COMPONENTS OF PORE WATER PRESSURE AND THEIR ENGINEERING SIGNIFICANCE

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ABSTRACT

Pore fluid pressures that develop within soil masses as a result of both mechanical and physico-chemical effects influence the magnitude of the intergranular or effective stresses. The intergranular stresses control, in many cases, soil behavior in shear and compression. The physical significance of pore water pressure in a cohesive soil is examined in terms of several components which combine to give the total pressure. An analysis of fluid pressures at various points within a soil mass based on a condition of no flow at equilibrium shows that changes in any one component of the total pressure from point to point are offset by changes in other components.

The analysis shows that a pore pressure measurement reflects interparticle repulsive pressures and water adsorptive forces as well as purely hydrostatic pressures arising from mechanical effects. The individual components of total pore pressure are not directly measurable except in special systems. Data are presented which indicate that a significant portion of the swell of a compacted soil is attributable to water pressure deficiencies caused by mechanical and capillary components which act in addition to osmotic pressure components.

Component water pressures are related to stresses between particles. It is shown that intergranular pressures are dependent on osmotic and adsorptive components of the total water pressure.

INTRODUCTION

The principle of effective stress is one of the most important concepts of modern soil mechanics. It has been found useful as a basis for the understanding of stress and strain characteristics of soils and has become increasingly important in practical engineering problems. Terzhagi (1925) appears to have been the first to recognize the importance of effective stresses within soil masses. Bishop (1960b) has summarized the historical development of the concept of effective stresses in soil masses and has considered the theoretical aspects of the principle in detail.

According to the principle of effective stress the strength and compressibility properties of a soil depend not on the total stress applied to the soil

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mass, but rather on the difference between the total stress and the stress carried by the pore fluid. This difference is termed the effective or inter-granular[1] stress and is given, for a saturated soil, by

$$\bar{\sigma} = \sigma - u \tag{1}$$

where $\sigma =$ the total normal stress,
and $u =$ the hydrostatic pressure in the pore fluid.

Analysis of the forces acting within a soil mass across a surface which approximates a plane, but passes through the pore space and points of interparticle contact, indicates that the average intergranular force per unit area of the horizontal projection of the plane is given by

$$\sigma_i = \sigma - (1 - a_c)u. \tag{2}$$

where a_c is the contact area of the soil particles per unit area of the plane. Bishop (1960 b) has demonstrated, however, that $\sigma - u$ controls the volume changes of a granular soil independently of the contact area. Bishop also points out that whether shear strength depends on $\sigma - u$ alone and not on a_c is still a matter for conjecture. In most soils, however, the contact area is probably of very small magnitude; thus eqs. (1) and (2) could be expected to yield very nearly the same result.

The validity of the effective stress principle in saturated cohesionless soils has been demonstrated by Bishop and Eldin (1950). The principle has been found extremely useful in saturated clays as well; however, as pointed out by Lambe (1960), Lambe and Whitman (1959), Seed, Mitchell and Chan (1960), Rosenqvist (1959) and others, forces in addition to those arising from applied loads and hydrostatic water pressures come into play owing to the surface activity of the clay particles. In spite of these additional forces, the effective stress principle and information obtained through pore water pressure measurements correlate well with observed behavior of many fine-grained soils. At the same time, pore water pressure studies have yielded valuable information relative to soil structure and other physico-chemical aspects of soil behavior.

Recent studies have shown effective stress as stated by eq. (1) to be in-adequate to account for the behavior of partially saturated soils. Modifications of eq. (1) have been made by Bishop (1960a, 1960b, 1960c), Jennings (1960), Aitchison (1960), and Croney and Coleman (1953). The expression suggested by Bishop is the most general in that it accounts for pore air pressures different from 1 atm, a condition that may easily arise in practice. His expression is:

$$\bar{\sigma} = \sigma - u_{air} + x\left(u_{air} - u_{water}\right), \tag{3}$$

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[1] It should be noted that intergranular stress defined in this manner is not the actual stress transmitted between grain contacts, but represents the force carried by the solid structure per unit area of the mass.

where x is a parameter ranging between 0 in a dry soil and 1 in a saturated soil. Such modifications of eq. (1) are necessary since, in partially saturated soils the water pressure acts over only a part of the area of any plane through the soil.

As indicated by eqs. (1) and (3) the value of the pore pressure, u, usually determined by direct measurement, plays an essential part in the evaluation of effective stress. In systems where forces of a physico-chemical nature are active, the physical significance of pore water pressure may be somewhat different than in systems free from particle surface forces. Water located within the clay particle force fields may be expected to behave in a manner different from free water. It would seem logical, therefore, that pore water pressures should reflect the influence of these force fields as well as stresses induced by mechanical strain of the system. Expressions have been developed relating the change in pore water pressure to a change in applied total stress in terms of the relative compressibilities of the soil structure and water (Bishop and Eldin, 1950), and to shear and volumetric strains (Marsal and Resines, 1960). Fundamentally, however, the stress–strain and compressibility characteristics of a clay soil are functions of its composition and interparticle force systems.

In this paper the physical significance of pore water pressure is examined in terms of the various components induced by mechanical and physicochemical phenomena. Conclusions are drawn pertinent to studies of the engineering behavior of soil. The analysis relies heavily on the study of total and component potentials of moisture in soil developed by Bolt and Miller (1958).

ACKNOWLEDGMENTS

TOTAL AND COMPONENT PORE WATER PRESSURES

Total Pressure

It is convenient to consider the total pressure as the sum of several components measured with respect to a specified reference state, usually a body of free pure water with a flat surface exposed to atmospheric pressure and at the same temperature as the water under investigation. It is important to note that at equilibrium the total pressure is the same at every point and flow can occur only between points of different total pressure.

Positive pressures cause a flow from the point under investigation towards the reference pool.

Components of Total Pressure

A number of investigators, *e.g.* Edlefson and Anderson (1943), Baver (1956), Bolt and Miller (1958), Low and Deming (1953), Low (1958), Marshall (1959), have considered the total soil moisture or pressure in terms of several components. Both mechanical and thermodynamic analyses have been used. The results are generally similar in form, although certain differences in definition and detail have been pointed out by Bolt and Frissel (1960) who reviewed the pertinent literature on soil moisture thermodynamics and derived a general equation of soil moisture equilibrium.

The analysis of total and component potentials presented by Bolt and Miller (1958) is easily adapted for the present considerations of pore water pressure and is summarized extensively in a later section. In the present paper pressures rather than potentials are considered, since pressures have wide application in engineering practice.

The four components of the total fluid pressure (or total head) under conditions of constant temperature are assumed due to gravity force fields, hydrostatic pressure effects, osmotic or ionic concentration difference effects; and adsorptive force fields. As pointed out by Bolt and Frissel (1960), in any breakdown of the total pressures into components there is the danger of counting certain effects twice as they may be hidden in other terms. It may be, for example, that a rigorous division between osmotic pressures arising from double layer interactions and adsorptive effects cannot be made, thus invalidating the use of separate components for these two effects. This poses no serious problem in the present paper, however, since qualitative relations and concepts rather than rigorous mathematical relationships are desired.

(1) *Positional or elevation head, i.e. gravity pressure, z*, arises from the differences in elevation between two points.

(2) *Hydrostatic pressure, p*, arises from such factors as incomplete saturation which leads to curved air–water interfaces and in reaction to externally applied stresses. The hydrostatic component is the "pushing" or "pulling" component between particles. It is the pressure that would be reflected by a pressure gage inserted at the point in question. As will be shown, the magnitude of the hydrostatic component is dependent on the magnitudes of the other pressure components as well as the mechanical stresses.

(3) *Osmotic pressure, π*, arises from differences in ionic concentration from point to point. The concept of osmotic pressure in soils has been described elsewhere, *e.g.* Bolt and Miller (1958), Low (1959), Ladd (1959).

It is important to keep in mind that when considering two points at different ionic concentrations, if the ions are not free to move, water will tend

to flow from the point of lower concentration towards the point of higher concentration.

The osmotic pressure between two points of different concentration is approximated for dilute solutions by the Van't Hoff equation,

$$\pi = RT \left(\Sigma c_2 - \Sigma c_1 \right), \tag{4}$$

where R = universal gas constant,

             T = absolute temperature,

Σc_1, Σc_2 = sum of concentrations of all ionic constituents at points 1
             and 2, respectively.

It is assumed in the development of eq. (4) that ionic force fields do not mutually influence each other.

(4) *The adsorption pressure*, a, arises from the attraction of water molecules by clay surfaces. Bolt and Miller (1958) place attractive soil–water forces in two categories:

> (a) Short-range chemical forces, extending only a few molecular layers from the surface, caused by local ionic interactions, hydrogen bonds and London van der Waals forces.
> (b) Long-range forces, which may be effective beyond 100 Å, caused by interaction of a water dipole with the electrostatic field originating in a charged soil surface.

Bolt and Miller consider it reasonable to ignore the effects of short-range forces in usual soil–water systems as particle spacings exceed the range of influence of these forces. In the view of the author the more usual condition for soils encountered in engineering problems is one of essentially particle-to-particle (or particle–thin film of water and cations–particle) contact and therefore short-range forces should not be neglected. If the short-range forces are neglected the adsorptive pressure at any point is given by

$$a = - \frac{\varepsilon - 1}{8\,\pi}\, d\,(\nabla)^2, \tag{5}$$

where ε = dielectric constant,

       ψ = electrical potential at the point.

The potential ψ may be approximated by means of the Gouy–Chapman theory of the double layer. Adequate theories for expression of the short-range forces have not yet been developed.

The total pressure P is given by

$$P = z + p + \pi + a, \tag{6}$$

where z and p may be either positive or negative and π and a are usually negative in soils.

NEGATIVE PORE-WATER PRESSURES IN SOILS

Before analyzing component and total pressures at various points within a soil, a few comments and data relative to negative pore pressures will be considered as they aid in an understanding of osmotic effects and the influence of capillary effects on hydrostatic pressure. It is well known that water pressures less than atmospheric exist in soils. Whether or not pressures less than zero absolute (*i.e.* a state of tension in the water) can exist has been the subject of considerable dispute in the literature. Aitchison (1960) has considered the relative points of view and concludes that there is no satisfactory argument against the use of a capillary model and the development of large water tensions for describing the behavior of pore water at pressures less than atmospheric.

The fundamental causes of negative pressures appear to be osmotic and adsorptive effects and the surface tension of water. Mechanical factors such as a tendency towards straightening of bent particles on load release or a tendency towards dilation of the soil structure on shear may also cause negative water pressures but, in the final analysis, these effects are all dependent on the adsorptive forces between water and soil.

Negative pressures may exist in both saturated and partially saturated soils. The principal causes of negative pressures in saturated soils are probably osmotic effects, hydrostatic stresses resulting from a dilating tendency on shear, and hydrostatic effects resulting from the stresses carried by bent particles and distorted particle groups.

In partially saturated systems the surface tension of water comes into play in conjunction with the adsorptive forces at particle surfaces, leading to curved air–water interfaces which, in turn, result in additional pressures either positive or negative, depending on whether curvature is concave or convex across the interface. It is most convenient to think of this pressure as a capillary hydrostatic pressure given by $2\tau/r$ where τ is the surface tension and r is the radius of curvature of the meniscus.

It may be readily demonstrated that in partially saturated systems of nominal salt content (say 0.05–0.5 N) the osmotic component and the hydrostatic component due to capillary effects are of the same order of magnitude for menisci of about $0.5\,\mu$ radius of curvature. Capillary stresses become increasingly effective at smaller pore diameters and for radii of curvature of $0.1\,\mu$ or less they may exceed the osmotic pressures by ten times or more.

Data illustrating a condition where osmotic and hydrostatic tension due to mechanical effects are of the same order of magnitude in a partially saturated soil are presented in Fig. 1. Samples of an expansive sandy–clay soil were prepared to a water content of 17.3 percent, a dry density of 111.3 lb per ft³ and degree of saturation of 90.6 percent, with distilled water as the pore fluid. Two methods of compaction were used: kneading, which

at this water content induces a dispersed structure, and static, which leads to a more flocculent structure as demonstrated by Seed and Chan (1959). Samples were then permitted to expand under a surcharge of 0.1 kg per cm² in solutions of calcium acetate of different concentration. The relation between the amount of swell and solution concentration in Fig. 1 shows
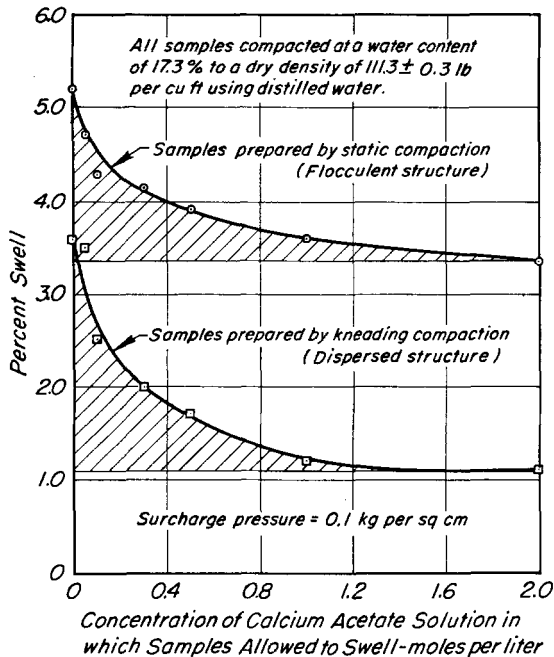

FIGURE 1.—Effect of structure and electrolyte concentration of absorbed solution on swell of compacted sandy clay.

that as concentration increases, swell decreases. This is a result of the decrease in the difference between the osmotic pressure in the double layers between particles and the osmotic pressure of the calcium acetate solution with increasing concentration of calcium acetate. Similar results have been obtained by Ladd (1959) for another soil. Also evident in Fig. 1 is the significant difference between the amount of swell of samples prepared by different methods of compaction.

It may be noted that the present percent swell approaches a constant value at the high salt contents, suggesting that osmotic or ionic concentration difference effects have ceased to be an important factor in influencing the amount of swell. It is doubtful that any significant swell would be observed in samples immersed in solutions more concentrated than 2 N if osmotic effects were the sole cause of expansion. Thus, as indicated by

the shaded areas in Fig. 1, osmotic effects probably can account for some 0 to $2\frac{1}{2}$ percent swell depending on the concentration of the swelling solution.

The remainder of the swell—about 1 percent for the samples prepared by kneading compaction and $3+$ percent for the samples prepared by static compaction—is probably attributable to initial hydrostatic pressure deficiencies arising from a combination of capillary and particle deformation effects and from water adsorptive forces at the clay particle surfaces. Measurements have consistently shown, *e.g.* Lambe (1961), that as-compacted pore water tensions are significantly greater for statically compacted than for kneading compacted samples of the same soil. The flocculated structures associated with static compaction lead to more pronounced effects from bent particles and capillary stresses (which hold bent particles in plane until relieved by exposure to water) as a result of the edge-to-face particle associations. In the more dispersed kneading structures the large shear strains during compaction enable particles to slide into a more parallel association with a resultant condition of less particle distortion. As a result, the release of capillary stresses is not accompanied by large volume expansions.

Water adsorptive forces at particle surfaces may have contributed somewhat to the swell of these samples since it has been observed that an initially air-dried sample will freely absorb water from a $98\pm$ percent relative humidity atmosphere to a water content of about 20 percent. As previously noted the compaction water content was 17.3 percent for these tests.

ANALYSIS OF COMPONENT AND TOTAL WATER PRESSURES AT VARIOUS POINTS IN THE SOIL–WATER SYSTEM

Following the analysis of Bolt and Miller (1958), it is convenient to consider first an "ideal" clay–water–electrolyte system at equilibrium. In Fig. 2 is shown a schematic representation of a part of a saturated soil mass composed of equally spaced parallel flat plates of uniform thickness. Hydrostatic pressures due to externally applied stresses may be acting.

Consider a piezometer inserted into the mass as shown in Fig. 2. The piezometer contains fluid that is in equilibrium with, but outside the force field of, the water in the double layers. The fluid level in the piezometer is adjusted so that a no-flow condition exists.

If all pressures are measured relative to a datum of pure water at the same elevation, then at point 1, Fig. 2: $z = 0$; $a = 0$, since the point is out of the range of adsorptive force fields; $p_1 = h\gamma_f$, where γ_f is the unit weight

of the fluid in the piezometer; and $\pi_1 = RTc_1$. Thus the total pressure at point 1 is

$$P_1 = p_1 + \pi_1. \tag{7}$$

Point 2 lies midway between two clay surfaces separated by a distance $2d$. Theoretical equations (Bolt, 1955) have been developed enabling the computation of osmotic pressures between interacting plates, and solutions have been tabulated (Bolt and Miller, 1955; Bolt, 1956). Thus π_2 can, in theory at least, be evaluated.
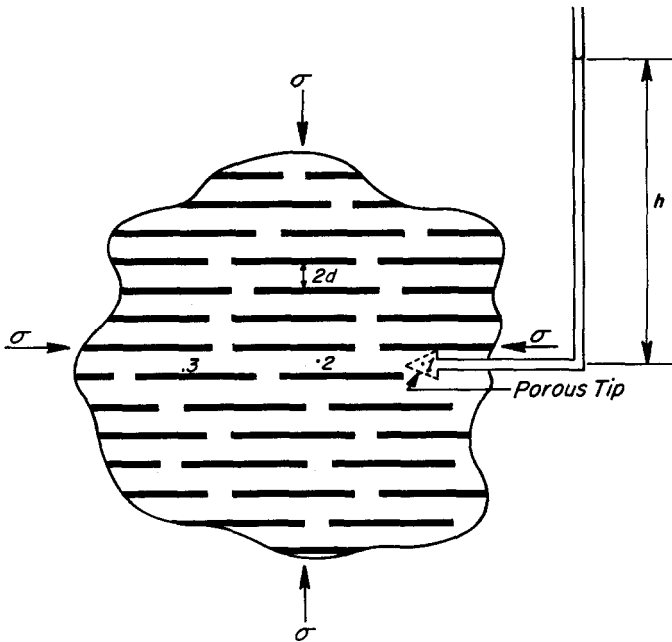

FIGURE 2.—Schematic diagram of idealized saturated clay–water system.

Since $\nabla \psi_2$ must equal 0 if short-range forces are neglected because the double layer is symmetrical about the mid-plane, then $a_2 = 0$, according to eq. (5). At equilibrium, the condition $P_2 = P_1$ must exist, and assuming that the elevation of point 2 is the same as that of point 1:

$$P_2 = \pi_2 + p_2 = \pi_1 + p_1. \tag{8}$$

Equation (8) shows that the hydrostatic pressure components p_1 and p_2 must differ by an amount equal to the difference in osmotic pressure.

Point 3 represents the general case of a point located between two parallel interacting plates but not at the center line. For simplicity let

$$z_3 = z_1 = 0.$$

The adsorptive component is given by eq. (5), and ψ_3 and π_3 are given theoretically by the double layer equations. Since $P_3 = P_1$,

$$a_3 + \pi_3 + p_3 = \pi_1 + p_1. \tag{9}$$

Equation (9) indicates that since the total pressure remains the same at all points, the individual components at point 3 assume values such that their sum equals P_1. If the osmotic and adsorptive components are given correctly by the theoretical expressions, then the hydrostatic component p_3 assumes a value appropriate to satisfy the condition $P_3 = P_1$.
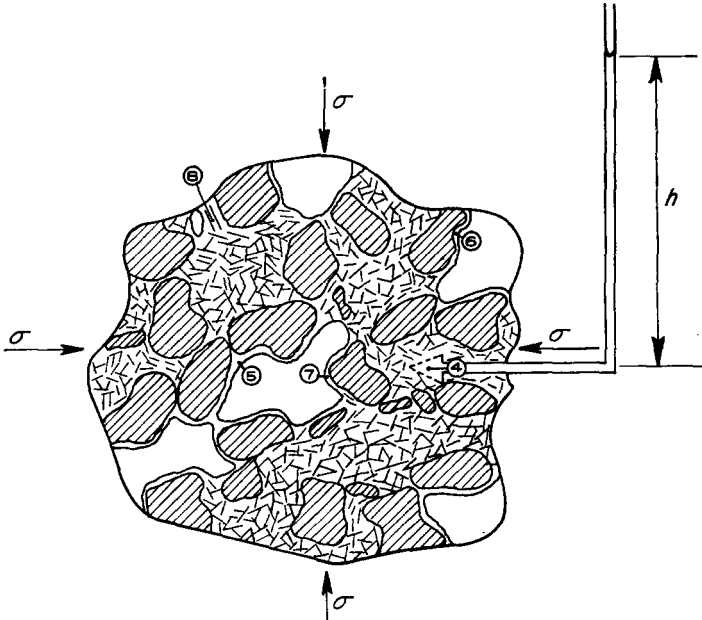

FIGURE 3.—Schematic diagram of partly saturated soil containing coarse particles and imperfect clay orientation.

An ideal clay–water system such as considered to this point is rarely, if ever, encountered in nature. In the more general case the presence of coarse particles and the influence of incomplete saturation must be considered. In Fig. 3, incomplete saturation and the presence of coarse particles are indicated; a piezometer with water level adjusted for equilibrium is shown. As in the preceding analysis, let it be assumed that all points are at essentially the same elevation so that z components may be neglected.

Point 4 represents a point in the fluid in equilibrium with the sample and, as for an ideal clay water system,

$$P_4 = \pi_4 + p_4, \tag{10}$$

both of which are measurable.

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Point 5 is located just inside a curved air–water interface out of the range of the double layers surrounding fine or coarse particles. Analysis of static equilibrium across the interface shows, with τ positive as shown, that

$$p_5 = p_a - \frac{2\tau}{r_5}, \tag{11}$$

where r_5 = radius of curvature of the meniscus at point 5,
   τ = surface tension,
   p_a = air pressure in the void.

Since both points 4 and 5 are out of the range of double layer fields, the ionic concentrations are equal ($\pi_4 = \pi_5$) and adsorptive forces are zero, so that:

$$P_5 = p_5 + \pi_5 = p_a - \frac{2\tau}{r_5} + \pi_5 = \pi_4 + p_4, \tag{12}$$

because $P_5 = P_4$ at equilibrium.

It follows from (12) that

$$p_4 = p_a - \frac{2\tau}{r_5}. \tag{13}$$

This expression has been developed by others, *e.g.* Hilf (1956), and has been usedas a basis for the measurement of negative pore pressures of a magnitude greater than would cause cavitation in usual measuring devices. By increasing the air pressure around the sample the point of reference is translated into the positive pressure range.

In Fig. 3 a water film is shown extending around the periphery of the coarse particles, as it is known that coarse particles do possess an adsorbed water film even though this layer is not thick relative to the particle size as is the case for clays. There are, therefore, points of negative as well as positive curvature within the system. At point 6 just inside the surface the curvature is positive and the point lies within the double layer. It follows that

$$P_6 = p_a - \frac{2\tau}{r_6} + \pi_6 + a_6. \tag{14}$$

Since $P_6 = P_5 = P_4$,

$$p_a + \pi_6 - \frac{2\tau}{r_6} + a_6 = p_a + \pi_5 - \frac{2\tau}{r_5} = p_a + \pi_4 - \frac{2\tau}{r_5}, \tag{15}$$

and

$$\pi_6 + a_6 = \pi_4 + 2\tau \left(\frac{1}{r_6} - \frac{1}{r_5} \right). \tag{16}$$

Equation (16) indicates that the sum of the osmotic and adsorptive pressure components at a point within the double layer is equal to the osmotic pressure at a point outside the double layer plus the difference in hydrostatic components due to interface curvature changes between the point in the

double layer and a point in the field-free regions. It was assumed in the development of eq. (16) that the air pressure in all voids is the same. This may be expected in partially saturated granular soils where particle surface phenomena are negligible and in partially saturated clays containing inter-connected air voids. If air voids are isolated, however, the air pressures need not be the same from point to point. Isolated air voids would be expected above some relatively low degree of saturation. Thus, the pressure conditions within a partially saturated clay are undoubtedly much more complicated than the preceding analysis would indicate.

Similar relations may be developed for point 7, which has negative meniscus curvature, and point 8, which lies midway between parallel clay plates. The curvature and concentration at any points are, of course, dictated by a number of factors such as arrangement of particles, size of pores, degree of saturation, magnitude of the electrolyte concentration in the system as a whole, and water structure. It is logical to assume that the actual variations of radii of curvature, osmotic pressure, etc., are such that the energy of the system is a minimum at equilibrium.

LIMITATIONS OF THE ANALYSIS

Bolt and Miller (1958) point out that the expanding lattice clays, *e.g.* montmorillonoid minerals, are the only ones that approach the properties assumed for ideal clay–water systems. Other clay minerals, such as the illites and kaolinites, are much thicker, have a much lower specific surface, may deviate widely from the assumption of thin flat plates, and may exhibit behavior typical of both coarse and fine particles. In addition terraced rather than planar surfaces may exist, complicating the determination of interparticle spacing.

The Gouy-Chapman theory of the double layer is itself severely limited in application because of the restricting assumptions required for the solution of the differential equations. Although it has been rather conclusively demonstrated that certain clay particles may be positively charged at their edges, the effect of this reversal of charge from surface to edge is not considered in the theory. The facts that most natural soils are more or less random mixtures of several electrolytes and minerals and that the clay plates are not likely to be arranged in a precisely uniform parallel array are further complicating factors. In addition, interparticle spacings are generally small, possibly introducing pressures in the water not accounted for by existing theories.

The adsorptive components were found in most cases to be zero. In actuality this would probably not be the case. It is more likely that a component of the adsorptive force fields would be active at all points between two adjacent particles, even at points where the electric field strength is zero. It is quite possible that water is adsorbed by the formation of hydrogen

bonds with particle surfaces or by other mechanisms not related to the electric field. Nowhere in the analysis has the effect of interparticle attractive forces on water pressure been considered. There is no reason to believe that van der Waals and other attractive forces known to exist between particles go entirely unfelt by the water phase.

Thus, while quantitative applicability of the relationships indicated by the above analysis has been demonstrated for certain highly idealized systems (Bolt, 1956), direct application of the results to soils of the type encountered by the engineer has been unsuccessful (Mitchell, 1960). There are, however, several fundamental concepts that emerge from such an analysis that may aid in the understanding of clay behavior when considered on a purely qualitative basis.

CONCLUSIONS FROM THE ANALYSIS

The analysis of component and total water pressures leads to the following conclusions relative to the understanding of the significance of pore pressure measurements in fine-grained soils.

(1) The total water pressure in a soil–water system at equilibrium is everywhere the same and may consist of any one or a combination of several component pressures. At a given density external forces must usually be applied to the soil mass or water phase in order to maintain a no-flow equilibrium condition when the liquid phase is in contact with external water.

(2) The total pressure is measurable and given by eq. (7) for the case of no elevation head.

(3) Double layer interactions and the consequent repulsion between opposing particles are reflected by the osmotic pressure.

(4) At equilibrium the component water pressures vary from point to point in such a way that the total water pressure remains constant.

(5) Air–water interfaces of both negative and positive curvature may exist in the same soil–water system. Interface curvature that results from an equilibrium between surface tension, adsorptive forces, amount of available water and system geometry is reflected by a change in hydrostatic pressure from the value existing beneath a flat air–water interface.

THE PHYSICAL SIGNIFICANCE OF PORE PRESSURES AS MEASURED IN SOIL MECHANICS TESTS

On the basis of the pressure concepts developed in the preceding section it is of interest to attempt an interpretation of the physical significance of pore water pressures as usually measured during strength and compression

tests on clays. Saturated clays only will be considered. Pore pressures commonly are measured by a sensing element (usually either a porous tip inserted within the sample or a porous stone at the base of the sample) which is connected to a measuring device, as shown schematically in Fig. 4. Connection is made from the sample through the porous stone to a null
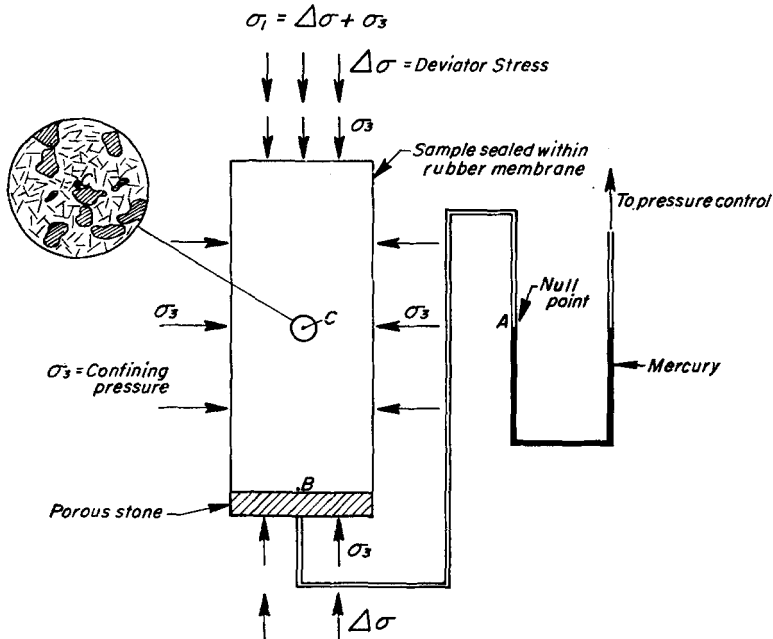

FIGURE 4.—Schematic diagram of triaxial test arrangement with null-point pore pressure measuring system.

balance point, A. The water level is maintained at point A throughout the test; the positive or negative hydrostatic pressure, p_A, required to maintain the level at A is taken as the pore water pressure within the sample; this value of pressure is substituted for u in eq. (1).

Let point C be some representative point between particles. If the system is in equilibrium, then the total pressure at all points is the same and $P_A = P_C$. The components of total pressure at point A are p_A given by the applied back pressure, and the osmotic pressure at A, π_A, which depends on the composition of the fluid at point A. The osmotic pressure at C is determined by the double layer conditions which, in turn, are dependent on the electric field strength from the particle surfaces and the

amount and type of electrolyte in the sample. The hydrostatic pressure p_c must have a value appropriate to satisfy the condition $P_A = P_c$; thus

$$p_c + \pi_c + a_c = p_A + \pi_A,\tag{17}$$

and if pure water is used in the measuring system,

$$\pi_A = 0, \quad \text{and} \quad p_A = p_c + \pi_c + a_c.\tag{18}$$

It would thus appear that measured pore water pressure includes the hydrostatic pressure, the osmotic pressure and the adsorptive pressure between particles. During a shear or consolidation test, pore pressure measurements should reflect the net effect of changes in the various components. It should be noted also that the pressure components may vary from point to point within the sample. Thus the hydrostatic component, which is the one that is "felt" by the particles in the sense of a thrust which influences the static equilibrium of the structure, is a varying quantity throughout the sample.

In the development of eqs. (17) and (18) it was assumed that the sample was in equilibrium with the measuring system. This implies not only a condition of no-flow but also complete osmotic equilibrium. Such a condition will exist only if the electrolyte concentration in the porous stone and measuring system is the same as the free electrolyte concentration of the sample. If pure water is used in the measuring system then equilibrium will exist initially if the clay contains only sufficient electrolyte to just satisfy double layer requirements and no excess. Otherwise the measuring system should contain electrolyte of the same type and concentration as the free salt in the clay for eqs. (17) and (18) to be valid.

If, as is usual, a clay containing free salt is connected to a measuring system containing salt-free water then equilibrium will not be reached within any convenient test period. There will be an abrupt electrolyte concentration change in passing from the sample to the porous stone. Since the stone cannot restrict the movement of the free salt ions there will be a slow diffusion of ions through the stone until the free salt concentration in both the sample and the system is the same. This process is extremely slow and may take weeks or months to reach completion. Several tests were run wherein 0.5 N NaCl and pure water were placed on opposite sides of a saturated fine porous stone of about 0.4 in. thickness. No salt was detectable on the water side of the stone after a 24 hr period. Since the porous stone was not impermeable to ions, however, there was no measurable osmotic pressure across the stone.

In view of these considerations, therefore, eqs. (17) and (18) will not be correct in most cases. The osmotic pressure term, π_c, may be visualized as consisting of two components, π_c' due to the concentration of free salt, and π_c'' caused by the additional ions needed at point C in order to

satisfy double layer requirements. The double layer ions are not free to diffuse into the measuring system; therefore, osmotic pressure π_c'' will be reflected by p_A. The osmotic pressure π_c' due to excess salt will not influence the value of p_A. Thus, eq. (18) would be

$$p_A = p_c + \pi_c'' + a_c = p_B, \tag{19}$$

where π_c' is an osmotic pressure due to a difference in salt concentration from point C and a point out of the particle force fields, but containing free salt, such as point B in Fig. 4. Thus, if points A and B are at the same elevation the pore pressure measurement gives the total pressure at C relative to a fluid containing free salt in equilibrium with the clay; *i.e.* the pressure at C relative to B. This is precisely the pressure desired in practice since the hydrostatic pressures that develop relative to the *in situ* pore fluid when a clay is sheared or compressed are the ones that will influence the effective stresses.

The foregoing considerations should hold true regardless of the salt content of the measuring system so long as an appreciable quantity of salt and water does not diffuse in or out of the sample before or during a test. If a significant amount of free salt diffuses out of the samples and into the measuring system, for example, there will be a change in the double layer osmotic pressure, π_c'', since double layer concentrations are extremely sensitive to free salt content of the clay. It would appear desirable, therefore, to test samples as soon as possible after placing them in contact with the measuring system, to keep the volume of water in the measuring system to a minimum, and to prevent the flow of fluid between sample and measuring system, in order to maintain the original salt concentrations at all points within the sample. For long term testing equal concentrations of free salt in both the measuring systems and sample would be desirable.

Tests are currently in progress which are designed to establish the validity of these conclusions. The limited data obtained thus far would tend to support the conclusion that the measured pore pressures are independent of salt concentration in the measuring system.

A PHYSICAL INTERPRETATION OF EFFECTIVE STRESS

It was pointed out at the outset of this paper that forces other than those due solely to applied stresses and purely hydrostatic water pressures are active in fine-grained soils. The analysis of component and total water pressures indicates that some of these additional forces are reflected through the pressures developed in the fluid phase. The concept of effective stress may be examined in the light of these findings. Mechanistic approaches to the expression of effective stresses in terms of interparticle forces as well

as applied forces have been presented by Lambe (1960), Lambe and Whitman (1959) and Seed, Mitchell and Chan (1960), among others, all leading to final expressions of the same general form. An alternative approach based on the analysis of water pressure is possible.

Clay particles are assumed to be essentially in contact with the typical particle association being corner- or edge-to-face as shown in Fig. 5. Whether the contacts are mineral-to-mineral or there is some few Ångströms separation with the intervening space filled with adsorbed water and cations is
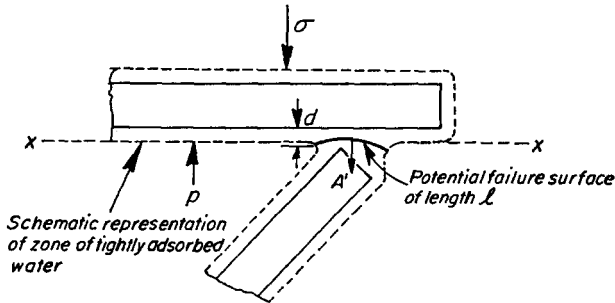

FIGURE 5.—Idealized representation of forces between clay particles.

not known. It does not seem unreasonable, however, that some type of solid or semisolid contact exists. A completely saturated system is assumed. It is recognized that no sharp boundary, such as shown by the dotted line, separates free and bound water. Such a line is shown only to aid in the definition of a surface of length l through which slip must occur if the interparticle bond is to fail in shear.

In examining the static equilibrium of this condition the forces to be considered, Fig. 5, are:

σ = the applied average external stress.

A' = the net physico-chemical attractive force between particles. It is assumed that attractions predominate in the contact zone and are responsible for true cohesion in clays. These forces may arise from van der Waals attraction, cementation, cation linkages, hydrogen bonds and other mechanisms. Repulsive forces may also be active in the interpartical contact zone; A' is taken as the excess of attraction over repulsion.

p = the hydrostatic component of the total pore water pressure.

The hydrostatic pressure p at any point between clay particles is related to the hydrostatic pressure in the fluid at a point out of the particle force fields p_B; according to eq. (19),

$$p_B = p + \pi'' + a, \tag{20}$$

where π'' is the osmotic pressure at the point between particles relative to the osmotic pressure at a point outside of the particle force fields, and a is the adsorptive component of water pressure. Both π'' and a are negative components; that is, the higher concentration of ions between clay particles resulting from interacting double layers and the adsorptive pressure tend to draw free water in. The effect of these pressures then is to keep particles apart. They may be considered as one term R, the interparticle repulsive pressure due to the physico-chemical effects introduced by the surface activity of the clay particles. Thus eq. (20) becomes:

$$p_B = p + R,$$

but since the pressure $p_B = p_A$, the measured pressure, and p_A is conventionally termed u we obtain

$$u = p + R.$$

Further, since R is a negative component we may write

$$u = p - R. \tag{21}$$

Examination of the static equilibrium of the particles, Fig. 5, shows

$$\sigma a' + A' = p a' + C,$$

where a' is the effective area covered by the particles and C is the contact force between particles.

Converting forces to stresses gives

$$\sigma + A'/a' = p + C/a'.$$

It may be noted that C/a' is now the actual stress transmitted between particles per unit area of the mass; *i.e.* the true intergranular stress, $\bar{\sigma}$.

Letting $A'/a' = A$ and $p = u + R$, we obtain $\sigma + A = u + R + \overset{''}{\sigma}$, or

$$\bar{\sigma} = \sigma + A - (u + R). \tag{22}$$

This expression, developed from a consideration of pore water pressure components, is of the same general form as previously derived expressions for intergranular stress (Lambe and Whitman, 1959; Lambe 1960; Seed, Mitchell, and Chan, 1960). Relations such as (22) unfortunately are quite limited in their application; not only because of the assumptions and approximations used in their development, but also because methods are not available for the quantitative determination of the quantities A, R and a'.

Figure 6 has been prepared to illustrate the various interrelationships that exist between applied, pore water, and intergranular pressures. Four possible soil conditions are indicated with three cases of measured fluid pressure shown for each condition. The $u = 0$ case refers to a measured pore water
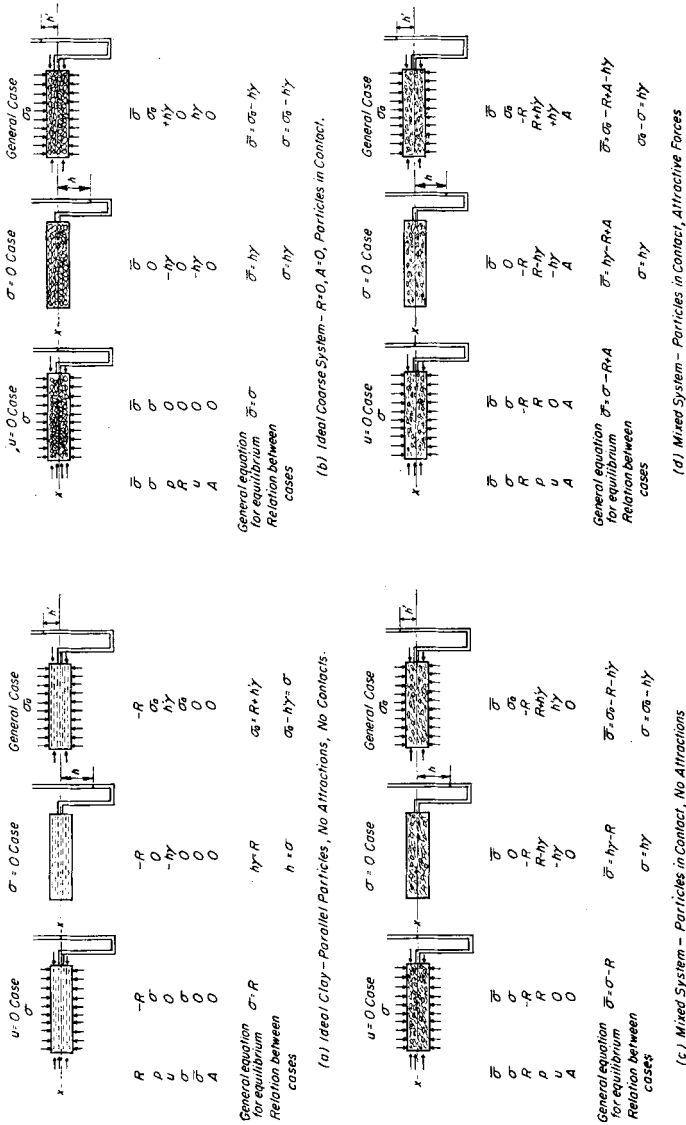
FIGURE 6. — Relationship between applied pressures, pore water pressures, interparticle attractive and repulsive forces, and intergranular pressure at equilibrium.

pressure of 0, the $\sigma = 0$ case refers to an equilibrium maintained by stressing the fluid that communicates with the sample, and the general case refers to the usual condition of both an applied normal stress and a measured pore pressure. Complete saturation and the same void ratio are assumed for each case.

Fig. 6(a) represents an ideal clay–water system composed of parallel particles, not in contact, with no attractive forces active. Three possibilities exist for maintaining equilibrium. As indicated by the $u = 0$ case the total pressure of the pore fluid can be increased to zero by the application of a normal stress σ to the sample, which must be carried entirely by the pore fluid since particles do not contact. The applied pressure induces a positive hydrostatic component in the pore water which balances the negative repulsive component. A second possibility is to decrease the total pressure of the free water to a value of $-R$ as indicated by the $\sigma = 0$ case. The third possibility consists of adjusting the total water pressures inside and outside the sample to the same value by applying a combination of normal pressure, σ_a, and stress to the free water, $h'\gamma$. Of course, an infinite number of σ_a and $h'\gamma$ combinations can be found so that equilibrium will be maintained, and the $u = 0$ and $\sigma = 0$ cases are actually special cases of the general case.

The values of the various pressure terms listed under the diagrams in Fig. 6 can be deduced readily from a consideration of the pressures shown on the diagram, the type of soil structure depicted, and an analysis of the statics of equilibrium. It may be seen that for the ideal system of Fig. 6(a) R is measurable.

Fig. 6(b) represents a coarse system with insignificant R components and contact between particles. Contact areas between particles have been assumed to be insignificant. Conditions (a) and (b) are combined in Fig. 6(c) where a mixed system with particles in contact and electrical repulsion forces active, but no attractive forces, is shown. It may be noted that a portion of the applied normal stress σ must be carried by the water in order to bring the total water pressure to zero. Only the difference $\sigma - R$ can be effective in creating an intergranular pressure $\bar{\sigma}$.

The most general case is shown in Fig. 6(d). It is evident from a consideration of the relationship shown under the figure that the intergranular stress is increased by the amount of the attractive pressure A. The general equation for equilibrium for the general case, Fig. 6(d), is of the same form as eq. (22) where $h'\gamma$ corresponds to u.

There arises from the analysis a further point of considerable interest; namely, what is the true meaning of effective stress in a clay? It is obvious, as shown by eq. (22), that in a soil with significant electrical repulsive and attractive forces, the intergranular pressure is not simply the difference between the total applied pressures and the measured pore water pressures. Lambe and Whitman (1959) have considered this point in considerable detail.

The possibility exists that effective stresses and intergranular stresses are not synonymous in clays. Seed, Mitchell and Chan (1960) suggest that strength should be a function of true intergranular pressure, and Lambe and Whitman (1959) point out that the observed behavior of expansive clays does not correlate well with effective stress as defined by the difference between applied pressures and measured pore water pressure. It is by no means sure, on the other hand, that true intergranular stress completely controls behavior in fine-grained soils. For example, void ratio–pressure relationships for even highly plastic soils, where R and A forces may be of appreciable magnitude, have been found to correlate well with an "effective stress" defined as the difference between total applied stresses and observed pore water pressures. Complete clarification of the problem of the significance of intergranular and effective stresses in fine-grained soils must await the results of future research.

SUMMARY AND CONCLUSIONS

It has been the objective of this paper to examine the physical significance of pore water pressure in clays and to point out some implications pertinent to soil engineering. Total pore water pressure may be conveniently considered to consist of the combined effect of four components; elevation, hydrostatic pressure, a pressure due to the adsorptive force fields of clay particles, and an osmotic pressure arising from the presence of dissolved salts and interparticle double layer effects. Data are presented which indicate a significant portion of the swell of a compacted soil on exposure to water is attributable to water pressure deficiencies caused by mechanical and capillary effects which act in addition to the pressure deficiencies arising from double layer interactions.

An analysis of the pressures at various points in a clay, based on the condition that at equilibrium (no flow) the total pressure is everywhere the same, shows that changes in any one component from one point to another are offset by changes in the others.

It may be concluded from the analysis that a pore water pressure measurement in a soil test reflects interparticle repulsive pressures (osmotic pressures) and water adsorptive forces as well as purely hydrostatic pressures arising from mechanical effects. The individual components of the total pressure are not measurable, however, except in special systems, and limitations in theory prevent the direct computation of these components.

A physical interpretation of effective stress is attempted by relating component water pressures to stresses between particles. It is shown that this approach leads to a relationship of the same general form as expressions for intergranular pressures previously developed using different approaches. Models are presented illustrating the interrelationship between component

water pressures, total water pressure, total applied pressure and intergranular pressure in clays.

It is to be expected that continued research will lead to improved understanding of the forces active in a clay–water system and aid in the determination of the true intergranular pressures in a soil mass and the evaluation of the true "effective" stress; i.e. the stress controlling soil behavior.

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