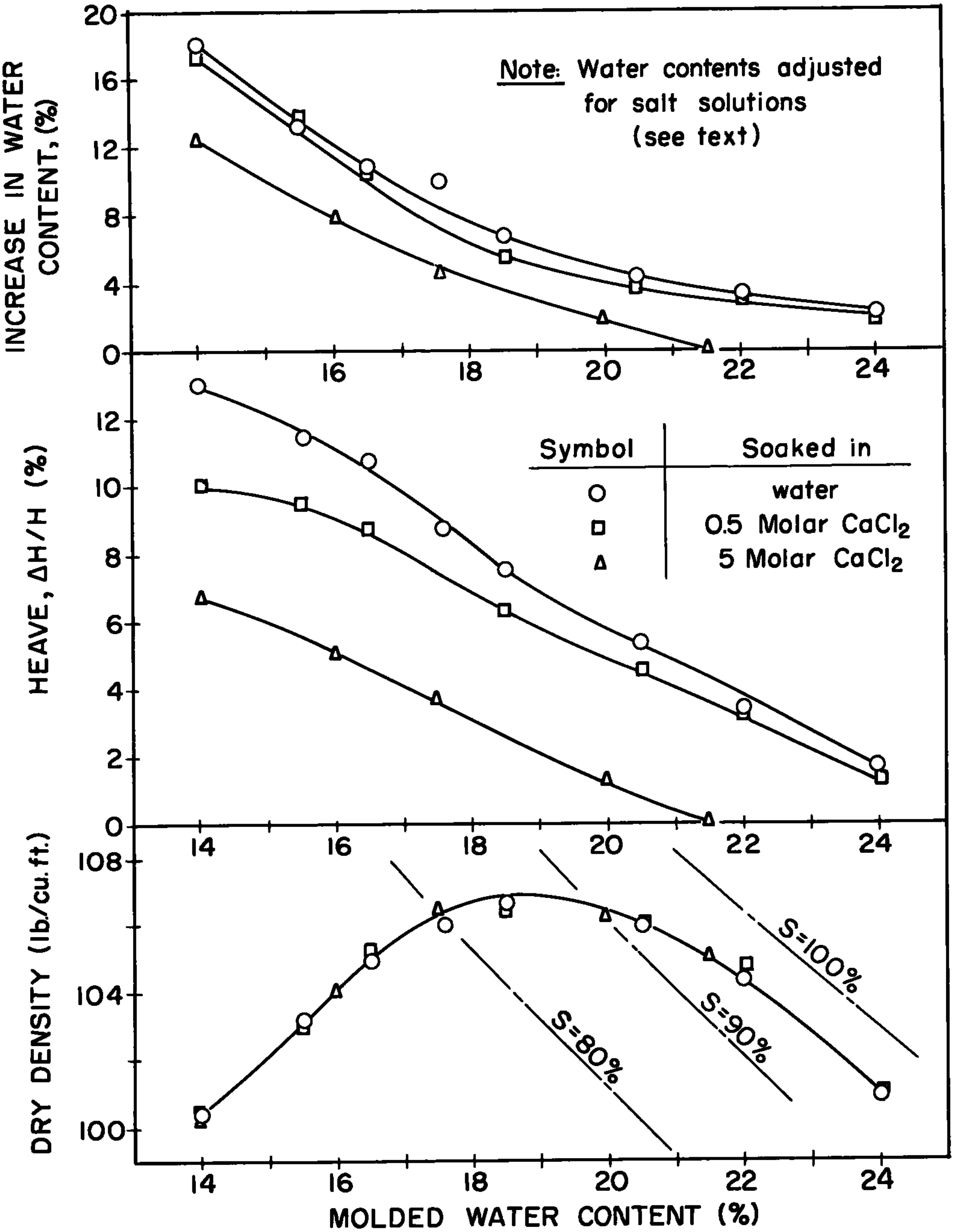

Figure 8. Effect of salt concentration on swelling behavior.

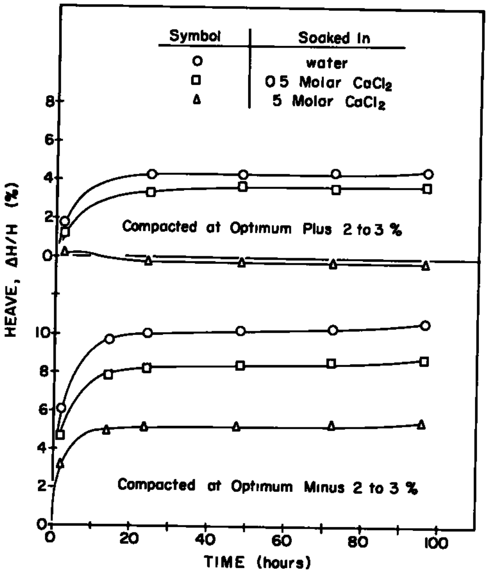

Figure 9. Effect of salt concentration on rate of swelling.

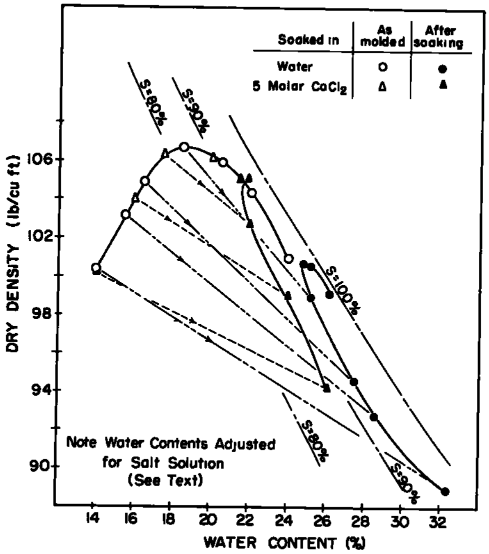

Figure 10. Effect of salt concentration on density and water content after soaking.

that occurs when the samples are soaked in pure water. However, an increase in salt concentration may also influence other interparticle forces besides osmotic pressures. A better assumption would be that the 5 molar salt solution reduces swelling due to osmotic pressures to a negligible amount, since rough calculations show that the concentration of ions in a 5 molar $CaCl_2$ solution far exceeds the concentration of the exchangeable cations between clay particles. (For a monomolecular thickness of water on the

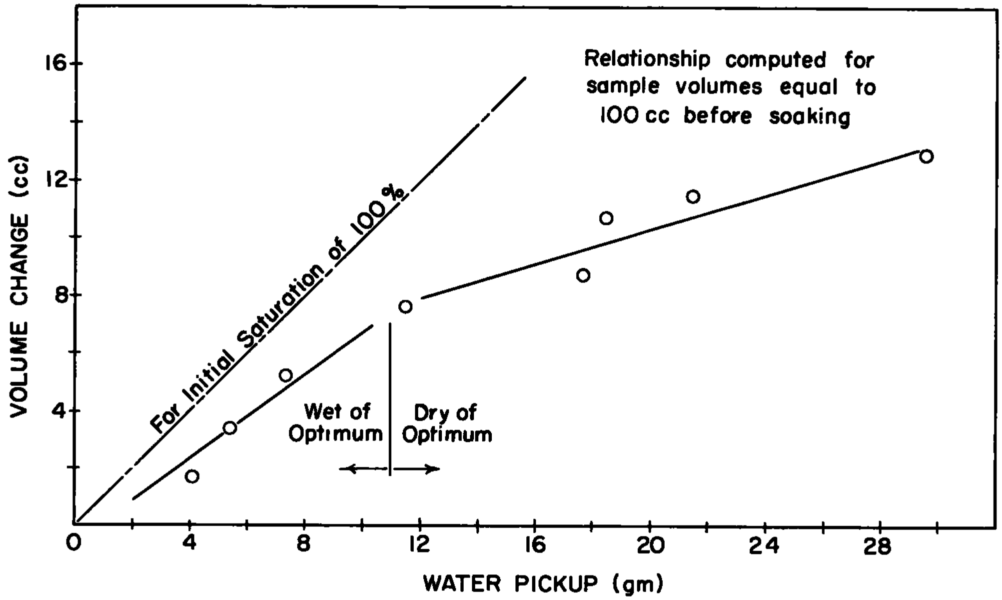

Figure 11. Relationship between volume change and water pickup for samples soaked in water.

clay, the average concentration of exchangeable cations, assumed to be calcium, around the particles is only about 2 molar. Reducing the surface area by a factor of two still only yields a 4 molar concentration. Hence the total ion concentration in a 5 molar $CaCl_2$ solution far exceeds the concentration of exchangeable cations in the soil for all water contents.)

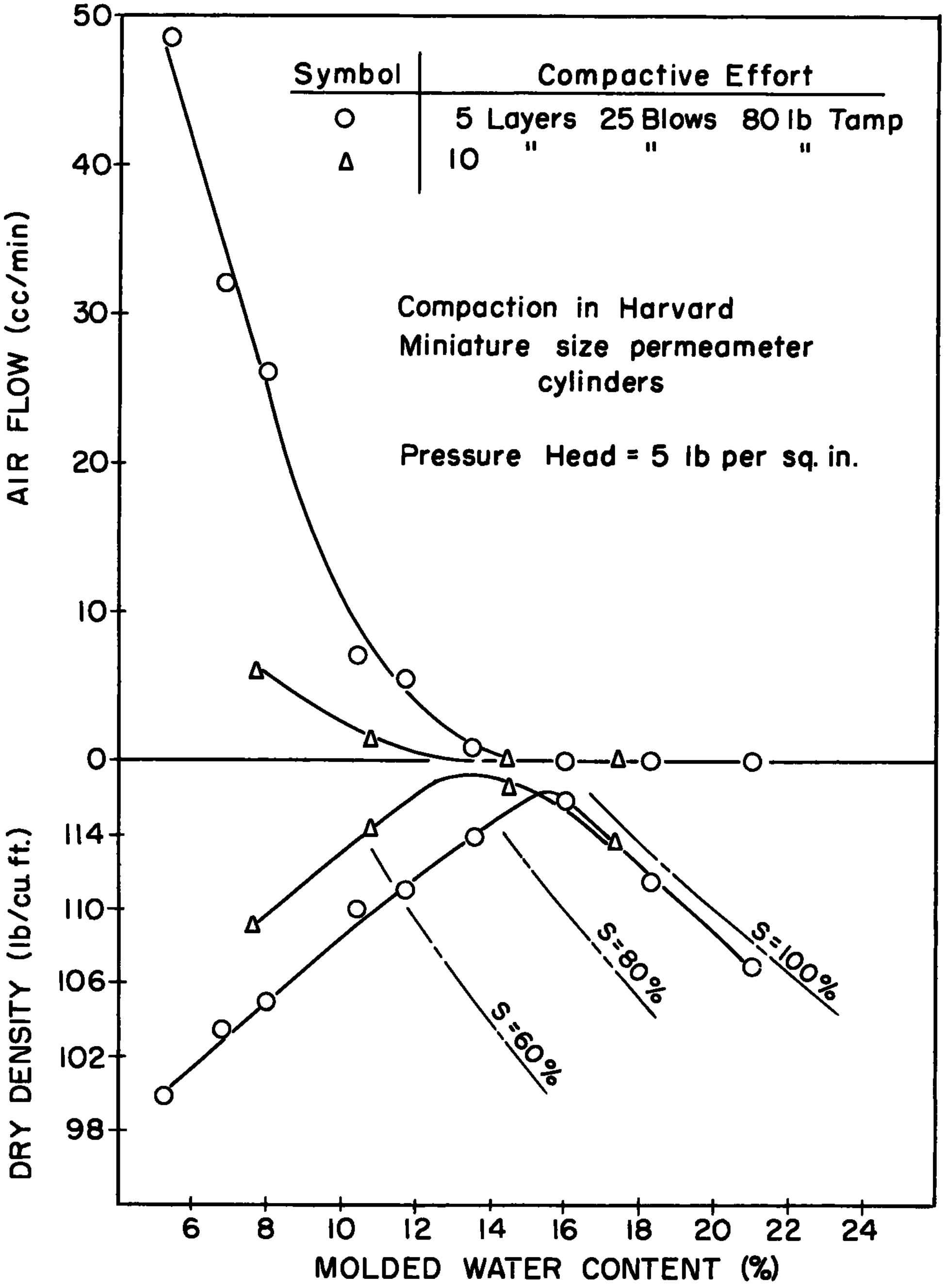

Figure 12. Air flow data.

The increased swelling dry of optimum over and above that which might be attributed to osmotic pressures may be due to a number of factors. As previously mentioned under Theoretical Considerations, there are mechanisms other than osmotic pressures by which clay particles can imbibe water. These were: (a) The effect of the negative electric and London van der Waals force fields on water; and (b) the effect of cation hydration and the attraction of the particle surface for water, at least for small double-layer thicknesses. The relative importance of the first factor is unknown; the second factor should be relatively important for the molded water content range investigated, since the molded water contents exceeded the equilibrium water content of the soil at 99 percent relative humidity. Elastic rebound and a flocculated particle orientation may contribute to the increased swelling dry of optimum; the magnitude of their influence is not definitely known. Finally, there is the role of air in swelling. The increase in volume of fluid pickup with decreasing molded water content can be partially explained by the decreasing initial degree of saturation (increasing volume of air voids). The following data suggest that a buildup of air pressures within soil voids during the soaking process may contribute to the increased swelling dry of optimum water content.

Data Relative to the Role of Air

Figure 11 has shown that the volume of air in the compacted clay samples decreases upon immersion, particularly dry of optimum water content, since the volume of water pickup exceeded the total volume increase of the samples during soaking. It seems possible that some of this decrease in the volume of air voids is caused by a compression of air by capillary forces. An increase in the air pressure (preliminary attempts to measure the pressure in the air voids after soaking proved unsuccessful) within entrapped nonspherical pockets of air, as previously discussed, is thought to cause swelling of soil.

One would expect swelling due to air pressures to be of greatest importance in compacted samples containing numerous interconnected air voids prior to soaking. With decreasing molded water content dry of optimum, the volume of air voids in the compacted clay increases. Data are presented in Figure 12 which indicate that at least some of the air in the samples compacted dry of optimum is also continuous (and therefore at atmospheric pressure), whereas the air in samples compacted wet of optimum appears to be discontinuous. Clay samples were compacted at two compactive efforts in a constant head permeability apparatus, an air pressure applied at the top of the sample, and the quantity of air flow measured with a rotameter at the outlet. For both compactive efforts the quantity of air flow decreased rapidly with increasing molded water content, and at optimum water content the flow became too small to measure (less than 1 cc per minute).

The data thus show that the samples having the greatest amount of swelling also initially have the greatest amount of interconnected air voids. While this fact supports the air pressure hypothesis, the author has no data to prove its validity. Further experimentation is planned.

Reliable data have not been obtained relative to the amount of air present in double-layer water.

CONCLUSIONS

The data have shown that the salt content in the pore fluid has an important effect on the swelling behavior of Vicksburg Buckshot clay. Furthermore, it appears that the osmotic pressure concept can satisfactorily explain a good portion of the swelling that occurs when this clay is soaked in water, particularly for samples compacted wet of optimum water content. Dry of optimum water content, however, swelling is influenced by factors in addition to osmotic pressures. These other factors may be: the effect of negative electric and London van der Waals force fields on water, cation hydration and the attraction of the particle surface for water molecules, elastic rebound, particle orientation, and the presence of air. The relative importance of these other factors is not known.

Although the experimental data are limited to one clay, the theoretical considerations indicate that many of the same concepts can be extended to other compacted soils. With other soils, however, the relative importance of the different swelling mechanisms may be altered.

An understanding of some of the factors which influence swelling may help the soil engineer predict, at least in a qualitative sense, the swelling behavior of compacted clays, and it may even enable him to alter swelling behavior to better suit his needs. This is illustrated by the following examples:

1. Fat clays, with their relatively large surface area and hence greater proportion of double-layer water, almost always swell more than lean clays.
2. Clays compacted at low water contents, where the water deficiency in the double layer is high and the degree of saturation is low, will often swell more than clays compacted at high water contents.
3. The replacement of low valency exchangeable cations by higher valency cations (for example, calcium for sodium) can reduce swelling, since the number of exchangeable cations in the double layer is reduced.
4. The mixing of salt with a compacted clay can reduce swelling, since the ion concentration in the pore water is increased. This may partially explain why the treatment of road subbases and subgrades with salt often improves stability.
5. Leaching compacted clays with salt solutions will result in less expansion and hence less strength loss than leaching with pure water. Salt leaching might be employed to increase the stability of a dam or to reduce heaving at the bottom of an excavation.

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Effect of Rate of Strain on the Strength of Compacted Soil

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This paper reports the results of a laboratory investigation of the effects of rate of strain on the strength of remolded soil. Two soils were selected for purposes of this study: (a) a clay derived from limestone and pedologically classified as Frederick, and (b) a glacial silty clay, pedologically classified as Crosby, "B" horizon. These soils were selected primarily on the basis of their difference in plasticity.

Rate of strain was considered the most important variable and it was studied from 0.55 in./min. to 1,780 in./min. The factors of moisture content and dry density were also of prime importance. Consequently, three compactive efforts were used and specimens were molded and tested on both sides of the optimum moisture content of each compactive effort.

The unconfined compressive strength test was used as the strength criterion and the effect of the aforementioned variables upon the ultimate strength and modulus of deformation of the samples, as determined by this test, are reported.

● THE FACT that the compressive strength is a function of the time required to reach the failure load has long been recognized. However, this area of soil mechanics has not been extensively explored and much work remains to be done, in order that the effects of this phenomenon can be properly evaluated. The specific areas where this information would be of the greatest benefit are as follows:

1. Stability of slopes subjected to earthquakes and other forms of transient loading.
2. Transmission of forces from explosions through soils.
3. Design of highway pavements.
4. Design of airfield pavements.

STABILITY OF SLOPES

In areas where there is a possibility of earthquakes, it is of the utmost importance to investigate the stability of slopes, both natural and man-made, under transient conditions. Such an investigation is especially necessary when failure of the slope in question would be disastrous. Earthquake shocks induced in the earth represent transient loading conditions and critical slopes should be designed and analyzed on this basis.

TRANSMISSION OF FORCES FROM EXPLOSIONS

The transmission of forces from explosions through soil, due to the short time of loading, is another example of where structures subject to such forces should be designed and analyzed on the basis of transient loading tests. Such a rigorous study would be valid only for critical military installations.

DESIGN OF HIGHWAY PAVEMENTS

It is generally recognized that the stress-strain characteristics of pavements are a function of the rate of strain. This can be readily seen by observing the condition of pavements at critical sections along a given route.

From such studies, it has been found that road intersections, uphill grades and other sections where traffic is required to move slowly, or to stop, show distress much more rapidly than their counterparts; that is, open road, free of stops, and downhill grades. This is believed to be caused, in part, by the difference in speed of travel over the aforementioned sections. As a result, there is much need of a comprehensive study on the effect of rate of strain on the behavior of soils.

DESIGN OF AIRFIELD PAVEMENTS

Because of the high speed at which airplanes travel over runways, the latter are subjected to transient loading conditions which are vastly different from the relatively static conditions to which the aprons, taxiways, and ends of runways are subjected. Hence, it is desirable to evaluate the stress-strain characteristics of the paving materials under both transient and static loading. Such a procedure would lead to the most economical as well as the best design.

PURPOSE AND SCOPE

The previous discussion points out that the effect of rate of strain on the strength of soil is important. Consequently, the primary purpose of the research reported herein was to investigate the strength properties of a clay and silty clay under conditions of transient loading. Specifically the aim was to ascertain the relationship between rate of strain and unconfined compressive strength at various moisture contents and densities. Also, it was hoped to relate the aforementioned variables to the modulus of deformation.

Rate of strain was considered the most important variable, and it was studied from 0.55 in./min to 1,780 in./min. Soil texture was a second variable and two fine-grained soils of significantly different characteristics were chosen—a silty clay and a clay. All soils are native to Indiana.

The factors of moisture content and dry density were also of prime importance. Three compactive efforts were used and specimens were molded and tested on both sides of the optimum moisture content (O.M.C.), of each compactive effort.

DEFINITIONS

It is assumed that the reader is familiar with terms pertaining to conventional strength tests on soils. However, before proceeding further he should familiarize himself with the following terms:

1. A Slow Transient Compression Test is one in which the rate of strain lies within the range of 0.45 in./min to 0.6 in./min.
2. A Medium Transient Compression Test is one in which the rate of strain lies between 11.0 in./min and 15.5 in./min.
3. A Fast Transient Compression Test is one in which the rate of strain is greater than 250 in./min.
4. Time of Loading is defined as the difference in time between the start of a loading test and the time at which the maximum compressive stress is reached.
5. Compressive Stress is the axial load per unit of cross-sectional area of a test specimen. In computing the compressive stress, a correction was applied to the area assuming that no volume change took place; that is, change in area assumed proportional to the change in height.
6. Modulus of Deformation is a secant modulus defined as the slope of a line from the origin through the point on the stress vs strain curve at which the stress is one-half of the compressive strength. Or, if the initial part of the stress vs strain curve is straight, it is the slope of this portion of the curve.
7. Rate of Strain is defined as the deformation at failure divided by the time required to reach failure.
8. Strength Ratio, S_u, is defined as the ratio of the strength, for a given rate of strain, to that for a slow transient test at the same moisture content and for the same compactive effort.

9. Modulus of Deformation Ratio, M_D, is defined as the ratio of the modulus of deformation for a given rate of strain to that for a slow transient test at the same moisture content and for the same compactive effort.

10. Strength, in this report will always apply to the axial load required to produce failure in an unconfined compression test.

It should be noted that the above definitions may be at variance with those by other investigators, but were adopted for this study because they give the best representation of the data.

REVIEW OF LITERATURE

The effect of time of loading on the strength and modulus of elasticity of metal and concrete has been extensively investigated. Also, a considerable amount of work has been done on rock and wood, but little on soil.

The standard methods of measuring the time effect, in previous investigations, was either by the time of loading, the rate of loading, or the rate of strain. In general, the method utilized was determined by the characteristics of the testing machines; that is, with the equipment available which of the aforementioned methods of measurement can be used most accurately and efficiently. Nevertheless, whenever the time-stress-strain relationships are known, any one of the above ways of measuring the time effect may be converted to either of the other two.

SOIL

In connection with the design of the Panama Canal, several triaxial compression tests were made to determine the effect of time of loading on the compressive strength of Gatun black muck (14). In one test, with a time of loading of approximately 90 sec, the compressive strength was about 40 percent greater than in four other tests, in which failure was obtained in about 1 hr.

A series of triaxial compression tests, on remolded-consolidated Boston blue clay, was reported by Taylor. The rate of strain ranged between 1 and 0.0005 percent per min and the compressive strength for the fastest test was approximately 20 percent greater than that for the slowest test (1).

Casagrande and Shannon (1) found that the modulus of deformation of clays for fast transient tests was approximately twice as great (for both unconfined and consolidated quick tests) as that for slow tests. Also, the modulus of deformation of Manchester sand increased slightly with decreasing time of loading; that is, the average value for static tests is about 300 kg per sq cm and for fast transient tests about 400 kg per sq cm.

Furthermore, for all tests on clays the transient compressive strength was greater than the static compressive strength. The fast transient compressive strength, taken at a time of loading of 0.02 seconds, ranges between 1.4 and 2.6 times greater than the 10-minute static compressive strength. The above holds true regardless of whether the clay is undisturbed or remolded.

Tests on the Manchester sand indicate that the transient compressive strength of sand in a fast transient compression test was only about 15 percent greater than the static compressive strength.

Investigations by Seed and Lundgren (15), on the effect of transient loading on the strength of saturated sands supports the above. It was found that the strength of dense specimens in rapid transient tests was 15 to 20 percent greater than the strengths of similar specimens in static or slow transient tests.

Whitman (23), also, found that the strength of both undisturbed and disturbed soil specimens was significantly increased by transient loading. This work was done over a wide range of cohesive soils and three sands—the latter ranging from uniform Ottawa sand to well-graded sands.

On the basis of the data collected Whitman concluded that the effect of rate of strain on cohesive soils should be evaluated from triaxial tests with confining pressures adequate to prevent splitting or shear plane development prior to the occurence of the peak load. Considering dry sand and moist sand, it was found that increasing the rate of strain did not produce a significant change in the strength of the material. However, the converse was true for saturated sand and Whitman attributed this to differences in pore water pressures in the slow and rapid tests.

DESCRIPTION OF MATERIALS

Two soils were selected for purposes of this study: (a) a red-colored clay derived from limestone, and (b) a brown glacial silty clay, pedologically classified as Crosby, "B" horizon; both soils are native to Indiana. These soils were selected primarily on the basis of their difference in plasticity (Table 1).

TABLE 1

SUMMARY OF ATTERBERG LIMTS

Soil	LL	PL	PI	Classification HRB	Classification Unified
Silty Clay	35.4	19.5	15.9	A-6	CL
Clay	72.0	28.1	48.9	A-7-6	CH

Both soils were aid dried, pulverized, and passed through a No. 40 U.S. standard sieve. It is recognized that a soil containing clay is irreversibly altered by drying, particularly if the drying is allowed to reach the shrinkage limit. Thus, the laboratory behavior of these soils is peculiar to their processed condition and may differ significantly from the in situ behavior.

PROCEDURE

Moisture-Density Relationships

The only relationships studied were those for kneading type compaction with three compactive efforts being used. There is no recognized standard procedure for the equipment used, for the Harvard Miniature Compaction apparatus; however, the compaction procedure recommended (22) was followed.

It was hoped to obtain densities which approximated the Standard AASHO and Modified AASHO compaction tests as well as a density intermediate between the two. The moisture vs density curves are shown in Figures 2 and 3. Also as a comparison between the dynamic and the kneading compaction characteristics of these soils Table 2 is presented. The data presented in Table 2 on kneading compaction represents the average values obtained from the three moisture density curves in Figures 2 and 3.

The Harvard Compaction Apparatus produces specimens 2.816 in. long and 1$^5/_{16}$ in. in diameter. The volume of the compaction cylinder is $^1/_{454}$th of a cu ft which means that the weight of a compacted specimen in grams is numerically equal to its unit weight

TABLE 2

COMPARISON OF KNEADING AND DYNAMIC COMPACTION (16)

Soil	Compactive Effort	Max (pcf)	O.M.C. (%)
Silty-Clay	Standard AASHO	107.5	19.2
	Kneading Compactor (3 layers at 25 blows)	109.5	16.3
	Modified AASHO	119.3	13.7
	Kneading Compactor (10 layers at 50 blows)	117.0	13.8
Clay	Standard AASHO	91.7	27.8
	Kneading Compactor (3 layers at 25 blows)	94.8	26.0
	Modified AASHO	106.7	19.5
	Kneading Compactor (10 layers at 50 blows)	102.5	22.7

in pounds per cubic foot (pcf). The samples so obtained were used to establish the moisture density curves as well as for strength tests.

The compactive efforts used were 10 layers at 50 blows per layer, which approximates the Modified AASHO, and three layers at 25 blows per layer which gives dry densities comparable to the Standard AASHO test. The third compactive effort varied depending on the soil being tested. The latter was necessary in order to obtain significant differences in the dry densities obtained from the three compactive efforts. For the Crosby "B" soil the intermediate compactive effort was ten layers at 25 blows per layer while that for the Frederick Limestone soil was ten layers at ten blows per layer.

Unconfined Compression Tests

The unconfined compressive strength tests were run on either a hydraulic loading apparatus or the impact loading apparatus depending upon the rate of strain desired. The dynamic loading apparatus consists of a 10-lb weight dropped 39 in. upon a piston which applied the load to the specimen. In order to prevent damage to the loading frame and measuring instruments, and to slow down the rate of loading, a spring was inserted atop the loading piston. Another advantage was that the elasticity of the spring caused the weight to bounce, whereupon it was caught before it could again come in contact with the piston.

Collection of Data and Instrumentation

Low voltage differential transformers were employed to measure the load as well as the deformation during the compression process. In this type gage the output voltage of the secondary coils which are excited by a primary coil is proportional to the displacement of a magnetic core within these coils.

A schematic diagram of the load and deformation apparatus is shown in Figure 1. It can be seen that as the proving ring deforms, the position of the core moves relative to the coils. Also, as the arm of the strain gage deflects, the core contained therein moves relative to its primary and secondary coils. These movements produce changes in the output voltage, of these two instruments, which can be measured. A Model 655 Audio Oscillator operating on a frequency of 2, 000 cps was used to energize the transformers.

To record the changes in stress and strain during the progress of the test a Dumont Cathode-Ray Oscilloscope was used. In this study, the deflection of the oscilloscope trace was made proportional to the output of the differential transformers.

RESULTS

Strength

In both the clay and the silty clay, the strength vs moisture content curves all exhibited a "peaked" shape, as indicated in Figures 2 and 3. Theoretically the strength should continue to rise as the moisture content decreased. However, there was a tendency for a reduction in this "peaked" condition as the rate of strain increased.

There are two possible reasons for the aforementioned: (a) the condition is the result of the inherent characteristics of the test; that is, at low moisture contents the specimens fail by crumbling rather than shear, due to a lack of lateral confinement, or (b) the condition is a result of the molding process; that is, kneading type compaction imparts this characteristic to the soil.

At the lower moisture contents tested, 9-10 percent for the silty clay and 14-15 percent for the clay, the specimens were brittle and would crumble around the edges if not handled carefully. There is great possibility that this tendency existed at higher moisture contents but was just more difficult to detect.

In all cases the maximum strength, as indicated from these tests, occurred at a moisture content less than optimum (Figs. 2 and 3). Exactly how much seemed to depend upon the rate of strain and the compactive effort; that is, as the compactive effort decreased there was a tendency for the difference between O. M. C. and the moisture content at which maximum strength occurred to increase. The maximum difference

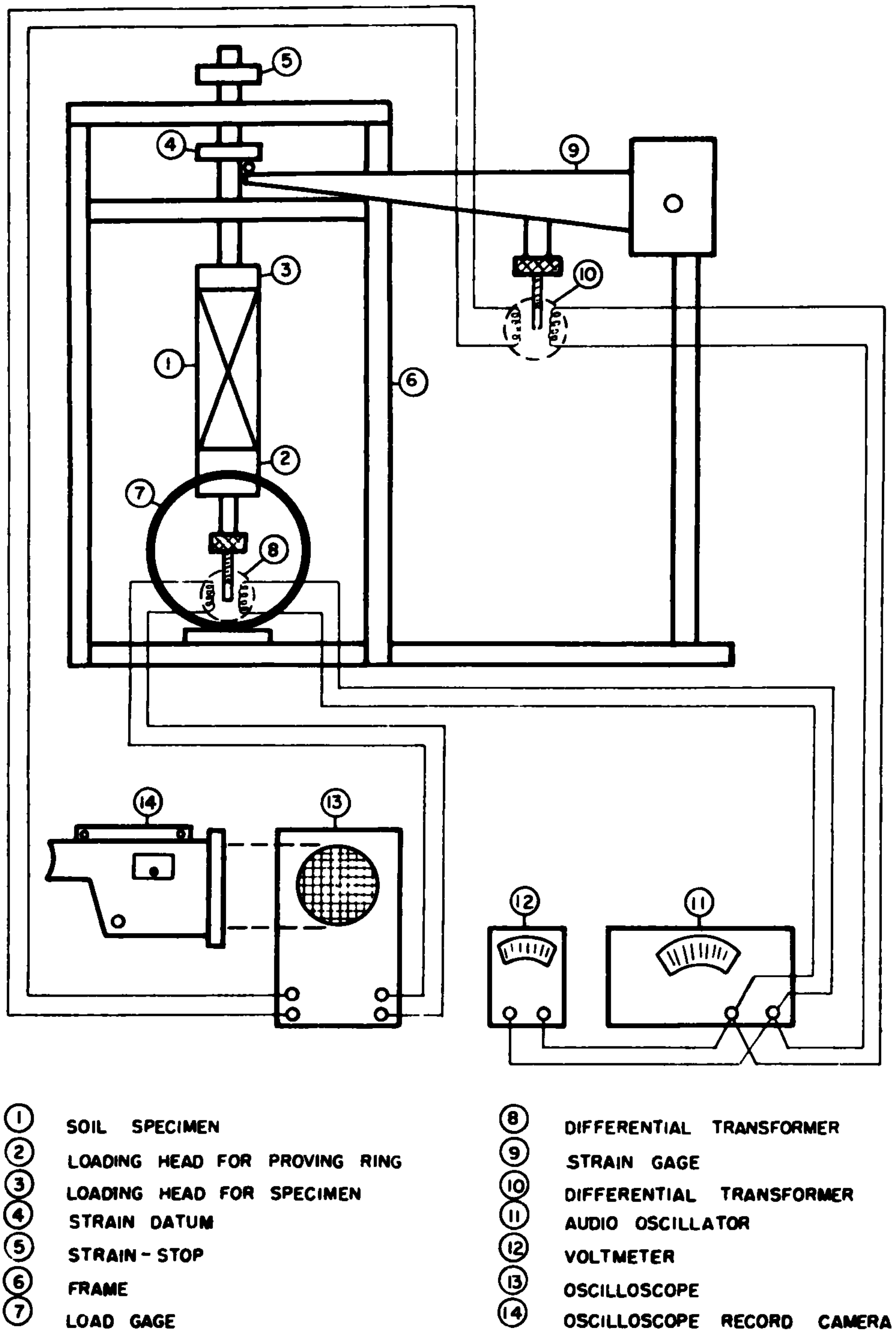

Figure 1. Schematic diagram of test set-up.

between the latter two was approximately 4 percent in the silty clay and 5 percent in the clay.

Once the moisture content at which the optimum strength was obtained was passed, the change in strength for a given change in moisture content increased significantly (Figs. 2 and 3). It appears that the greater the rate of strain the steeper this portion of the curve becomes.

The foregoing is true for both the silty clay and clay. However, in the clay, the rate of increase of strength for a given increase in moisture content on the dry side of optimum strength was equal and in some instances greater than the rate of decrease which occurred on the wet side. Whether or not this is significant depends on the validity of the tests conducted on the dry side; that is, whether or not the specimens were failing principally by crumbling or by shear.

Silty Clay

It was found that, for a given compactive effort and moisture content, increasing the rate of strain produced a measurable increase in the strength of the soils (Figs. 4 and 6). In the case of the silty clay, for the lowest compactive effort, an increase of over 100 percent in the unconfined compressive strength was obtained by increasing the

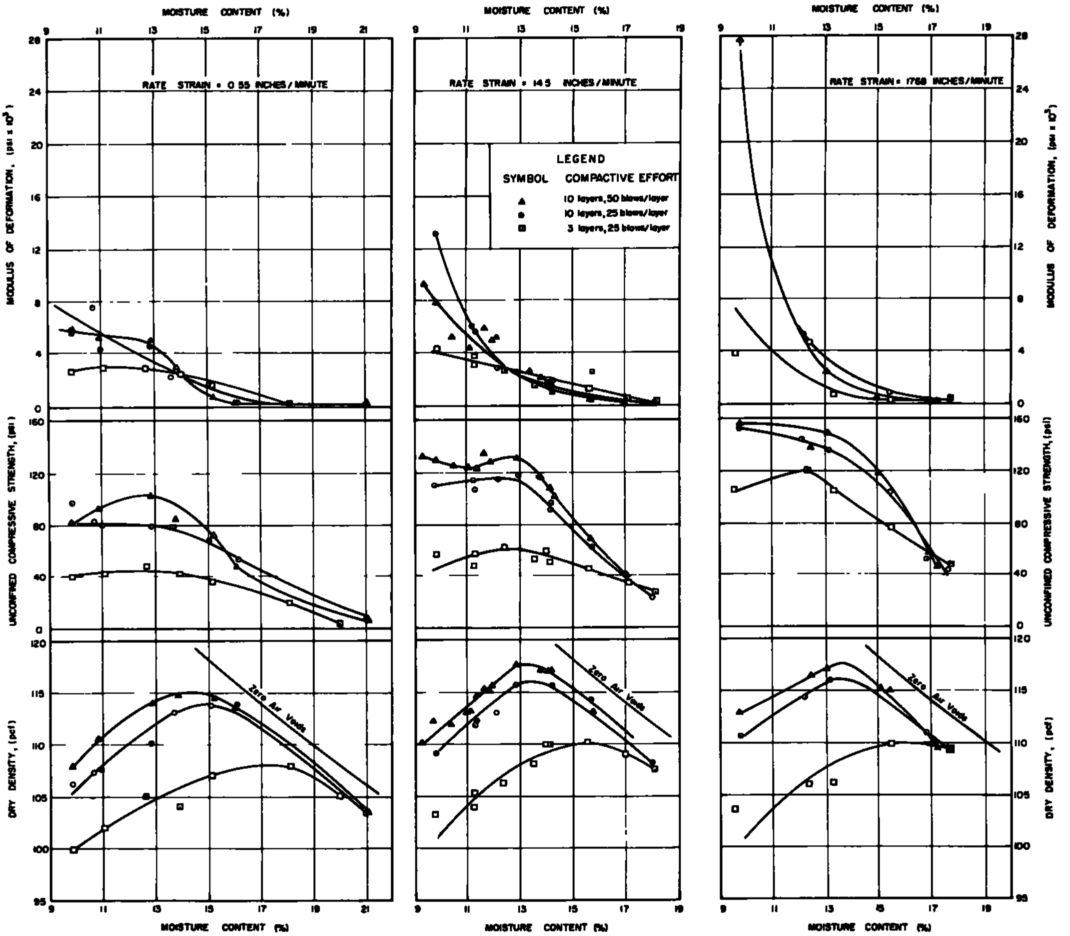

Figure 2. Moisture content vs dry density, unconfined compressive strength, and modulus of deformation—silty clay.

rate of strain from 0.55 in./min to 1,768 in./min, regardless of the moisture content of the sample (Table 3). For the highest compactive effort the increase in strength for the same change in rate of strain ranged between 52 and 87 percent while for the intermediate compactive effort the range was from 20 to 89 percent.

As regards the change in strength of the medium transient test with respect to the slow transient test, for a given density and compactive effort, the increase in strength was much less (Table 3). Considering the lowest compactive effort the increase in strength ranged from 12 to 32 percent, for the intermediate compactive effort the change ranged from -17 to 40 percent, and for the highest compactive effort the change in strength was from +10 to +51 percent.

There appears, on the basis of the data collected, to be a general trend toward a decrease in the effect of rate of strain as the moisture content increases, for a given

TABLE 3

SUMMARY OF RESULTS OF STRENGTH TESTS—SILTY CLAY

	Compactive Effort					
Rate Strain	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	Moisture Content = 10%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	43	1.00	81	1.00	85	1.00
14.5 in./min.	48	1.12	110	1.36	128	1.51
1768 in./min.	109	2.54	153	1.89	159	1.87
	Moisture Content = 12%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	46	1.00	83	1.00	103	1.00
14.5 in./min.	61	1.33	116	1.40	130	1.26
1768 in./min.	119	2.59	146	1.76	156	1.52
	Moisture Content = 14%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	43	1.00	77	1.00	91	1.00
14.5 in./min.	57	1.32	96	1.25	110	1.21
1768 in./min.	95	2.21	127	1.65	140	1.54
	Moisture Content = 16%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	34	1.00	57	1.00	51	1.00
14.5 in./min.	43	1.26	56	0.98	62	1.22
1768 in./min.	71	2.09	88	1.54	93	1.82
	Moisture Content = 17%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	28	1.00	46	1.00	38	1.00
14.5 in./min.	35	1.25	38	0.83	42	1.10
1768 in./min.	57	2.04	55	1.20	58	1.53

compactive effort (Fig. 4 and Table 3). This, in all probability, is due to the greater effect of pore water pressures which would tend to decrease the strength of the sample. This could also be the reason why the strength ratio values tend to decrease as the compactive effort increases.

From the foregoing discussion, it can be seen that the effect of rate of strain is more significant in increasing the strength of material at low densities. In comparing the fast and slow transient tests, over 100 percent increase in strength was obtained for the lowest compactive effort. Considering the same range for the other two compactive efforts the maximum increase was 89 percent, and for the majority of cases it was much less.

Furthermore, as the compactive effort increased the effect of an increase in moisture, on the strength, increased for a given compactive effort (Fig. 4). As aforemen-

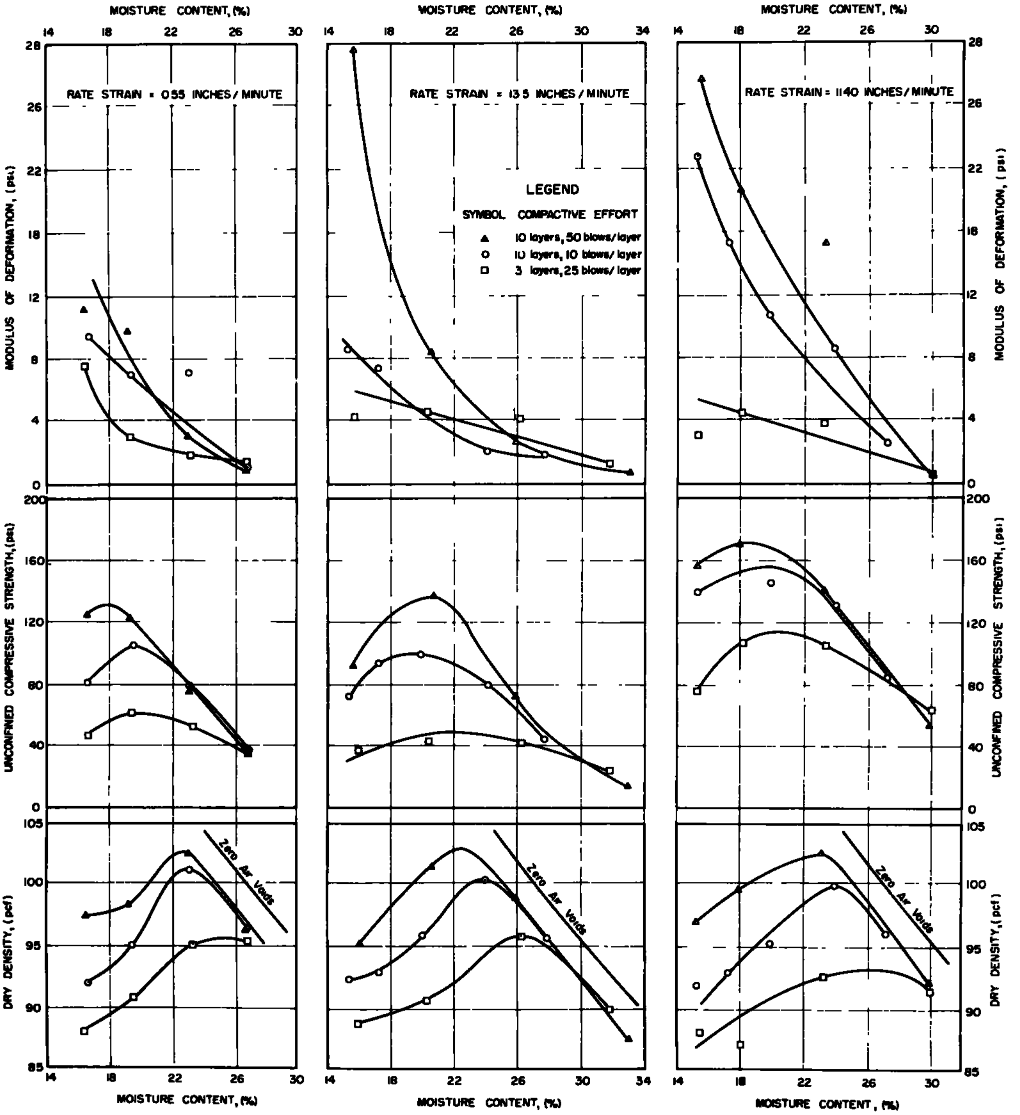

Figure 3. Moisture content vs dry density, unconfined compressive strength, and modulus of deformation—clay.

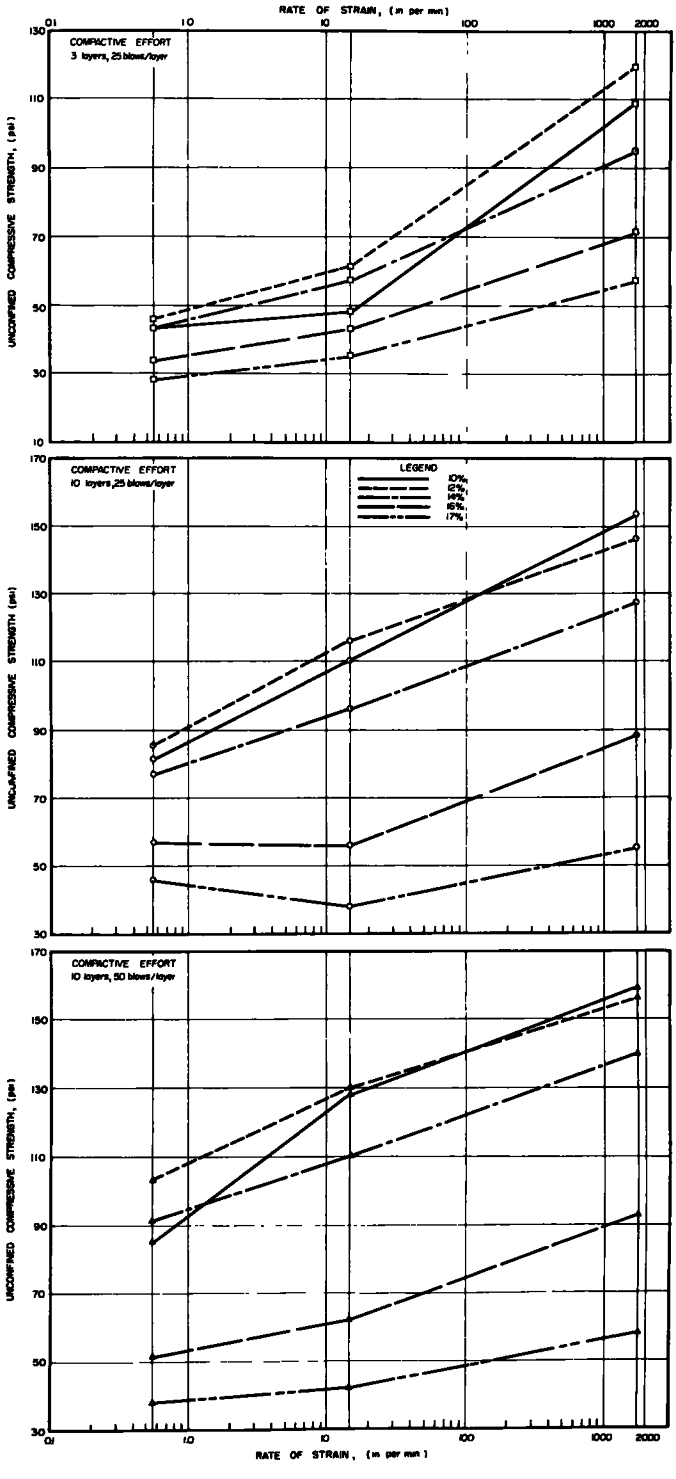

Figure 4. Rate of strain vs unconfined compressive strength—silty clay.

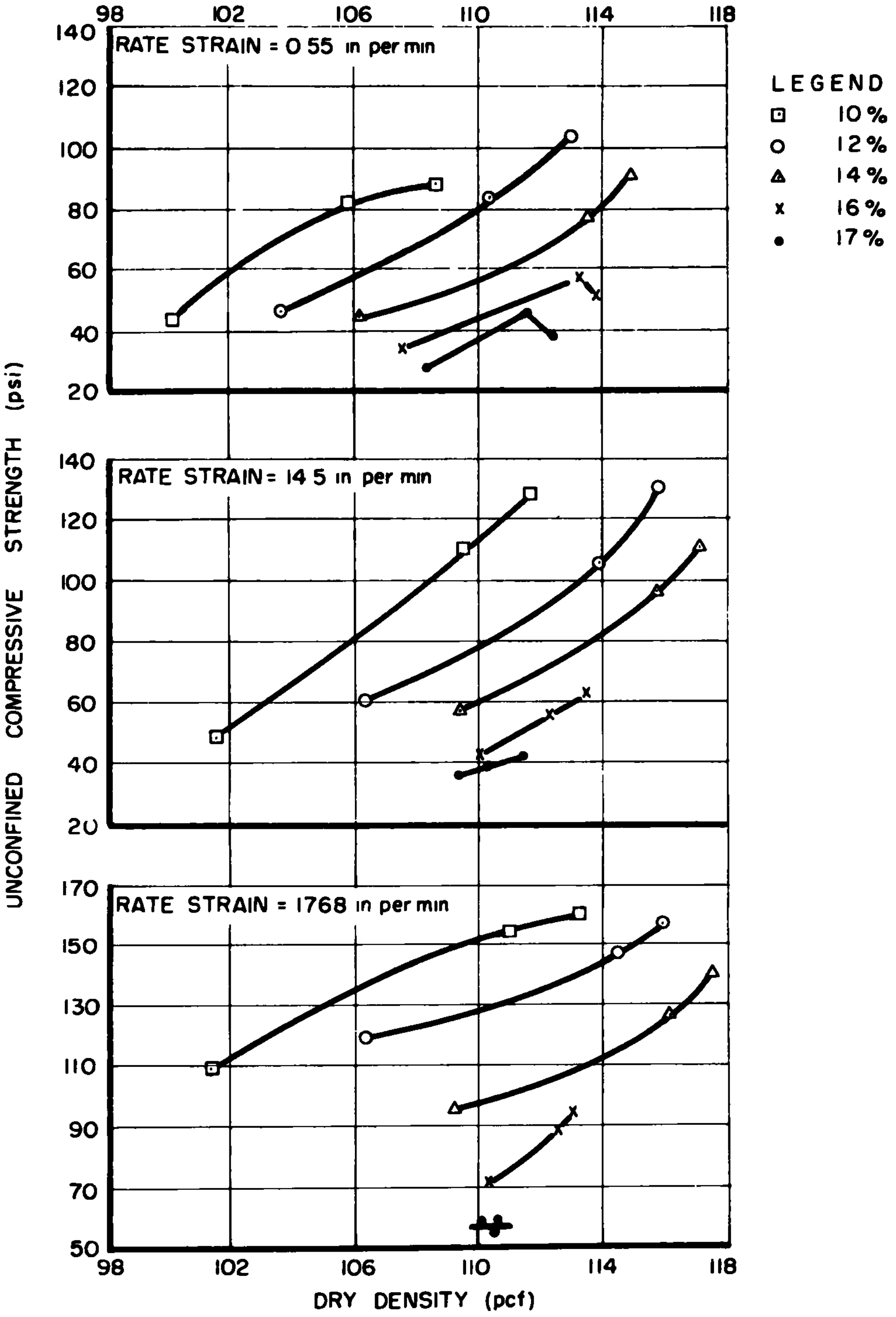

Figure 5. Dry density vs unconfined compressive strength—silty clay.

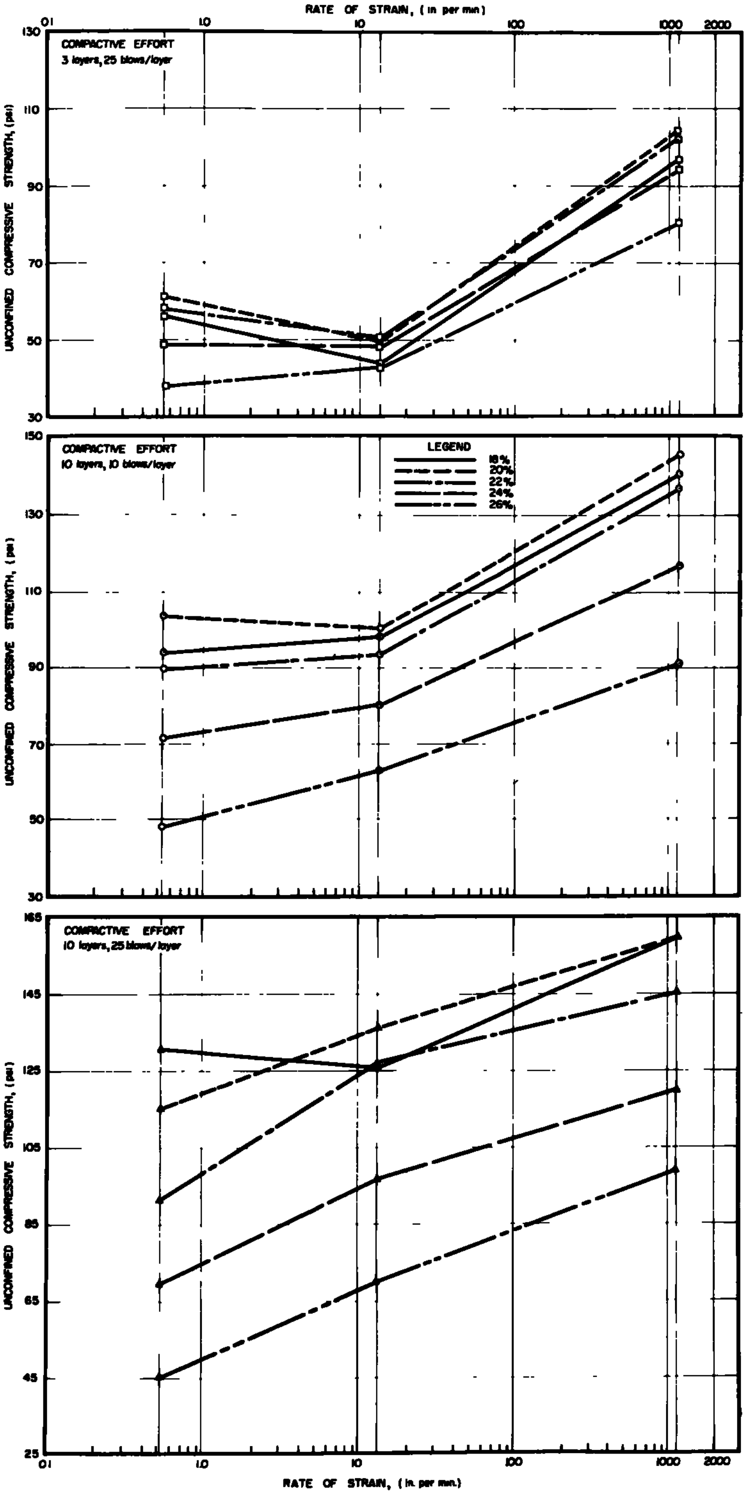

Figure 6. Rate of strain vs unconfined compressive strength—clay.

tioned, this is due to an increase in pore pressure brought about by a reduction in void ratio due to increased compaction.

From Figure 5, which is a plot of unconfined compressive strength vs dry density, at constant moisture content, it can be qualitatively stated that the effect of increased density decreases as rate of strain increases. For example, it can be noted that the slopes of the curves for rate of strain equal to 1, 768 in. per min are less than those for either 0.55 or 14.5 in. per min. Also, moisture content did not effect the slopes appreciably.

Clay

For the clay, it was also found that for a given compactive effort and moisture content, increasing the rate of strain produced a measurable increase in the strength of

TABLE 4

SUMMARY OF RESULTS OF STRENGTH TESTS—CLAY

	Compactive Effort					
Rate Strain	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	Moisture Content = 18%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	56	1.00	94	1.00	131	1.00
13.5 in./min.	44	0.79	98	1.04	126	0.96
1140 in./min.	97	1.73	142	1.51	160	1.22
	Moisture Content = 20%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	61	1.00	103	1.00	115	1.00
13.5 in./min.	49	0.80	100	0.97	136	1.18
1140 in./min.	104	1.71	146	1.42	160	1.39
	Moisture Content = 22%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	58	1.00	89	1.00	93	1.00
13.5 in./min.	50	0.86	93	1.04	128	1.36
1140 in./min.	102	1.76	137	1.54	146	1.57
	Moisture Content = 24%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	49	1.00	71	1.00	69	1.00
13.5 in./min.	48	0.98	80	1.13	98	1.42
1140 in./min.	92	1.88	117	1.65	120	1.74
	Moisture Content = 26%					
	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio	Strength (psi)	S_u Ratio
0.55 in./min.	38	1.00	48	1.00	45	1.00
13.5 in./min.	43	1.13	63	1.31	70	1.56
1140 in./min.	80	2.11	91	1.90	95	2.11

the soil (Fig. 6). As with the silty clay, the strength ratio, for all moisture contents, was greatest for the lowest compactive effort (Table 4). Also, from Figure 6 it can be seen that moisture content has less effect on the strength, at a given rate of strain, the lower the compactive effort; that is, as the compactive effort is increased the effect of moisture content increases.

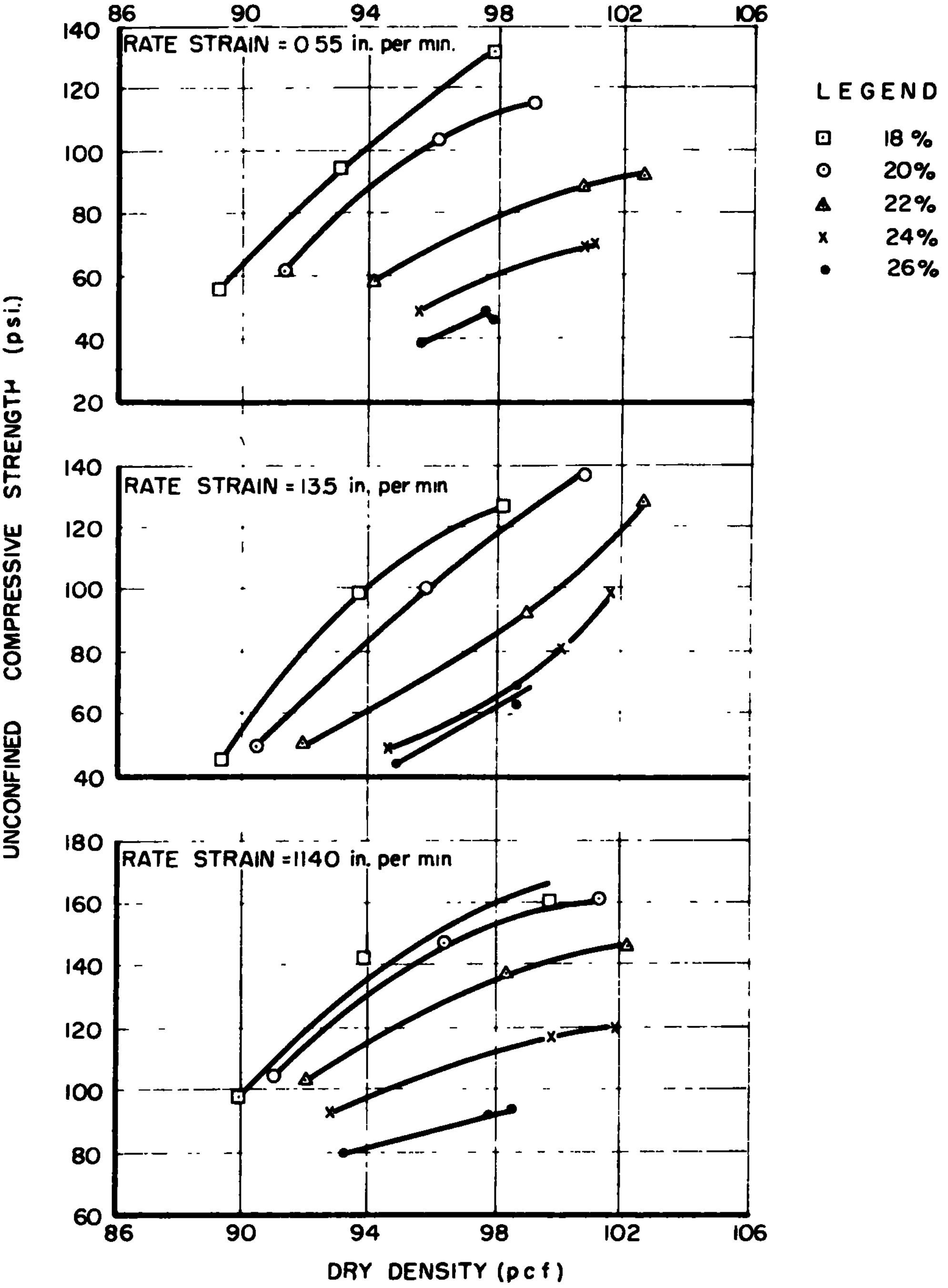

Figure 7. Dry density vs unconfined compressive strength—clay.

Table 4 shows that as the moisture content increases the effect of rate of strain on increasing the strength increases for a given compactive effort. However, it should be noted that this relationship is generally valid for only the dry side of optimum since the data on the wet side are incomplete.

Table 4 also shows that some clay soils may decrease in strength with an increase in rate of strain. This tendency existed at all except one moisture content, the highest, at the lowest compactive effort. In the two higher compactive efforts the condition existed at only one moisture content, in each. Since the latter values are so close to unity, it could be assumed that there was not a decrease and the stress ratio is unity.

However, due to the fact that the stress ratios for the medium transient tests are consistently less than 1.0 this might be a significant trend, for the lowest compactive effort. It can also be noted that a value of the stress ratio greater than 1.0 occurs at nearly the optimum moisture content. This indicates that on the wet side, stress ratios greater than 1.0 would be obtained. Also, it must be recognized that since the stress ratio for the medium transient tests are generally very close to unity, at low moisture contents and low compactive efforts, and since an average curve is drawn through the data obtained this would account for the stress ratio being less than 1.0.

A study of Figure 6 reveals that the higher the compactive effort the greater the effect of a change in moisture content on the strength of the clay soil. For example, as the compactive effort is increased the curves in Figure 6 become farther apart.

It should be noted that the 18 percent moisture content curve in Figure 6 is out of line with the remainder of the data. This is probably because of specimens tested at this moisture content failed, partially at least, by crumbling.

Figure 7 illustrates, qualitatively, that the effect of an increase in density, decreases as the rate of strain increases. This can be noted from the fact that the slopes of the curves for rate of strain equal to 1,140 in. per min are less than those for either 0.55 or 13.5 in. per min.

It is worthy of note that the strength ratio for the clay increases with increasing moisture content while that for the silty clay decreases with increasing moisture content. Also, for a given rate of strain the effect of moisture content on the strength is greater for the silty clay than the clay. However, the latter trend decreases as the compactive effort decreases.

Modulus of Deformation

The data on the modulus of deformation, for both soils tested, was the most difficult to interpret. This was due to the large scattering of the values (Figs. 2 and 3). The latter situation being caused by the difficulty in picking values from the oscilloscope trace. It was quite easy to determine the maximum load, on the specimen, and the strain at which it occurred. However, at low loads it was, in many instances, impossible to accurately determine the load on the specimen and the strain at which it occurred. As a result, in practically all cases, the initial portions of all stress strain curves appeared to be out of line.

Consequently, the modulus of deformation vs moisture content curves, for a given compactive effort and rate of strain can only be used to obtain general trends as indicated by the results shown (Figs. 2 and 3). Since the modulus of deformation vs rate of strain curves (Figs. 8 and 9), for a given moisture content and compactive effort, were obtained from the curves previously indicated they, also, are only of qualitative value.

Silty Clay

Regardless of the scattering of the points in Figure 2 it can be readily seen that moisture content has a tremendous effect on the modulus of deformation of the silty clay. It is also apparent that as the rate of strain increases the effect of moisture on the modulus of deformation also increases.

This rate of change of modulus of deformation with respect to a change in moisture content decreases as the moisture content increases until a point is reached where the compactive effort has a negligible effect on the modulus of deformation. This condition

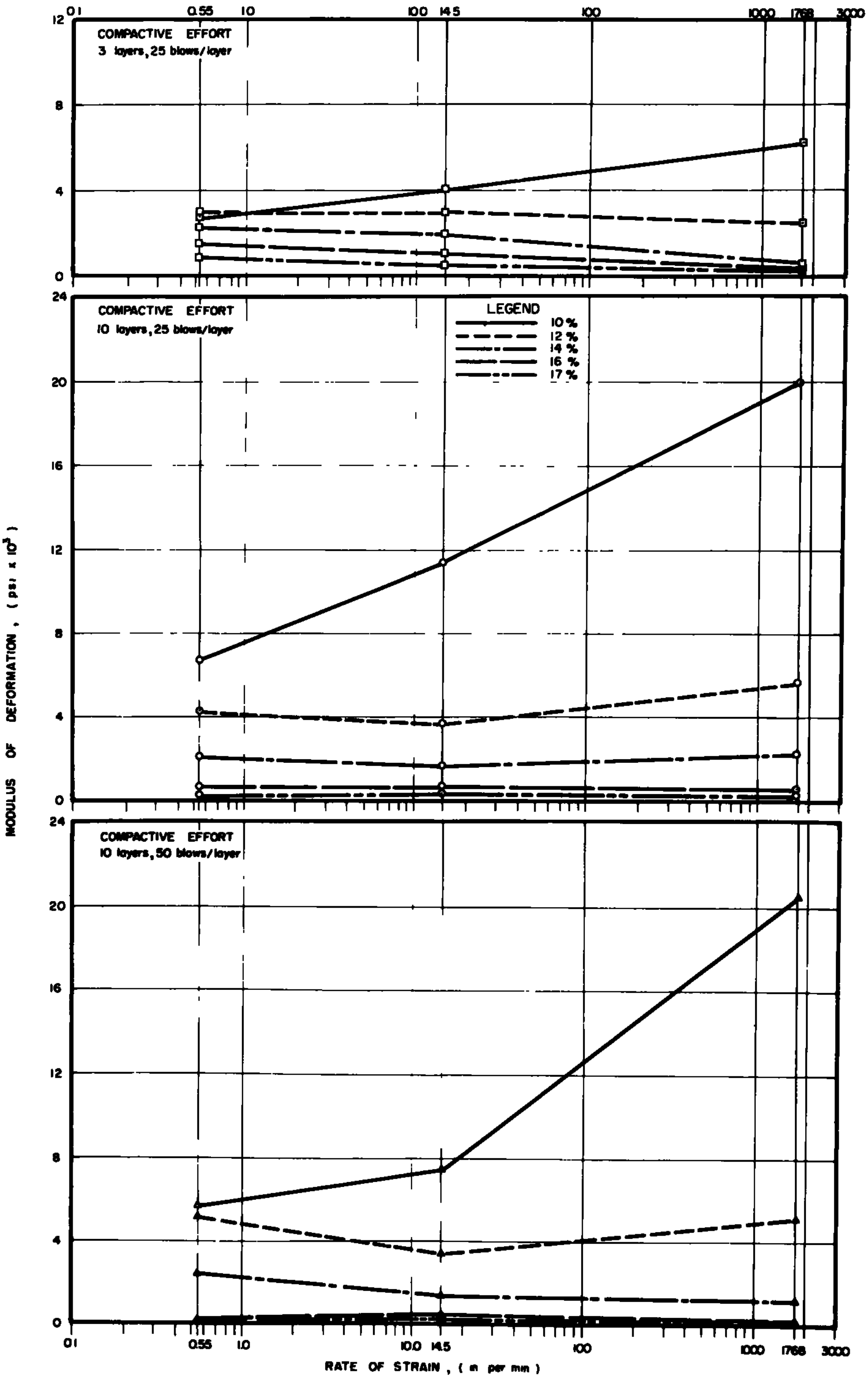

Figure 8. Rate of strain vs modulus of deformation—silty clay.

occurs on the dry side of optimum for the slow and medium transient tests, for all compactive efforts, and approximately at optimum for the fast transient tests.

In fact, if a soil is to be compacted at a moisture content which approximates the O.M.C. for standard compaction, considering the fast transient test conditions, it appears that use of a compactive effort greater than Standard Proctor is not warranted. Considering the fact that a subgrade usually gains approximately 2 percent moisture before coming to equilibrium with its natural surroundings, this would make compaction at an effort greater than standard AASHO impractical, unless methods of preventing an increase in moisture content were provided. The latter statement is assuming that the modulus of deformation is to be used as the criterion of strength.

Considering the moisture content range tested it appears a significant increase in modulus of deformation due to an increase in rate of strain was obtained only for the specimens compacted at about 10 percent moisture (Fig. 8). This is true regardless of the compactive effort. Also, on the basis of Figure 8 it can be easily seen that moisture content has a much more significant effect on the modulus of deformation than does rate of strain.

TABLE 5

SUMMARY OF MODULUS OF DEFORMATION DATA—SILTY CLAY

	Compactive Effort					
Rate Strain	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	Moisture Content = 10%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	2,700	1.00	6,700	1.00	5,700	1.00
14.5 in./min.	3,900	1.46	11,300	1.69	7,400	1.30
1768 in./min.	6,200	2.30	20,000	3.22	20,600	3.61
	Moisture Content = 12%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	2,900	1.00	4,200	1.00	5,200	1.00
14.5 in./min.	2,950	1.02	3,600	0.86	3,400	0.65
1768 in./min.	2,400	0.83	5,600	1.33	5,200	1.00
	Moisture Content = 14%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	2,200	1.00	2,050	1.00	2,500	1.00
14.5 in./min.	1,950	0.88	1,600	0.78	1,400	0.56
1768 in./min.	550	0.25	2,250	1.10	1,250	0.50
	Moisture Content = 16%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	1,040	1.00	600	1.00	150	1.00
14.5 in./min.	1,000	0.96	600	1.00	500	3.33
1768 in./min.	250	0.24	550	0.92	150	1.00
	Moisture Content = 17%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	800	1.00	200	1.00	75	1.00
14.5 in./min.	450	0.56	350	1.75	200	2.67
1768 in./min.	200	0.25	200	1.00	50	0.67

As regards moisture contents of 12 percent or above it appears that an increase in the rate of strain will not produce an increase in the modulus of deformation. From Figure 8 and Table 5 it can be seen that for certain moisture contents there appears to

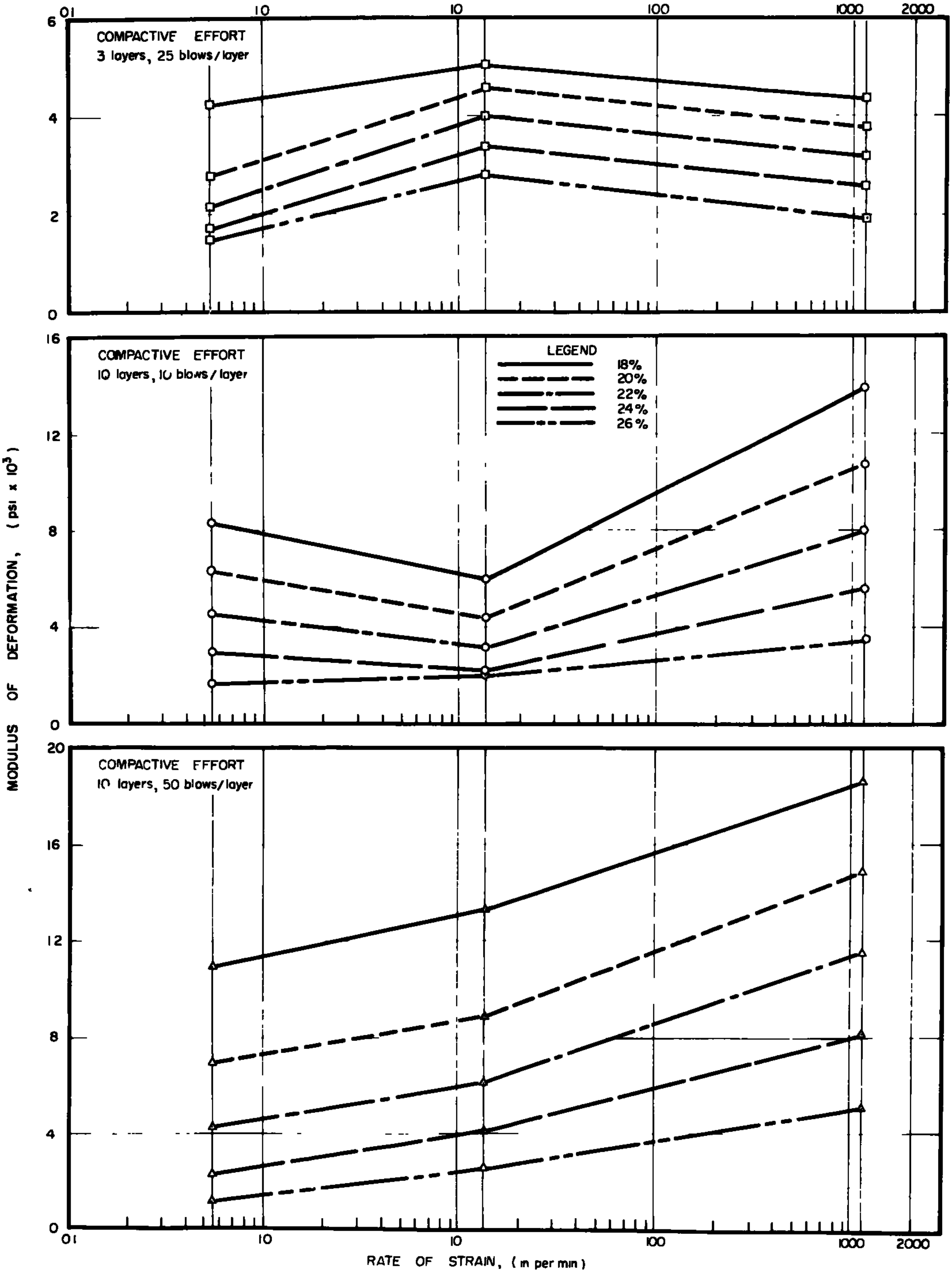

Figure 9. Rate of strain vs modulus of deformation—clay.

be a decrease in the modulus of deformation with an increase in the rate of strain. This can probably be attributed to the scattering of data. Also, the scattering of data might be the reason there was a significant increase in modulus of deformation with rate of strain for the 10 percent moisture content specimens.

Table 5 indicates an important trend. For a given compactive effort and rate of strain the M_D ratio decreases with increasing moisture content. There was, however, too much variability of data to make quantitative comparisons between various compactive efforts.

Clay

In the clay soil there was also a substantial decrease in the modulus of deformation for a given increase in moisture content at a given rate of strain (Fig. 9). This condition was more pronounced the higher the compactive effort and the greater the rate of strain. On the basis of the data given, at moisture contents approximating the optimum

TABLE 6

SUMMARY OF MODULUS OF DEFORMATION DATA — CLAY

	Compactive Effort					
Rate Strain	3 Layers/25 Blows		10 Layers/25 Blows		10 Layers/50 Blows	
	Moisture Content = 18 %					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	4,300	1.00	8,300	1.00	10,900	1.00
13.5 in./min.	5,100	1.19	6,000	0.72	13,400	1.23
1140 in./min.	4,400	1.02	13,900	1.68	18,700	1.71
	Moisture Content = 20 %					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	2,800	1.00	6,300	1.00	7,000	1.00
13.5 in./min.	4,600	1.64	4,400	0.70	8,900	1.27
1140 in./min.	3,800	1.36	10,700	1.70	15,000	2.14
	Moisture Content = 22 %					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	2,200	1.00	4,500	1.00	4,200	1.00
13.5 in./min.	4,000	1.80	3,100	0.69	6,200	1.48
1140 in./min.	3,200	1.45	8,000	1.78	11,600	2.76
	Moisture Content = 24%					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	1,700	1.00	3,000	1.00	2,300	1.00
13.5 in./min.	3,400	2.00	2,200	0.73	4,100	1.78
1140 in./min.	2,600	1.53	5,600	1.87	8,300	3.61
	Moisture Content = 26 %					
	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio	M_D (psi)	M_D Ratio
0.55 in./min.	1,500	1.00	1,600	1.00	1,200	1.00
13.5 in./min.	2,800	1.87	2,000	1.25	2,600	2.17
1140 in./min.	1,900	1.27	3,600	2.25	5,300	4.41

moisture content for Standard AASHO the effect on the modulus of deformation of an increase in compactive effort is negligible. However, as the rate of strain increases the effect of the amount of compactive effort becomes apparent.

From Figure 9 and Table 6 the effect of rate of strain and moisture content on the modulus of deformation can be observed. For all compactive efforts and rates of strain there was a tendency toward an increase in modulus of deformation with an increase in rate of strain. In a few instances, particularly the medium transient tests at the intermediate compactive effort, there was a slight decrease in the modulus of deformation with an increase in rate of strain. This can be attributed to the scattering of the data, and the attempt to draw a representative curve through it. The aforementioned reason was also the cause of the M_D ratios being less than one, in all probability (Table 6).

Consequently, it can be stated that for a given rate of strain the M_D ratio increases with increasing moisture content. Also, the M_D ratio, for a given moisture content and the fast transient test conditions, increased as the compactive effort increased. This tendency was not apparent in the M_D ratio for the medium transient tests.

In the case of the intermediate effort, considering the medium transient tests, the value of the M_D ratio obtained was much less than either of the other two compactive efforts. Also, the caliber of the data did not produce results from which a general trend could be predicted for the medium transient tests; that is, no definable tendency of the variance of the M_D ratio of the medium transient tests at a given moisture content but varying compactive effort.

As can be seen from Figure 9 moisture content has an increasingly important effect on the modulus of deformation the higher the compactive effort. Also, it is shown that significant increases in the modulus of deformation may be obtained by increasing the rate of strain. It should be noted that the higher the compactive effort the greater the effect of rate of strain on M_D.

For the lowest compactive effort, a decrease in M_D for an increase in rate of strain was noted (Fig. 9). This is probably due to the scattering of data and the use of average curves.

Comparison of Clay and Silty Clay

In both the clay and silty clay, a significant increase in unconfined compressive strength occurred with an increase in rate of strain for all compactive efforts and all moisture contents tested. However, with the silty clay, as the moisture content increased, the strength ratio decreased while for the clay the converse was true.

In both the clay and the silty clay the strength ratios for the fast transient test were greater for the lowest compactive effort. No definite relationship could be established for the medium transient tests between the strength ratios and the compactive effort.

In both the clay and the silty clay the moisture content proved to be more significant than the increase in rate of strain, as regards affecting the unconfined compressive strength. This tendency became less as the compactive effort decreased.

Moisture content has a great effect on the modulus of deformation of the silty clay as well as the clay. This condition was more pronounced the higher the compactive effort and the higher the rate of strain. Furthermore, for the clay soil, increasing the rate of strain produced a measurable increase in the M_D ratio, but this was not the case for the silty clay.

It is worthy to note that the data presented in this report do not give a straight line relationship, on a semi-logarithmic plot, for the variance of strength or modulus of deformation with respect to rate of strain (Figs. 4, 6, 8 and 9). The aforementioned is not surprising due to large number of variables present. Nevertheless, such a relationship can be obtained for certain soils (23).

From Tables 3 and 4 it can be readily seen why the effect of rate of strain is more significant in increasing the strength of the material at low density. This fact may be ascertained by comparing the difference in strength between the fast and slow transient tests for the same moisture content but varying densities. It can be seen that the increase in strength is relatively constant regardless of the compactive effort. Thus, due to the much lower slow transient strength of the material at low density, it is to be expected to show a greater percentage increase in strength for a given increase in rate of strain.

Also, it is interesting to note that the aforementioned difference is less variable, with moisture content, for the clay than the silty clay. This is because the silty clay is much more susceptible to changes in moisture content.

In contrast to Whitman (23), the data show a general trend toward a decrease in the strain at failure with an increase in the rate of strain. This is attributed to the fact that the faster the rate of strain the faster the load is applied which results in the specimen not having the opportunity of deforming greatly before it devlops its maximum shear strength.

Finally, it is of interest to investigate the possibility of thixotropic action affecting the results, since the specimens were tested approximately 24 hr after molding. Both Moretto (27) and Skempton and Northey (25) observed that thixotropic strength regain decreased with decreasing water content below the liquid limit. Also, Seed and Chan (26) showed that thixotropic effects are relatively small, for samples compacted on the dry side of optimum for the compactive effort being used.

It is realized that the thixotropic effect is not a well known phenomenon. Consequently, the results published by one investigator may be used by another only if the material and the test procedures are quite similar. Nevertheless, it was felt that the aforementioned work could be used as a guide to the design of the experiment. Thus, by testing specimens at moisture contents well below the liquid limit and by limiting the time of storage to a maximum of twenty-four hours, it was hoped that the effects of thixotropy would be relatively constant for all tests.

CONCLUSIONS

On the basis of these tests, and for the soils involved, it can be concluded that:

1. As the compactive effort is increased the greater the effect of changes in moisture content on the unconfined compressive strength, regardless of the rate of strain.
2. The lower the compactive effort the greater the effect of rate of strain on the strength. (a) Considering the silty clay, as the moisture content increases the strength ratio tends to decrease; and (b) considering the clay, as the moisture content increases the strength ratio increases.
3. As the rate of strain increases the effect of an increase in dry density, at a given moisture content, on the unconfined compressive strength decreases.
4. Increasing the rate of strain produced a measurable increase in the M_D ratio of the clay soil. (a) For the clay soil, as the moisture content increased the M_D ratio increased for a given compactive effort but the highest compactive effort yielded the greatest increase; and (b) in the silty clay soil there was a tendency toward a decrease in M_D ratio for a given rate of strain and compactive effort.
5. On the basis of these tests it must be concluded that to obtain significant increases in strength or modulus of deformation it takes a rate of strain equivalent to a fast transient test.
6. As the rate of strain decreases there is a tendency for the strain at failure to increase.

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