SYMPOSIUM ON PERMEABILITY OF SOILS

Presented at the
Fifty-seventh Annual Meeting
American Society for Testing Materials
Chicago, Ill., June 15, 1954

ASTM Special Technical Publication No. 163

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AMERICAN SOCIETY FOR TESTING MATERIALS
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FOREWORD

The papers and discussions in this Symposium on Permeability of Soils were presented at the Eleventh and Seventeenth Sessions of the Fifty-seventh Annual Meeting of the Society in Chicago, Ill., on June 15, 1954. The Symposium was under the sponsorship of Subcommittee R-4 on Physical Properties, under the chairmanship of Mr. Edward S. Barber, Civil Engineer, Arlington, Va., of ASTM Committee D-18 on Soils for Engineering Purposes. Mr. A. W. Johnson, Soils and Foundation Engineer, Highway Research Council, Washington, D. C. acted as Chairman for the Eleventh Session; while Mr. Harold Allen, Chief, Nonbituminous Section, Bureau of Public Roads, U. S. Dept. of Commerce, Washington, D. C. presided over the Seventeenth Session.

CONTENTS

SYMPOSIUM ON PERMEABILITY OF SOILS

INTRODUCTION

By Edward S. Barber[1]

In principle, determination of the permeability of soils is quite simple. However, due to natural variations of material in place, it is often difficult to relate tests on small samples to larger masses. In sampling soils it is hard to prevent disturbing the moisture or density or particularly the structure of the soil. Changes in the air or chemical or organic content of the permeating fluid can cause large differences. Migration of particles may occur both in the field and laboratory. While some variables can be arbitrarily controlled or eliminated in the laboratory, it is often necessary to consider them in field applications.

The Symposium includes papers discussing the importance, evaluation, and control of most of these factors. Field permeability tests are compared, described, and evaluated by formulas. Correlations are presented between permeability and density and gradation of granular materials. A new sampler and a device for testing under small gradients are described. The importance of relating tests to field conditions is stressed.

The test value is expressed as length divided by time in a variety of units, but it is generally called coefficient of permeability, although hydraulic conductivity is suggested as being more consistent with other fields such as electrical and thermal conductivity. While the variety of field situations seems to preclude a single standard test method, it should be possible to increase the consistency of results by recommendation of preferred practices.

Previous work is covered in a list of selected references.

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[1] Civil Engineer, Arlington, Va.; chairman of Subcommittee R-4 of Committee D-18 on Soils for Engineering Purposes.

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PRINCIPLES OF PERMEABILITY TESTING OF SOILS

By Donald M. Burmister[1]

The permeability of soils is a most important physical property since some of the major problems of soil and foundation engineering have to do with the recognition, evaluation, and proper handling of drainage problems encountered in the design and construction of structures. These problems include drainage of highways and airports, seepage through earth dams, uplift pressures beneath concrete dams and structures below ground water level, unwatering of excavated sites to permit construction in the "dry," seepage pressures causing earth slides and failures of retaining walls, etc. In all of these, the permeability characteristics of soils have a controlling influence on the effective strength properties of the soils and on their responses under stress, and hence on stability conditions. Drainable soils will act essentially as "open systems" with free drainage and fully effective shearing strength, whereas soils of low permeability may act as "closed systems" under rapid application of stress, with the development of pore pressures and reduction in shearing strength.

The determination of the permeability of soils is therefore a most important aspect of soil testing. The purpose of this paper is to formulate into a more complete form certain attitudes, concepts, and principles of a fundamental and comprehensive approach in permeability testing of soils and to increase the adequacy, reliability, and practical value of permeability data.

Controlling Environmental and Imposed Conditions

A basic fact in soil and foundation engineering is the inherently variable and complex character and behavior of soils and the dominating influences of environmental and imposed conditions upon the responses of soils. Soil engineers should realize that they are actually dealing with a very unconventional and in many respects a very unusual kind of engineering material. Hence in contrast to the essential uniformity of the common structural materials, the predictability of their behavior within the range of common working stresses, and the marked constancy of their properties for all common conditions of usages unaffected by external conditions, soils should not be expected to follow such simple conceptions and patterns of behavior. These facts may be summarized in two basic concepts (1):[2]

1. The character and responses of soils in any particular situation not only are predetermined by and are a part of the environmental conditions prevailing in that situation, but they are always markedly conditioned and modified in direct response to inevitable changes in those prevailing conditions by the new controlling conditions imposed by the structure itself.
2. In each situation, as a particularized case, the character and potential behavior of the soils (soil tests) must be considered directly in relation to the specific conditions

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[2] **The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 19.**

that control in the environment: whether different conditions are inherent in different locations in the natural situation, or whether the imposed conditions are different.

The drainage characteristics of residual soil deposits are determined by the character and structure of the soils in the different horizons of the weathering soil profile. These soil profiles have regular patterns and sequences of horizons with their character and thickness varying with the type and extent of weathering. The character and structure of the different soil layers forming the soil profiles of sedimentary deposits are determined by the constantly varying character of geological processes of deposition operating to form soil deposits.

But of equal or even greater importance in drainage problems is the dominating influences of the changed conditions imposed by the construction of structures; these may be favorable, neutral, or actually detrimental in character with regard to the behavior and responses of soils. The type and character of construction may materially change the pattern of subsurface drainage conditions. The lack of or the type of drainage system, its method of installation, and its effectiveness may play governing rôles. The time-delay aspects of drainage phenomena may be of major importance in construction. These conditions should be recognized, investigated, evaluated, and taken into account in design and construction. It is therefore important to give consideration to (1) the fundamental behavior characteristics of soils, in which permeability plays a dominating rôle, and (2) certain controlling conditions which govern drainage conditions. These conditions and behavior characteristics are outlined and discussed in detail in a previous paper (1, pp. 251–252).

In view of the large number, range, and varied character of environmental conditions to be expected in natural situations and of the changes in conditions imposed by construction of structures, which may govern drainage conditions, a most important and fundamental step in soil investigations is an accurate and complete visualization and appraisal of each situation with regard to the nature, relative dominance, and favorable or adverse aspects of the controlling conditions. Such an appraisal requires more detailed and adequate exploratory information on subsurface soil conditions, and more adequate and reliable soil test data than commonly provided and considered necessary.

CONTROLLED TEST METHODS AS APPLIED TO PERMEABILITY TESTING

At present little is really known about the fundamental properties of soils and of their responses in the original and changed environment. The difficulty is created by the inherently complex and variable character and responses of soils and by the fact of removal of soil samples from their natural environment. Paralleling the two concepts stated previously regarding the conditioning of the character and behavior of soils by their natural and changed environment, there is a third basic concept regarding the conditioning influences imposed during soil testing (3, p. 12):

> 3. During the observations and measurements of properties and responses of soils in soil tests, the observed properties and responses themselves are conditioned, modified, and changed by the very procedures and test conditions used to an important but unknown degree, unknown because there are no absolute response bases for references. Thus the observed and measured properties and responses are not necessarily those having any significant and direct relations to the actual properties and responses of the soils under the controlling conditions pre-

vailing or imposed in the natural situation. Therefore potential behavior only can be learned from soil tests.

Thus there are major difficulties involved even in the so-called simpler soil tests. In view of these three concepts, it should be realized that soil is a material which does not readily permit translation of responses made under one set of relatively arbitrary and even artificial conditions in a standard soil test into reliable predictions of responses under quite a different set of actual conditions in the field. Furthermore, the field conditions are not necessarily constant but may vary markedly from place to place for the same type of soil and with the seasons. The real soil testing problem is not to formulate a so-called average or norm of test conditions, but rather reasonably to bracket the actual possibilities in each situation. Therefore, as a basic and realistic approach, the properties and responses of soils should be determined by soil tests performed under conditions essentially equivalent to those to be expected under actual field conditions, both as to probable character and bracketing range of conditions. This is the essential nature and purpose of the controlled test methods in previous works by the author (2, pp. 83–89; 3, pp. 11–14), which state broad flexible principles of soil testing and describe general procedures, but which permit a wide latitude for making adjustments in the soil that are subject to control at the discretion of the soil engineer, without restricting methods to specific fixed procedures and types of apparatus. This approach represents a *conception of a new kind of standard soil test* as a valid basis for soil engineering investigations. Its principal objectives are reliably to reproduce responses representative of the natural situation. This is in marked contrast to the usual conceptions of standard soil test methods with their oversimplified average test conditions, their single fixed routine procedures for each method, and their emphasis on ease of application and on reproducibility of test data by different individuals.

The application of controlled test methods involves essentially introducing representative Test Conditions, which will properly condition each soil specimen by restoring, as nearly as possible, the original conditions that control in the natural environment, and by establishing thereafter in sequence the controlling conditions to be imposed by the construction of structures, in accordance with the following three basic principles of controlled test methods:

1. To appraise and to evaluate as completely as possible the real nature and degree of control of the environmental conditions in the natural situation and of the new conditions imposed in sequence by the construction of structures.

2. To translate this information into appropriate test conditions which are representative of and valid for a particular situation, and which can be adjusted and brought into significant agreement with and definitely made to fit the probable actual environmental and imposed conditions, as the responses of the soils become evident and are disclosed during the test.

3. To apply these test conditions by techniques and control in sequence during the conduct of a soil test, first, to determine accurately and completely the representative character and responses definitely impressed upon the soils by the natural environmental conditions; and, second, to determine the responses of the soils that would have direct, reliable, and valid applications in predicting the field responses and performances of the soils under actual conditions imposed by the construction of structures.

Soil testing is such an essential part of soil investigations for engineering works that every soil test should be treated essentially as a work of discovery under careful control. In view of the importance and real character of soil testing, it should be considered *design* in the true sense of the word, because it involves judgment in the practical applications of these three principles of soil testing, and because adequacy and reliability of the results of soil tests are not just simple matters of routine applications of standard test methods. Controlled test methods are therefore considered fundamental and realistic in soil and foundation engineering because they particularize each situation (1) to disclose and to evaluate the known and unknown conditions that control; (2) to provide specific answers that are representative of and directly applicable to each situation; and (3) to obtain the highest degree of agreement between the predictions of behavior and responses of soils and the actual observed soil phenomena.

There may be some difficulty in understanding the essential need for this apparently radical departure from the usual conceptions of standard soil testing. Many engineers engaged in soil work have had a structural background. As a consequence they may quite naturally be inclined to believe that analogous simple conceptions and treatments of soil phenomena and soil testing fit the facts and are acceptable as a valid approach. To a very large degree, attitudes and conceptions predetermine judgments and practice. The almost universal acceptance and use at present of standard soil tests are really predicated on the premise and fallacy that there is a simplicity and essential identity of action and responses of soils as *norms* in a standard soil test and under actual field conditions. A consideration of the foregoing three basic concepts concerning the important conditioning influences of the environment, of the structure itself, and of soil test conditions upon soil responses should establish the fact that such simplicity and essential identity of action and responses can seldom, if ever, be expected in the case of soils, whether in the natural situation or in a standard soil test made in accordance with usual conceptions. In this era of fully demonstrated value and use of basic scientific developments, scientific caution, common sense, and creative engineering imagination should reject the idea of simple answers to admittedly complex and difficult questions and problems. In soil engineering *adequacy* should not be permitted to become a fixed idea, but rather it should be constantly and consistently revised upward to keep pace with increases in knowledge and experience and to stimulate further advance.

Permeability Flow:

In order properly to conduct permeability tests, and to interpret and to use test data, consideration should be given to the nature of hydraulic phenomena. This paper is concerned with permeability flow, which takes place primarily through saturated soils under gravitational forces or under a pumping head with the water everywhere in the region of permeability flow under a positive hydrostatic pressure. In certain important cases such flow can take place through partially saturated soils. Sometimes under high-vacuum well-point pumping in the immediate proximity of the well-points, there may be a region where flow occurs under a negative pressure or suction, but with the voids of the soil flowing full of water. Capillary flow of water, in contrast, takes place under capillary forces primarily through a continuous interconnected system of thin capillary moisture films, principally at the grain contacts, with the water everywhere in a state of capillary tension, the distinguishing characteristic of capillary flow.

There are two important categories of flow, the one of principal importance being flow of water below the permanent ground-water level with the voids full of water (100 per cent saturated). The other now becoming of importance in soil engineering is the flow of water below a temporary elevated ground-water level or free water surface with different degrees of saturation and air-clogging of the soil voids. Important examples of this latter category are: (*a*) flow of water through river banks or levees caused by rising flood stages of a river; (*b*) flow of water through earth dams caused by rising water level in a reservoir; (*c*) flow of water through gravel drains or base courses beneath pavements during periods of large infiltration of rainfall; and (*d*) the rate of infiltration of rain water downward into soil. In these cases permeability flow applies some distance back from the advancing front of capillary flow, where pressure in the water has become positive hydrostatic.

TABLE I.—REALM OF VALIDITY FOR DARCY FLOW OF WATER IN GRANULAR SOILS.

Sieve	3 in.	1 in.	⅜ in.	No. 10	No. 30	No. 60	No. 200	
D_{10}, mm	76.2	25.4	9.52	2.0	0.59	0.25	0.074	0.02

	GRAVEL coarse medium fine	SAND coarse medium fine	SILT coarse fine
Realm of flow of water	Practically always turbulent flow.	Darcy laminar flow only for H/L less than about 0.2 to 0.3 for the loose state and 0.3 to 0.5 for the dense state.	Always laminar flow for the range of H/L found in nature.

In the initial stages of these categories, the flow is in a transient state and is time-dependent. After a period of time, depending on conditions, the transient state of flow approaches the steady state of flow with the final establishment of an equilibrium free water surface. If flow in the partially saturated condition persists long enough, the entrapped air will be gradually dissolved in the water. Only the steady state of flow with the voids full of water can be analyzed with any degree of exactness by present concepts and working hypotheses.

The principal realms of flow of water through soils are laminar flow or stream line flow, and turbulent flow. In laminar flow, viscous forces shape the character of flow with velocity proportional to the hydraulic gradient. Turbulence is initiated in soils at considerably lower velocities than usually recognized in the form of eddies and vortices in the larger void spaces, due to expansion, contraction, and change of direction effects. This turbulence results in increased resistance to flow and larger energy losses.

Based upon these considerations, only one type of flow, designated the Darcy type of flow or Darcy flow, is stable in character. It is described and strictly limited by four basic conditions: (*a*) the laminar realm, (*b*) the steady state, (*c*) flow with the soil voids 100 per cent saturated (no compressible air present), and (*d*) flow with the continuity conditions and basic equation of flow satisfied in a soil mass in which no volume changes occur (consolidation) during or as a result of flow. Permeability experiments (5) have established the realm of validity of the Darcy flow (Table I).

Darcy, as a result of his basic work on

the mechanics of flow of water through porous media in 1856, first stated the basic law of flow, which is absolutely general in its application within its realm of validity:

$$v = \frac{Q}{At} = K_D \frac{H}{L} \quad \text{..........(1)}$$

The velocity, v, is an average over-all velocity computed on the basis of the quantity of flow, Q, in a time, t, and of the entire gross cross-sectional area of the soil column, A. The hydraulic gradient, H/L, is expressed as the ratio of the head of water, H, causing flow to the length of the soil column, L, in which the head is lost. The Darcy coefficient of permeability, K_D, is therefore an over-all value, which provides an adequate, reliable, and stable basis for reference and for comparison of different conditions of flow. All other conditions of flow and their corresponding coefficients of permeabilities, designated by appropriate subscripts, are from the very nature of the phenomena inherently unstable in character.

Dominating Influences of Soil Material and Soil Structure:

The identification of the soil material (6, pp. 9–16) and the soil structure (5, pp. 1249–1255) are presented elsewhere. This discussion is limited to a consideration of the influences of character of granular soils that are relatively incompressible under flow of water, as required under validity condition (*d*), explained above, for the Darcy flow. For all finer grained soils, there is an important time delay in reaching a stable, consolidated soil structure. This makes the testing problem for such soils more difficult. The identifiable characteristics of the granular soil material that govern permeability flow are: (*a*) its composition and proportions of the gravel, sand, and silt components; (*b*) the gradation of these components expressed in terms of the relative predominance of the coarse, medium or fine fractions, as defined in Table I; and (*c*) the predominating grain shapes and surface characteristics. Since every recognizable and identifiable soil characteristic is certain to play an important and even dominating rôle in soil behavior and responses, as a basic concept it is important to identify accurately and completely the soils under consideration. The finest soil component and fraction, in general, appear to dominate permeability phenomena, because they tend to determine the sizes of the soil channels by their void clogging effects. Furthermore, it is important to give each soil a precise, significant, and distinguishing soil name (6, pp. 7-24) that will convey accurate information on those aspects of soil character in permeability investigations for present purposes as well as for future comparisons and correlations of permeability phenomena.

The character of the soil structure in natural deposits and compacted fills is first of all identified by its degree of compactness on a relative density basis (5, pp. 1249–1255). As a fundamental and practically useful concept in soil mechanics, relative density describes the significant state of compactness of the grain structure of granular soils. Furthermore, relative density relationships provide significant unifying bases for interpretations, evaluations, and practical applications (5, pp. 1255–1268). Such relationships give a clearer insight into and a better understanding of soil behavior and responses, as controlled by individual characteristics of soils and by the conditions inherent in natural situations. In addition, where no rigorous mathematical treatment is possible because of the nature of soil, these relationships permit a more complete statement to be made of the physical laws governing all granular soil phenomena by means of comprehensive graphical presentations with relative density as the common unifying basic argument. Relative density must be defined on the basis of the maximum and minimum densities (5, pp. 1250–1255), as definite physical limits for each granular soil and as the basic references for the 100 per cent and 0 per cent relative densities, respectively, in the defining equation:

$$\text{Relative Density: } D_R = \frac{e_L - e_N}{e_L - e_D} 100\ \%$$

$$= \frac{\frac{1}{W_L} - \frac{1}{W_N}}{\frac{1}{W_L} - \frac{1}{W_D}} 100\ \% \dots (2)$$

where:

e = the voids ratio of the soil,

W = the corresponding unit dry weight,

L = minimum (nonbulked) density reference,

D = the maximum density reference, and

N = the density of the soil in the natural or compacted state.

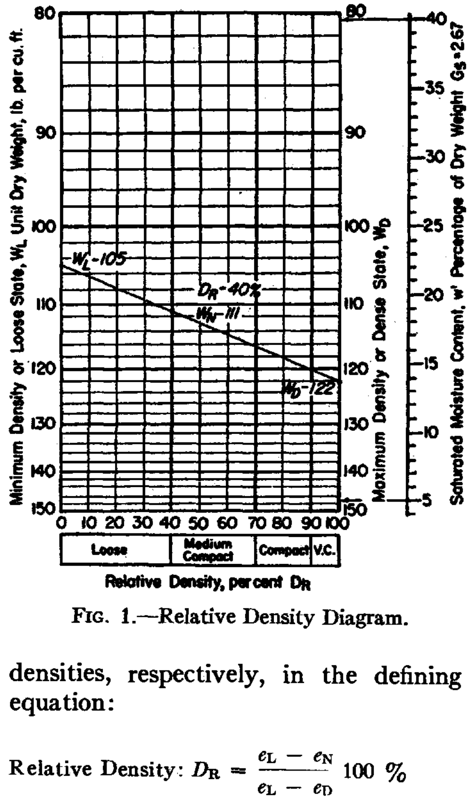


FIG. 1.—Relative Density Diagram.

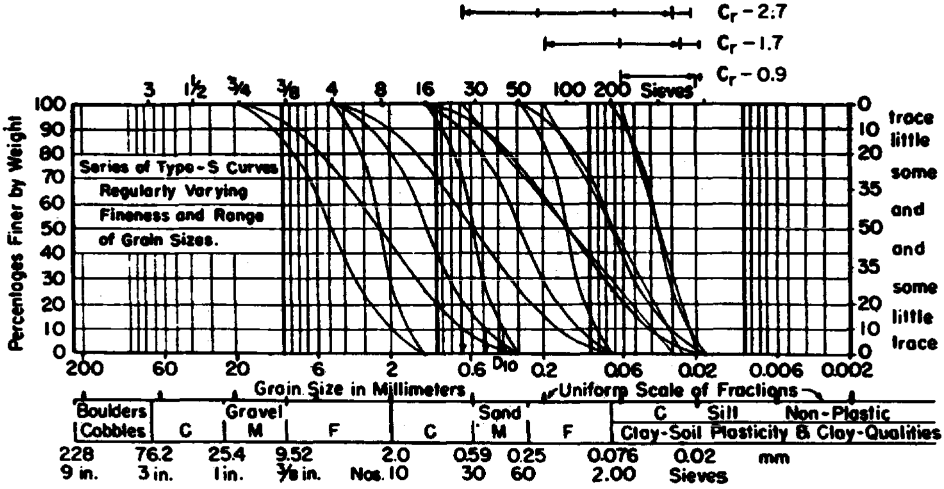
The relative density diagram given in Fig. 1 can be used conveniently for determining relative densities and unit weights. Experiments and correlations (5, pp. 1250, 1251) have shown that it is only in exceptional and unusual cases that natural deposits of granular soils have been found in a more dense state than the laboratory maximum density reference, or in a more loose state than the laboratory minimum density reference. The bulked densities of moist soils artificially placed in embankments would then have a minus relative density by Eq 2. The maximum density obtained for granular soils by vibration methods near resonant frequencies represents practically the maximum attainable by any method. This maximum density, therefore, is a stable reference for 100 per cent relative density.

Studies have shown (5, pp. 1252–1255) that the different geological processes tend to form soil deposits in relative density states characteristic of the soil material deposited. There is a significant and fundamental relative density concept, namely, the maximum and minimum density states, because they do represent practical physical limits for each soil, also *fix the limits of soil behavior and responses.* All natural soil phenomena for practical engineering purposes will fall between these limits.

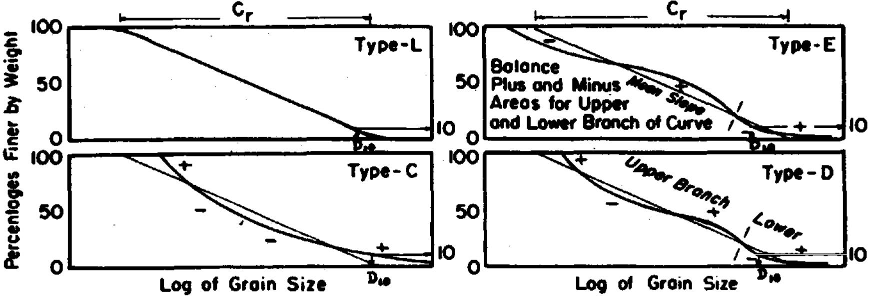
The character of the soil structure in natural deposits is determined by the nature of the geological processes of soil formation and by the character of the soils deposited. Thin layering and the degree of anisotropy of the layers govern the permeability properties of the layers in detail, particularly when visible.

PERMEABILITY - RELATIVE DENSITY RELATIONS

In order to disclose the nature and degree of control of the soil material and the soil structure upon permeability phenomena, investigations and research were carried out in 1943 and 1948 (5, 7). Test conditions were carefully formulated for the research on permeability to satisfy the three principles of soil testing and the four basic conditions of validity of the Darcy type of flow, explained earlier in this paper. These essential test conditions are listed as follows:



(a) Type-S Grain Size Curves



(b) Type of Grain Size Curve

FIG. 2.—Distinguishing Characteristics of Grain Size Curves.
Fineness, range of grain sizes, and type.

1. To bracket as to character and range of soil material from "coarse SAND" to "coarse SILT," synthetic granular soils were made up to produce by regular steps definite grain size distributions commonly encountered in practice, as shown in Fig. 2.

2. To bracket the full range of soil structure, tests were made at 0 and 100 per cent relative densities and at an intermediate relative density in the region of 70 per cent, using appropriate techniques to obtain reproducibility and uniformity of conditions (8, pp. 111–113).

3. To produce at each relative density an isotropic, homogeneous soil mass with practically no segregation effects or nonvisible anisotropy due to placing and compacting successive layers, and in order to provide the basic stable permeability references, just enough moisture was mixed into a test specimen (0.5 to 1.0 per cent by weight) so that the soil would not flow freely from a funnel with a ¼-in. spout for spreading thin layers. Then by additional mixing and testing with the funnel, the soil was partially dried and

brought to the correct moisture state, so that it would just flow freely from the funnel to form successive ½-in. layers for the loose state. The specimen was covered to prevent further loss of moisture while compacting each layer in the higher relative density states.

NOTE.—This isotropic condition can be checked at the completion of the permeability test by evacuating the water out of the test specimen. Any light and dark alternating streaks are evidence of segregation of fines and of anisotropy, the dark streaks being the finer segregated fractions, which have a larger water-holding capacity. True natural anisotropy cannot be successfully duplicated in the laboratory.

4. To hold the initial relative density state without volume change during saturation of the specimen and during the permeability test to satisfy condition (*d*) for the validity of the Darcy type of flow, a light spring pressure of about 2 psi over the area of the specimen was permanently applied through a suitable screen device to the top of the soil specimen prior to measuring the initial height of specimen and attaching the cap to the permeability device.

5. To saturate the specimen completely in accordance with validity condition (*c*), the permeability device was evacuated under full attainable vacuum (28 in of mercury or better, if possible) for 10 to 15 min to remove the air. This evacuation was followed by slow saturation of the specimen from the bottom upward under this vacuum.

6. To insure against air-clogging during the test, de-aired water was used, which was obtained from a special filter tank.

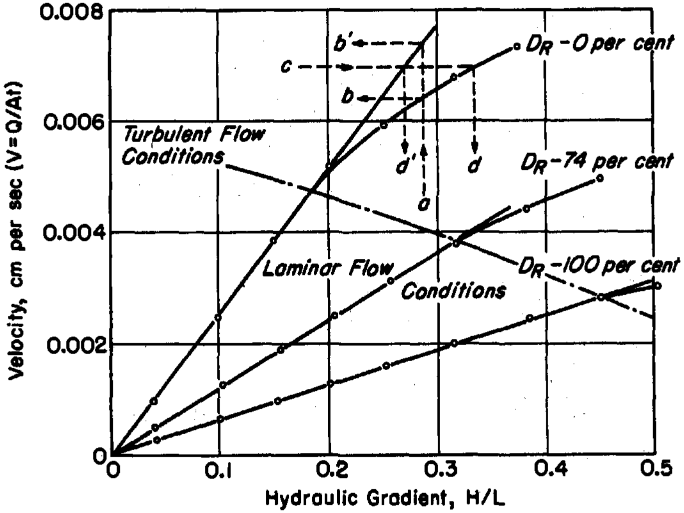
7. To insure laminar flow conditions in accordance with validity condition (*a*) and Table I, a sufficient number of points were obtained under constant head testing by varying the head in small steps below a gradient H/L of about 0.2 to 0.3 for the loose state and about 0.3 to 0.5 for the dense state (½-cm increments) in order to define the Q/At *versus* H/L permeability curve in the Darcy region of laminar flow. Thereafter the head was increased in larger steps to define the permeability curve in the region of turbulent flow. The lower values of H/L mentioned above obtain for the coarser soils and the higher values for the finer soils.

8. To insure a steady state of flow for each new head and plotting point in accordance with validity condition (*b*), the quantity of flow was measured only after a stable head condition in the manometers was attained.

NOTE.—In making the setup for the test, great care was taken to insure that the head manometers, tubes, and connections were free of air and were operating satisfactorily.

It should be evident from these test conditions that the common falling head type of permeability test is inherently unsuited for permeability testing of granular soils, because under a high starting head test condition No. 7 is not satisfied. Even in the constant head type of test in the laminar region of flow, it is not generally possible to go back and pick up a consistent point under a lower head. This is evidence that there has been some disturbance effects to the soil structure, even when test conditions No. 4 is reasonably satisfied.

Typical permeability test curves obtained by constant head testing are shown in Fig. 3; these define permeability flow conditions between the limiting maximum and minimum density states. The region of the Darcy flow is clearly defined in Fig. 3, in which the coefficient of permeability, K_D, for each relative density state is a stable constant determined from the linear portion of the curves for low values of H/L below the critical value defined by the curve separating the laminar flow region from the turbulent flow conditions. The research of Kane (5, 7) has established the fact that for the range of these soils the region of validity for laminar flow conditions is limited to hydraulic gradients below 0.2 to 0.3 for the loose state and below 0.3 to 0.5 for the dense state, the lower values being for the coarser



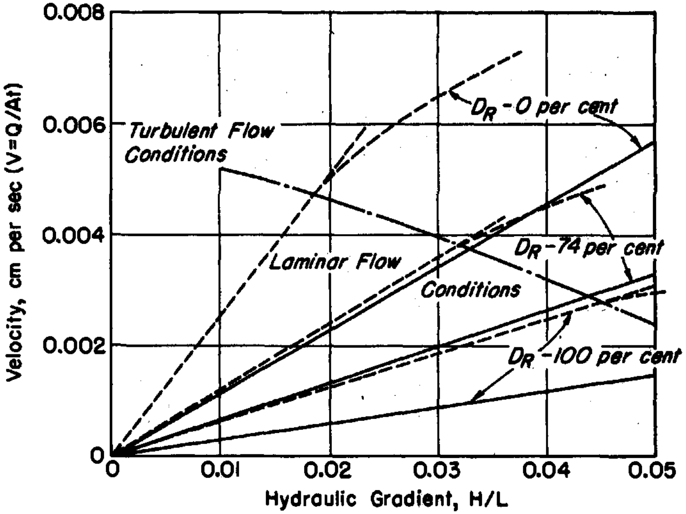
Test Sample No.3 "Fine SAND"

D_{10}-0.11 C_r-0.9 Type-S

50 per cent retained between No.48 and No.100 Sieve
50 per cent retained between No.100 and No.200 Sieve

Relative Density, per cent	Unit Dry Weight, lb per cu ft	Voids Ratio, e	Permeability, K, cm per sec
0	87.0	0.913	0.0263
74	99.0	0.682	0.0125
100	103.0	0.618	0.0065

FIG. 3.—Determination of Coefficient of Permeability Bracketing the Relative Density Limits.



Test Sample No.3 "Fine SAND"

D_{10}-0.11 C_r-0.9 Type-S

50 per cent retained between No.48 and No.100 Sieve
50 per cent retained between No.100 and No.200 Sieve

Relative Density, per cent	Unit Dry Weight, lb per cu ft	Voids Ratio, e	Per cent Saturation	Permeability, K, cm per sec
0	87.0	0.913	84	0.0115
74	99.0	0.682	80	0.0067
100	103.0	0.618	79	0.0030

FIG. 4.—Determination of Coefficient of Permeability Bracketing the Relative Density Limits Under Air-Clogging Condition After Resaturation.

soils. These values are much lower than commonly realized.

It is also apparent in the region of turbulent flow in Fig. 3, where the permeability curves depart from the linear relations of laminar flow, that a coefficient of permeability is definitely not a constant but is hydraulic gradient-dependent. Therefore in analyses of natural situations, where the natural hydraulic gradient in any localized region exceeds the above critical values, the only practical solution is obtained by the *direct use* of the experimental permeability curves in the region of turbulent flow. Such regions can be defined by Flow Net analyses. The direction and magnitude of the errors involved by the use of a constant Darcy coefficient of permeability in the turbulent region is dependent upon which quantity is fixed and controls in the situation. For example, for the case *a-b* in Fig. 3, if H/L at *a* is fixed, then the quantity of flow obtained at b' under turbulent conditions would be greatly overestimated. On the other hand, for case *c-d*, if the quantity desired at *c* is fixed as from a pumped well, then the required H/L at d' to produce this quantity under turbulent conditions would be greatly underestimated, due to large energy losses. These facts would have important implications in estimating seepage quantities and forces in pumping and stability problems if regions of turbulent flow are present.

In order to investigate the nature of the influences and the kind and degree of departure from the Darcy type of flow under conditions of partial saturation with air-clogging of the voids, the following two additional test conditions were applied at the completion of each regular permeability test to simulate conditions of infiltration of rain water into moist soil:

9. The specimens were drained by evacuation from the bottom for 15 min.
10. The specimens were then resaturated from the top down under aerated normal atmospheric conditions at low heads.

Under these specific conditions, the permeabilities in the region of laminar flow were found to be from 25 to 50 per cent of those for the completely saturated state of the Darcy type of flow. They tended toward the 25 per cent value for the finer grained soils, as shown in Fig. 4, using as references the Darcy flow permeability curves for the maximum and minimum densities. Investigations showed that the permeability characteristics under conditions of aeration were markedly dependent upon the history and sequences of events in obtaining partial saturation of the soil, namely, (1) the initial moisture state between the limits of air dry and almost complete saturation; (2) the rate of saturation; (3) the direction of saturation flow as affecting displacement of air and whether capillary "pull" is with or against gravity; (4) the soil material and the relative density state, as affecting the sizes of the void spaces; and (5) probably other unsuspected aspects of partial saturation.

The degree of saturation now increases under increasing heads due to the compressibility of the air in the void spaces. It is evident that the permeability testing of soils under such conditions is complex and difficult. In order to obtain comparable, representative, and useful test results, the program of testing would have to be well formulated with each series of tests made under a specific set of test conditions, which bracket certain limits only of the phenomena. Many practical problems in soil engineering which involve seepage flow under such conditions, as noted previously, are becoming of importance. In order adequately to analyze such problems, a great deal more will have to be learned about the real nature of such phenomena, and the range of conditions that control in natural situations. Because the permeability characteristics under such conditions are not constant but are essentially time-, gradient-, and per cent saturation - dependent, test conditions would have to be formulated carefully to bracket specific limits and possibilities in each natural situation that are representative and directly applicable with regard to the probable character and range of conditions that may control.

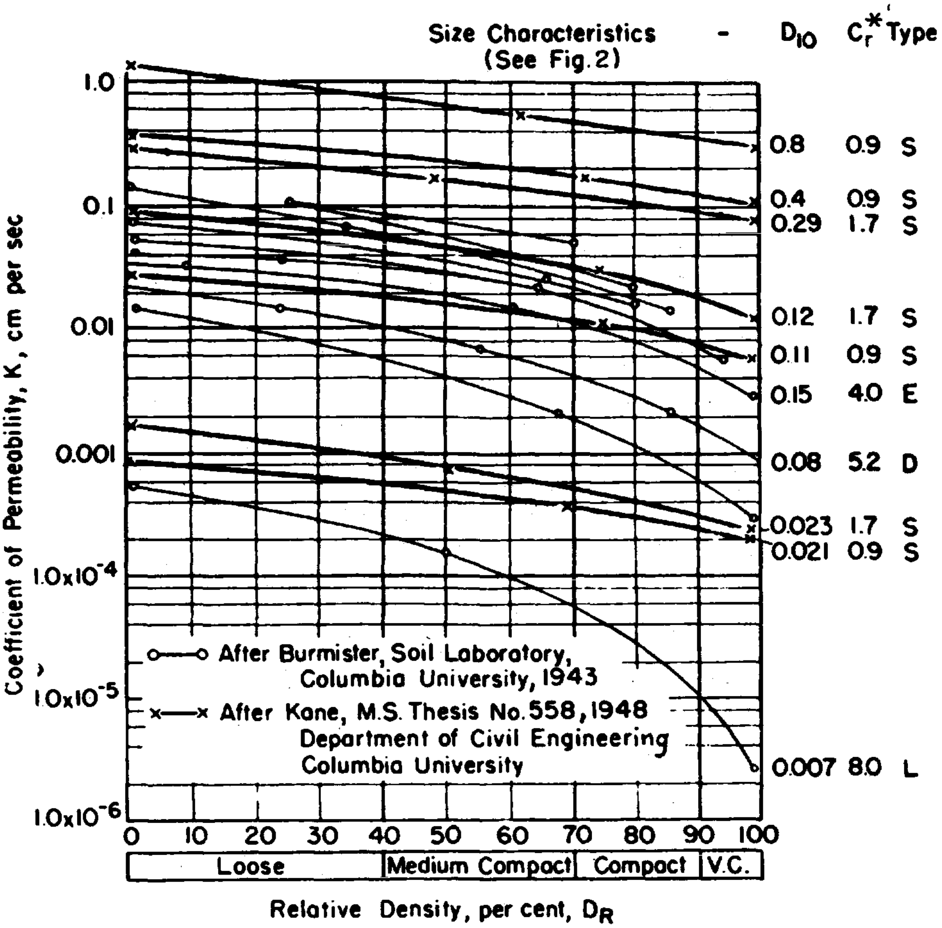


FIG. 5.—Permeability - Relative Density Relationships.

D_{10} = Hazen's effective size.
C_r = range of grain size defined by mean slope.
Type = letter designation of symmetric or asymmetric shape of grain size curves.
NOTE.—See Fig. 2.

The results of such investigations probably could be best established and presented in terms of the kind, degree, and probable range of departures from the Darcy coefficients of permeability as a stable basis for reference for practical purposes in such soil engineering work.

Basic Patterns of Permeability Relations:

On the basis of two series of investigations in 1943 and 1948 for the Darcy type of flow, two basic patterns of permeability relations were established (Figs. 5 and 6). These two basic patterns of permeability - relative density relations were first presented in 1948 as one of five examples to illustrate the importance and practical uses of relative density in soil mechanics, and were discussed very briefly from that viewpoint (5, pp. 1263–1265). They are discussed here from the point of view of permeability phenomena. The basic pattern of permeability - relative density relations is given in Fig. 5, the heavy-line curves bracketing the common range of soil material from "coarse SAND" to "coarse SILT" from the research of Kane in 1948 (7), and the light-line curves covering a wide range of composite gravel-sand-silt soils from the research of 1943. Both series covered the range of relative density from the loose to the intermediate to the dense state. The Darcy coefficients of permeability are plotted vertically on a logarithmic scale to cover the full range of values against relative density on an arithmetic scale to form a consistent pattern of curves with fineness of the soil, as indicated by the values of D_{10} noted in the right margin of Fig. 5. The basic pattern of permeability-D_{10} relations for a constant relative density of 40 per cent was obtained by interpolation from Fig. 5 and is given in Fig. 6. The logarithm of permeability is plotted vertically against the logarithm of D_{10} horizontally to define the reference lines of soil character and the general drainage characteristics for ratings of soils.

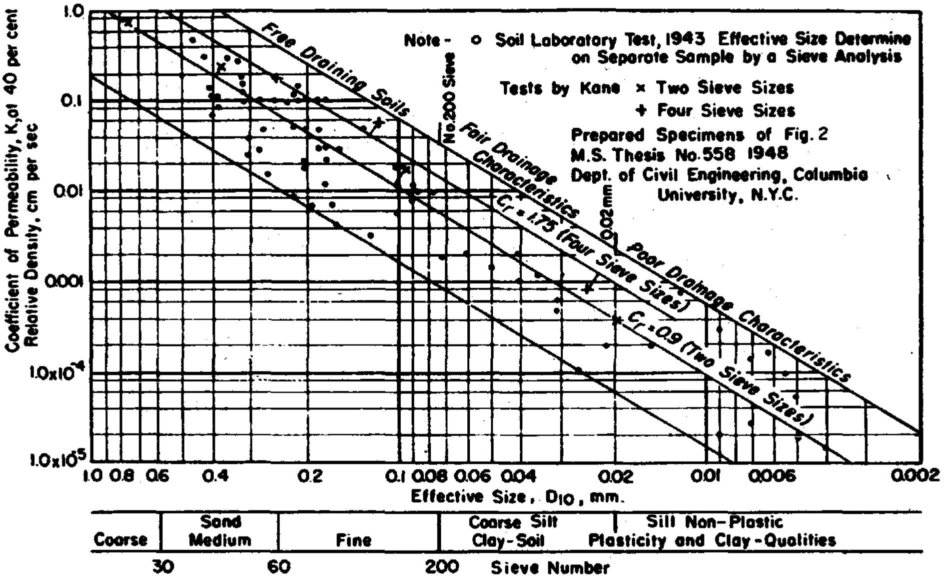


FIG. 6.—Relations Between Permeability and Hazen's Effective Size, D_{10}.
Coefficient of permeability reduced to basis of 40 per cent relative density by Fig. 5.

In order properly to interpret the permeability patterns of Figs. 5 and 6 with regard to the controlling influences of soil character, three distinguishing and significant size characteristics of granular soils are defined on the basis of grain size distribution curves, namely, fineness, range of grain sizes or mean slope of curve, and type of grading or characteristic shape of grain size curve. These three size characteristics of soils are necessary and sufficient to define grain size distributions of granular soils, and they are entirely independent of each other. Studies (6, pp. 18–20) have established the significant fact that grain size distributions of soils are not haphazard chance phenomena but are determined by and are characteristic of each different geological process of soil formation, and that these three characteristics significantly reflect these facts.

Hazen's effective size, D_{10}, has been widely used as an index of general fineness of soils, particularly in permeability phenomena, because of the control of this fine fraction in its clogging effects in the void of the soil. Actually, however, correlations show that D_{50} (50 per cent size) would be more significant in Fig. 6, resulting in a narrower reference band. An index of the range or spread of grain sizes should be representative of the entire grain size curves for all types of soil gradations, not only of the bulk of the soil material but also of the coarse and fine "tails" of the curve with regard to their relative importance and control of behavior. Hazen's uniformity coefficient was found to be too restricted and unrepresentative in character and to be an unsatisfactory basis. The effective range or spread of grain sizes, designated C_r, however, may be defined on a satisfactory basis of the "mean slope" (5, pp. 1266–1267) of the grain size curve, in accordance with common engineering principles. The mean slope is readily determined graphically by using a transparent scale and making the plus and minus areas enclosed between the grain size curve and the mean slope equal and balanced independently for the upper and lower branches and tails of the grain size curve, as illustrated in Fig. 2 for four different types of grain size distributions. Since the vertical intercept is always 100 per cent, the range of grain sizes, C_r, may be conveniently and significantly defined as the number of "coarse," "medium," and "fine" fractions in Fig. 2 intercepted on the horizontal scale between the 100 and 0 per cent terminal points of the mean slope. The mean slope and C_r have a direct correlation for Type-S grain size curves with the "standard deviation" used in statistical analyses.

The type or shape of a grain size curve is an index of the symmetry or of the kind and degree of asymmetry or "skew" in the distribution of grain sizes, which are characteristic of certain geological processes of soil formation. The almost symmetrical Type-S grain size curve is characteristic of the distributions so commonly found in sands and coarse silts which have been formed by the assorting action of sedimentation in flowing water or quiet water, by wave action, and by wind action. When the gravel content of sands exceeds about 10 per cent, the grain size distributions are markedly asymmetric with a predominating "tail" of gravel. The types of common and significant grain size curves are given characteristic letter designations, which are sufficient for most practical purposes of analyses in soil investigations, as shown in Fig. 2 (*b*) (5, pp. 1266–1267).

Certain important and significant facts regarding the controlling influences of the character of the soil material and of soil structure are disclosed by the permeability - relative density and the permeability-D_{10} patterns of Figs. 5 and 6 for the Darcy type of flow. First, there is a generally consistent pattern of decreasing permeability with fineness and decreasing values of D_{10} evident in Fig. 5. This pattern is more clearly defined in the permeability-D_{10} relations in Fig. 6, where permeability values are reduced (interpolated from Fig. 5) to a significant and common 40 per cent relative density basis, in order to obtain an essential and consistent basis for comparison and interpretation and for rating soils with regard to their significant drainage characteristics. Otherwise there can be no proper basis for comparison. The 40 per cent relative density, which is the dividing value between the loose and medium compact states, as noted in Figs. 1 and 5, was chosen as a significant common basis because so many granular soil deposits possess natural relative densities between about 30 and 50 per cent, the lower value being more representative of the finer granular soils and the higher value of the more gravelly soils.

Second, there is a consistent pattern in Fig. 5 of decrease in permeability with increase in relative density. As a basic fact of permeability phenomena, it is evident from the pattern of Fig. 5 that relative density can provide a unified and comprehensive basis for comparing and evaluating permeabilities of different soils deposited or placed in different degrees of compactness.

Third—and of equal importance and significance—this permeability - relative density pattern discloses the controlling influences of the range of grain sizes, C_r, upon the change of permeability with increase in relative density, a fact that is made clearly evident by the use of the more representative and significant definition of this size characteristic. For the narrowest range of grain sizes with C_r equal to 0.9 (spread of two sieves sizes only on the scale at the top of Fig. 2), the soils possess the flattest permeability - relative density curves, that is, the least change in permeability between the loose and dense states, and the curves are almost linear in character. With increase in the range of grain sizes to a value of C_r or 1.7 (spread of four sieve sizes), there is a noticeable increase in the change in permeability with increase in relative density from the loose state toward the dense state, particularly above 70 per cent relative density. Furthermore, it is clearly evident that the respective heavy-line permeability - relative density curves for C_r of 0.9 and 1.7 are essentially parallel to each other for the full range of D_{10} or fineness of these soils. This indicates that the change in permeability with increase in relative density is governed principally by the range of grain sizes, as the only variable quantity between the two sets of curves. With larger values of C_r, the influences of the range of grain sizes on the light-line permeability curves become more pronounced over the full range of relative densities, particularly for values greater than 70 per cent, where there is a marked and characteristic curvature downward toward the 100 per cent relative density. This is due to the more pronounced clogging effects of the finer soil grains in the void spaces of the soil toward the maximum density state, because with increasing range of grain sizes the maximum density also increases markedly (5, pp. 1266–1267). The general steepening of the permeability curves toward the minimum density is due to the more marked particle separating effects of the finer grain sizes on the grain structure in the loose state with wider ranges of grain sizes and C_r greater than 4.

In the permeability-D_{10} relations of Fig. 6, the influences of the range of grain sizes are also evident in defining two reference bands for C_r of 0.9 and 1.7, respectively. For the 40 per cent relative density basis of Fig. 6, which is at the upper limit of the loose state, the reference band for the wider range of grain sizes with C_r of 1.7 lies above that of the narrower range of grain sizes with C_r of 0.9. This increase in permeability is due to the greater particle separating effects of the finer sizes on the grain structure in the loose state as the range of grain sizes increases for soils having the same value of D_{10}. The influences for larger values of C_r could not be evaluated from the light-line curves of the 1943 data, because there was no regular pattern by steps in the grain size distribution of these natural gravelly soils. For the widest range of grain sizes with C_r of 5.2, the points fall below the uppermost reference band. Thus, not only is the permeability pattern with D_{10} made more clear in Fig. 6 by using the common 40 per cent relative density basis, but the important controlling influences of the range of grain sizes of the soil material upon permeability - relative density-D_{10} relations are significantly revealed in Figs. 5 and 6 for practical purposes.

PRACTICAL ASPECTS OF PERMEABILITY INVESTIGATIONS

In investigating drainage, seepage, and stability problems, it is essential to obtain adequate and reliable permeability data that are representative of and will bracket the range of soil character, the range of field relative densities, and the range and character of the conditions that control. For important projects it is advisable to obtain and to test as large a number of undisturbed samples of granular soils as possible for their permeability properties. Test conditions can be set up for each situation by a careful and complete visualization and appraisal of the soil conditions, both environmental and imposed, the nature of the permeability phenomena, and the character of the practical problems involved with regard to their relative dominance and control. Such test conditions, including Nos. 4 to 8 given previously, will yield permeability data having maximum reliability and usefulness.

Where relatively few undisturbed samples can be secured and tested, the test results can be used to test the general validity of Figs. 5 and 6. Then the permeability information obtained from undisturbed sample tests can be reliably supplemented by making two tests on each of a series of soils bracketing the range of soil character in the situation at relative densities of 0 and 70 per cent under proper test conditions, such as Nos. 2 to 8 listed above, in order to define a series of permeability - relative density curves such as given in Fig. 5. By entering these permeability - relative density curves with the bracketing ranges of field relative densities established for these soils, reasonably reliable estimates, bracketing the possibilities in a given situation, can be obtained for the probable range of permeabilities for design purposes. The scope and reliability of Figs. 5 and 6 can thus be extended and broadened. However, the only way to determine reliably the influences of the degree of anisotropy of soils is to make permeability tests on large undisturbed samples in the vertical and horizontal directions in order to establish representative values of K_v and K_h.

In certain cases it may be desirable to obtain preliminary estimates of permeabilities of a large number of soils from grain size distribution curves, using the size characteristics D_{10} and C_r. By estimating a range of permeabilities bracketing certain narrow ranges of similar soils from Fig. 6 at 40 per cent relative density, this range of K_{40} can be inserted in Fig. 5. Permeability - relative density curves, which conform in general slope to the pattern curves of approximately the same range of grain sizes, C_r, can then be interpolated for this range of K_{40}. Estimates of the probable range of permeabilities can then be obtained for any bracketing range of field relative densities for the given soils. The range of relative densities of granular soils may be estimated from an interpretation of the records of the driving resistances of the sampler in blows per foot, preferably in blows per 6 in. (5, pp. 1257–1259, Fig. 4, and Eq 6).

Thus these permeability - relative density - D_{10} patterns provide a powerful and useful tool for supplementing and estimating permeability information in soil investigations. Due to ignorance factors—such as lack of fully adequate and reliable information and lack of full comprehension of the real soil phenomena—a considerable spread in working values may be necessary in order to bracket the probable limits of soil character, of behavior and responses, and of controlling conditions in a particular situation. These working limits should be definitely and carefully established by a reasoned consideration of known conditions that control and of all reasonable possibilities that may be inherent in the situation. The influences of this spread in working values on the adequacy and reliability of the outcome and practical applications of an investigation should be carefully and completely assayed and evaluated, particularly as to which limiting values and combinations are the more unfavorable and most likely to govern in the particular situation. This is not generalization as commonly used in present practices. The major problems in soil engineering are to remove the ignorance factors from investigations by more adequate and reliable soil test data and thereby to reduce the spread in working values, but insuring a real and adequate but not excessive margin of safety in design and construction of structures.

By a reasoned and consistent application of controlled test methods and by a careful and complete appraisal and evaluation of each situation, the favorable aspects can be recognized and full advantage can be taken of them. The possibilities of improving conditions with regard to any adverse aspects can be fully explored and planned for in order to avoid construction difficulties and hazards. In any case they can be recognized and be taken into account fully in the planning and design by *fixing in advance the safe limits and time sequences* for design and construction of structures. This is learning how to work with nature by *fitting* foundation and earthwork design and construction methods to actual conditions, in order to achieve (1) closer agreement between predictions of soil behavior and the actual observed phenomena, and (2) higher standards of excellence, greater economy, and more enduring structures.

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(7) H. Kane, "Investigation of the Permeability Characteristics of Sands," Thesis No. 558 for Degree of Master of Science, Department of Civil Engineering, Columbia University, New York, N. Y., June, 1948. (Not published.)

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DISCUSSION

MR. E. D'APPOLONIA.[1]—The reliability of the results related to relative density depends on two factors: (*a*) the experimental errors made in the determination of the natural, minimum, and maximum dry unit weights, and (*b*) the procedure used to determine these unit weights. The first factor can be minimized and kept reasonably constant by the experimenter. However, the procedures used to determine minimum and maximum unit weights vary with each experimenter, and widely different results for the same soil are obtained. It has been shown[2] that differences in unit weights of 2 per cent will mean a 10 per cent difference in relative density.

The work of any one investigator may be consistent. But similar work, conducted by another person using different testing procedures, would not give the correlations between permeability and relative density discussed in this paper. Before the results of research of this nature can be utilized and properly compared with other similar work, it will be necessary to standardize the testing procedures for the determination of minimum and maximum densities.

MR. DONALD M. BURMISTER (*author's closure*).—There are two important aspects of permeability testing of granular soils that should be given further consideration: the first deals with the reliability of test results based on relative density as a common unifying argument, and the second deals with the practical problems of adequately rating large numbers of soils with regard to their potential permeabilities.

The remarks of D'Appolonia with regard to the first aspect are well taken. There should be a recognized method for determining the maximum and minimum densities as the 100 and 0 per cent laboratory references for relative density. A method of test was suggested by the writer in "Procedures for Testing Soils," (**8**)[3]. This method has been used and proved in testing granular soils in direct shear, triaxial compression, and permeability tests over the past 15 yr as a satisfactory basis which yields results reproducible within ±1.0 per cent. Due to the nature of the defining relative density equation, Eq 2, with differences in both numerator and denominator, errors in the determinations of any of the three quantities are considerably magnified. From experience in the use of relative density, it is believed that errors in the determination of the three quantities within 1.0 per cent will not be reflected in errors in relative density out of proportion to its great practical usefulness in soil mechanics (**2**).

The maximum and minimum density limits, however, are more than just reproducible values; they have a definite physical significance, namely, they are

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[1] Associate Professor in Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

[2] Elio D'Appolonia, "Loose Sands—Their Compaction by Vibroflotation," Symposium on Dynamic Testing of Soils, Am. Soc. Testing Mats., p. 138 (1954). (Issued as separate publication *ASTM STP No. 156*.)

[3] The boldface numbers in parentheses refer to the list of references appended to the paper and continued at the end of this discussion.

characteristic limiting values in which granular soils can exist in natural deposits. As such, they fix the limits of natural granular soil behavior and are used as basic references for shearing, consolidation, compaction, as well as for permeability phenomena for granular soils. Seldom in 20 yr of experience have natural granular soil deposits been found in a more loose state than the minimum density reference, excepting possibly loess deposits, which are formed under rather peculiar geological conditions; also, seldom have granular soils in natural deposits been found in a more dense state than the maximum density reference, except under very unusual geological conditions. These characteristic density limits are not haphazard phenomena but are definitely determined by grain size distribution and grain shape characteristics of granular soils (2, pp. 18–19). The natural relative densities of granular soils are determined by geological processes of soil deposit formation and by the accompanying grain size distribution and grain shape characteristics, as the inherent characteristics of the granular soil material.

Research by Mr. Jolls (9) has developed a number of significant facts concerning the maximum density reference. First, the maximum density obtained by a variable speed vibrator near resonant frequencies was about 2 lb per cu ft higher than that obtained by the "vibro-tool" method used in the suggested procedure (8). This value represents about the maximum attainable by any means and therefore is a suitable maximum density reference. Second, by varying the thickness of the vibrated successive layers, it was shown that for practical purposes, the specified "½-in." thick layers achieved consistent maximum densities and that for ¼-in. thick layers, the maximum density was only ½ to ¾ lb per cu ft higher. For practical purposes, the suggested method yields a stable and representative maximum density reference, to which may be added about 2 lb per cu ft for the probable true value. At least the suggested method is much simpler and is more readily performed.

In a research project undertaken by Mr. Cartwright (10), an attempt was made to duplicate natural minimum densities attained by sedimentation processes in quiet water conditions. This was thought to be the more probable minimum density condition in natural soil deposits. Flowing water, wave action, and wind action all tend naturally to produce higher natural densities than the 0 per cent relative density reference. A special set of test conditions was set up for this research to produce a very uniform loose layer of granular soil with no layering or normal segregation effects of the fine and coarse fractions by the sedimentation process. This was accomplished by allowing a uniform, continuously fed cloud of sand to settle by sedimentation to the bottom of a special cylinder of lucite that could be separated at a distance of 2 in. from its bottom. After measurement of the average thickness of the sand layer deposited, the soil was evacuated to reveal the degree of layering present.

It was found, as evidenced by light and dark streaks, that only a thin layer of a few millimeters in thickness at the top and bottom showed segregation and layering effects. In six of eight sands bracketing the grain size limits of common sands, a higher loose density was obtained than in the suggested method by spreading dry sand in ½-in. layers by a large spout funnel. In the two coarser soils of the eight soils tested, the density was found to be lower than the minimum density reference. These density values

TABLE II.—TENTATIVE CRITERIA FOR RATING SOILS WITH REGARD TO DRAINAGE, CAPILLARITY, AND FROST-HEAVING CHARACTERISTICS. CRITERIA FOR SOILS IN A LOOSE TO MEDIUM COMPACT STATE.

Fineness Identification	*"trace fine Sand"*	*"trace Silt"*	*"little Silt"* (coarse and fine)	*"some fine Silt"* *"little Clayey Silt"* Fissured Clay-Soils	*"some Clayey Silt"* Clay-Soils dominating
Approx. Effective Size, D_{10} mm.	0.4 0.2	0.2 0.074	0.074 0.02	0.02 0.01	0.01
Drainage	free drainage under gravity excellent	drainable by gravity good	drainable good to fair	drains slowly fair to poor	poor to impervious
Approx. range k, cm per sec 0.5 DR—40 per cent ←	0.2 0.10 —Deep Wells	0.04 0.02 ←—Well points successful	0.006 0.001	0.0004 | 0.0002 ——→	0.0001
Capillarity	negligible	slight	moderate	moderate to high	high
Approx. rise, H_c, ft: DR—0 per cent DR—40 per cent	 0.5 0.5	 1.5 1.0	 7.0 3.0	 15.0 10.0	 25.0
Frost-Heaving Susceptibility	non-frost heaving	slight	moderate to objectionable	objectionable	objectionable to moderate
Ground water within 6 ft. or $H_c/2$			←Doubtful→	←Drainage→ and/or Protective Installations required	
Station Depth Identification:					
13 1–6 yb mfS, l·f$\bar{S}$, t·mfG			—		
14 3–9 b cmS, t·fG	—				
15 2–8 yb mfS, t^+·cf$\bar{s}$, t·fG		—			
16 16–22 yb mfS, t^+·cf$\bar{s}$, l^+·mfG		—			
17 15–21 yb mfS, s^+·cf$\bar{s}$					
18 4–10 yb mfS, l·cf$\bar{s}$			—		
19 2–8 b mfS	—				
20 6–12 b mfS, t:cf$\bar{s}$		—			
21 8–12 b mfS, t^+·cf$\bar{s}$		—			
22 8–12 b mfS, t·mfG	—				
23 8–12 b cmS. t^-·cf$\bar{s}$	—				
24 1–7 b cmS, l·mfG	—				
25 1–6 b mfS	—				
P.R. Classification A-3		—	—		
Casagrande Classification SP	—	—	—		
Corp of Engineers Uniform Classification—Sand	—	—	—		

SYMBOL FORM OF IDENTIFICATION NAME (6)
Major Components (first):
G, S, $\bar{S}$
Minor Components: G, s, $\bar{s}$
Fractions of Components:
coarse to medium, cm
coarse to fine, cf
medium to fine, mf
fine, f
Proportion Terms:
and a· 35–50 per cent
some s· 20–35 per cent
little l· 10–20 per cent
trace t· 1–10 per cent
+ and − nearer upper or lower limit.
Color: brown, b; yellow-brown, yb; gray, g.

Potential Drainage, Capillarity, and Frost-Heaving Ratings of Subgrade Soils to Depths of 3 to 5 ft below proposed Subgrade Elevation.

Controlling Conditions in Natural Situations:

1. Identification and character of soils, geological origin and processes of formation.
2. Climatic conditions—seasonal precipitation, temperature and evaporation conditions, normal depth of freezing, average number of days below freezing temperature consecutively.
3. Soil Profile—type and pedological character, horizons, discontinuities, depth to rock. Stratified deposits—sequence of layering and character of soils, thickness of layers, uniformity or variability in lateral extent, lenticular strata, depth to rock.
4. Surface and subsurface drainage conditions, ground slopes, seepage zones, ground water level and probable seasonal variations at critical times.
5. Granular soils—natural compactness and coherence due to silt films at the grain contacts. Clay-soils—natural consistency, structure and state of aggregation, fissured character and fragmentary structure as controlling drainage conditions.
6. Natural moisture content, degree of saturation of natural and compacted soils after normal capillary saturation under conditions imposed by structure at critical times.
7. Relative permeabilities of different horizons or strata in natural deposits, or soils compacted in thin layers in subgrades and embankments.

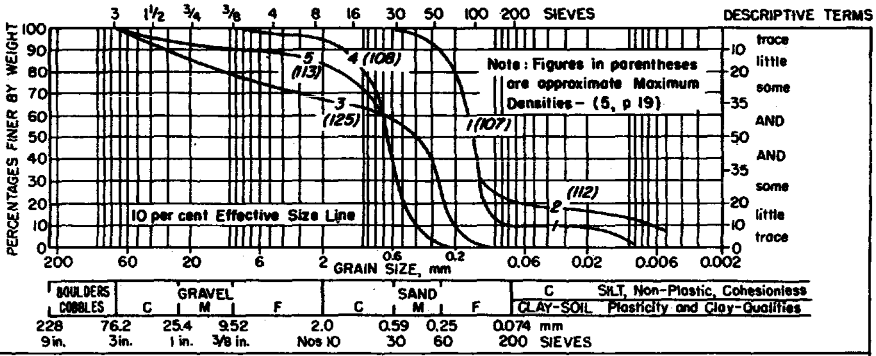
were from 1 to 2 lb per cu ft higher or lower than the minimum density reference. This research method was a difficult and time consuming one. Therefore, as a practical method, which has physical significance, it is believed that the suggested method of spreading the dry soil, or using test condition No. 4 listed for the permeability test to avoid or to eliminate segregation effects, would yield sufficiently representative and significant minimum density references for different granular soils for relative density determinations.

The second aspect of practical importance and use in soil engineering is that of adequately rating granular soils with regard to their permeabilities in natural deposits and in compacted embankments. This is a very practical problem, because, in general, only a few permeability tests can usually be made on a given project within time and economic limits, but information on the potential permeabilities of all soil samples obtained would be essential in order to obtain clear and definite conceptions and information on the range and kind of permeability problems to be encountered. The idea and conception of rating soils have come more prominently into use in the past few years. A rating of soils with regard to the fundamental behavior characteristics of soils is more specific, significant, and practically useful than is attainable by the broad generalizations of classification systems so commonly used at present. The basic question is more than that of the degree of generalization permissible in soil work and of how simply and easily applications can be made, but rather is one of attitudes and conceptions regarding the place and meaning in soil engineering of the two basic analyses of identification of soils, as factual information of first importance, and of rating of soils, as necessary and important interpretative information needed for making practical and effective applications. This is in contrast to simplified "all-purpose" classifications with their broad generalizations, confused mixing of factual and interpretative information, and their fixed and inflexible conceptions of adequacy. The shortcomings and limitations of classification have been discussed elsewhere (6, pp. 4–8).

A rating of granular soils with regard to permeability, capillarity, and frost-heaving susceptibility is given in Table II to illustrate the potential value, possibilities, and practical usefulness of ratings. A rating to be of real practical value must be based on specific criteria developed by experience or through research and investigation to define the limits of the ratings. A rating is essentially interpretative information. In the very nature of things, the criteria and the limits of the ratings must be brought up to date at frequent intervals as knowledge of soil phenomena increases. Furthermore the adequacy of ratings should not be permitted to become an inflexible and fixed idea.

The primary basis for a rating is an adequate identification of soils that provides factual information on the inherent characteristics of the soil material and of the soil structure that govern soil behavior. Criteria derived from the identifications of soils, either the identified finest fraction of a component from visual examinations and identifications of the soils in the field or laboratory, or the value of the effective size, D_{10}, from grain size analyses, then permit the rating of soils with regard, for example, to drainage characteristics and permeability through the permeability-D_{10} relations of Fig. 6 of my paper on a common 40 per cent relative density basis. Such a specific rating provides a systematic basis for making effective and definite interpretations for practical purposes. Still more specific information can be obtained directly from Fig. 6 if it is required for drainage investigations.

It should be noted in Table II that capillarity or potential capillary rise depends essentially upon the same criteria as does permeability, but varies as to magnitude in exactly the reverse order. Frost-heaving susceptibility depends upon both drainage and capillarity characteristics of soils with the region of principal susceptibility and maximum objectionable action in the silt sizes, where permeability is not too low nor capillarity too high. Ratings of soils should never be made apart from the criteria of the ratings, but should be made a part of the rating table, as is done in Table II; otherwise, the ratings lose their real significance and usefulness. The ratings also contain additional practical and useful information regarding the limits of applicability of certain methods of draining soils, which depend upon the range of permeabilities for most successful action, as established by experience. In the case of capillarity, the approximate effective heights of capillary rise are given for the common range of relative densities. Such useful information can be amplified and extended with increase in understanding of soil phenomena. These are distinct advantages of the rating conception.



Classification	Class Limits	Curve	Identification of Possible Coarse and Fine Limits
Public Roads A-3	51 per cent passing No. 40 Sieve Non-Plastic	3	"coarse to medium SAND, and + coarse to fine Gravel."
or H.R.B.	10 per cent passing No. 200 Sieve	1	"medium to fine + SAND, trace + fine Silt."
Textural Classification – Sand	80-100 per cent Sand, 0-20 per cent Silt	4	"coarse to medium SAND, trace fine Gravel."
chart, P.R.	0-20 per cent Clay	1-2	"medium to fine + SAND, little Clayey Silt."
Casagrande – SP	Poorly graded Sand, little or	5	"coarse to medium SAND, little medium to fine Gravel."
Classification	no fines	1	"medium to fine + SAND, trace + fine Silt.
Corps of Engineers – Sand	90-100 per cent Sand, 0-15 per cent Gravel	5	"coarse to medium SAND, little medium to fine Gravel."
Uniform Classification	0-10 per cent Silt or Clay	1	"medium to fine + SAND, trace + Clayey Silt."

FIG. 7.—Possible Coarse and Fine Grain Size Curve Limits of Classifications for Purposes of Rating of Soils with Regard to Drainage Characteristics.

In order to further illustrate the practical value and usefulness of ratings of soils with regard to their drainage and associated characteristics, ratings have been made in Table II of subgrade soils in a cut section of a highway. The relatively narrow spread in the limits of the rating for each soil is to be especially noted, being determined either by the possible spread in D_{10} or by the identification of the finest soil fraction (coarse, medium, or fine) of the finest component (sand or silt). A major problem in soil engineering is consistently to reduce the spread in the working values of soil properties and ratings due to ignorance factors by more adequate identifications, testing, and analyses of soils. The comparative spread in the D_{10} values inherent in classification methods is illustrated in Fig. 7, where the possible grain size curve limits are given for the class of soils in which most of the soils rated in Table II fall. The degree of generalization of classifications now becomes evident. In contrast, the direct use of grain size curve by sieve analyses or the accurate and complete identification of soils greatly reduces the spread in the comparative ratings in Table II. It should be evident that more adequate and specific information is obtainable by the identification-rating method of analysis than by classification methods.

Furthermore an appraisal of the controlling conditions in the situation, such as listed in Table II, will determine what significance should be attached to these ratings of potential behavior in a particular situation. Estimates can then be made of the probable actual susceptibility and behavior of soils, as a basis for determining what treatment is necessary to improve the qualities and responses of the soils, or what protective measures, such as installations of drains, base courses, or frost-heaving protection, may be required. Depending upon the proximity of ground water to the subgrade level during the season of highest groundwater level, drainage installations and protective measures should be installed in the doubtful regions and certainly in the regions of poorer drainage and objectionable frost-heaving characteristics. Thus potential drainage and frost heaving become actual problems if the normal depth of freezing exceeds $1\frac{1}{2}$ ft, or the ground water is within about 6 ft of the subgrade level or about one half the height of capillary rise of the soils, whichever factor controls. The rating provides a tangible and practical basis for judging each situation, as to its kind and range of drainage and frost-heaving problems, and as to the types of installations and protective measures that are likely to be most effective.

The ratings also provide a basis for comparison of expected performance of any section of highway, for example, with the actual observed performance in a condition survey after a year or two of operation in service. Thus it would be possible to build up an authoritative body of valuable information. The ratings, interpretations, and appraisals of probable actual performance, and the judgments regarding the requirements for drainage and protective installations could be checked, modified, and extended to cover actual conditions observed in the field under different climatic and soil conditions. Thus ratings could be made a powerful and practical tool for the soil engineer in making the most effective use of knowledge and experience and for improving conditions and practices in soil engineering.

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WATER MOVEMENT THROUGH POROUS HYDROPHILIC SYSTEMS UNDER CAPILLARY, ELECTRICAL, AND THERMAL POTENTIALS

By Hans F. Winterkorn[1]

Synopsis

The importance of the interaction between solid internal surface and pore water for water transmission under hydraulic gradients and electrical and thermal potentials is pointed out. This interaction results in the establishment of a restrained water phase possessing characteristic mechanical, thermal, and electrical properties. If only a hydraulic gradient is established, the restraint can be expressed as a volume factor which, however, is also a function of the hydraulic gradient, especially at high values of the latter. The characteristic thermal and electrical properties of the interphase are the necessary conditions for thermo- and electro-osmotic flow. The basic theories of these phenomena are presented in a simple manner. The considerations presented and the equations derived hold only for such soils or similar systems that do not possess a significant gas phase.

The movement of liquids through porous solid systems depends on the proportion and geometrical characteristics of the pore space, the physical properties of the liquid, and the interaction between liquid and solid internal surface, as well as on the energy potential in the direction of flow, the cross-section considered, and the time allowed. The first three factors are of intrinsic physical importance because they represent properties inherent in the system. The last two can be varied at will and are, therefore, not characteristic. The energy potential factor may be varied at will but also can and often does have a definite physical importance, especially when the intensity of the liquid-solid interaction varies with distance from the pore walls.

Water Transmission Under Hydrostatic Pressures

Systems possessing pores of sufficient size to allow the moving fluid to act in accordance with its statistical or mass properties and endowed with geometrical properties that result in laminar flow are usually treated as analogous to systems of capillaries of uniform diameter. The latter simplified systems must, of course, possess the total porosity and the same liquid transmission properties as the prototypes. In establishing the equivalent capillary system, it is customary to use the Poiseuille-Hagenbach equation:

$$v = \frac{\pi r^4}{8\eta} \frac{(p_1 - p_2)}{l} t \quad \ldots\ldots\ldots (1)$$

where:

v = volume of liquid in cubic centimeters,

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η = coefficient of viscosity in poises,
r = inside radius of the capillary tube in centimeters,
$p_1 - p_2$ = pressure head over the distance l,
l = length of tube, and
t = time in seconds.

If both sides of the equation are multiplied by n_c, the number of capillaries per square centimeter of cross-section normal to the direction of flow, and divided by the time t, setting at the same time the gradient $\frac{(p_1 - p_2)}{l}$ equal to unity, then the following equation is obtained for the water transmission per unit gradient per second per square centimeter:

$$\frac{vn_c}{t} = k = \frac{\pi r^4 n_c}{8\eta} \dots\dots\dots (2)$$

If the porosity of the system, n, is introduced and knowing that with uniform and constant cross-section of the capillary tubes the product $r^2\pi \times n_c = n$, and also that $r^2 = \frac{n}{n_c\pi}$, Eq 2 can be written as follows:

$$k = \frac{n^2}{8\pi n_c \eta} \dots\dots\dots\dots (3)$$

Since n^2 and n_c are constants for a specific system, $\frac{n^2}{8n_c\pi} = k_0$ can be introduced and an equation for the transmission constant, k, thus be obtained:

$$k = k_0 \frac{1}{\eta} \dots\dots\dots\dots\dots (4)$$

In this equation one factor is representative of the properties of the solid and the other of those of the liquid. This equation, however, can hold only for systems in which the solid-liquid interaction is exactly as assumed in the derivation of the Poiseuille-Hagenbach equation, not more and not less. This assumption is that the solid surface holds the first contacting layer of liquid molecules so strongly that they are fixed and immovable. If the fixation extends over a large number of molecular layers, the Poiseuille equation and Eq 4 derived from it are no longer strictly valid; nor are they strictly valid in the case in which the first contacting layer of liquid molecules are not fixed and are, therefore, subject to slippage. In most actual soils, the fixing effect extends to a very considerable distance; on the other hand, nonfixing and slippage is observed in certain resin stabilized soils. This paper will be limited to porous systems in which there exists a considerable interaction between the solid and liquid phases, chief representatives of which are systems composed of clay soils and water.

The author had the privilege on two occasions within the last two years to discuss the unique properties of the water substance that enable it to interact strongly and in various ways with solid mineral surfaces capable of ionic exchange (**1**, **2**).[2] As a matter of fact, most physical tests on cohesive soils, including those for hygroscopicity, consistency, consolidation, and shear characteristics, probe into the condition of the water, and actually are tests for the interaction between solid surface and water molecules. With respect to this interaction, the available experimental evidence renders it very probable that:

1. Below the hygroscopic moisture equivalent, the water substance is dissolved as the monomer (H_2O) in the solid mineral surfaces.
2. Between the hygroscopic moisture to approximately the plastic limit, the water substance possesses properties similar to those of a melt.

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[2] The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 35.

3. Above the plastic limit, the water phase behaves more and more as an ionic solution but has peculiar properties because of the fixation of the negative charges on and in the solid surface (2).

Depending upon the particular system and the intended purpose, the study of the water-solid interaction may be focused on one, several, or all of its immediate consequences. These are:

1. Water fixation reduces the volume available for viscous flow.
2. The highly electrical character of the interaction phase indicates use of electrical potentials as tools for studying the condition of water in such systems and as practical tools for drainage (1).
3. The fixation of the water molecules decreases their capacity of utilizing kinetic energy. As a consequence, part of the heat content possessed by free water must be given off on occasion of the fixation of the water molecules and becomes evident as heat of wetting. This and the negative temperature coefficient of the heat of wetting indicate use of thermal potentials as scientific and engineering tools.
4. The interaction between thermal and electrical molecular phenomena heralds the existence and potential usefulness of thermoelectric effects (2).

For normal permeability studies in saturated flow, the problem of liquid-solid interaction can be scientifically reduced to a consideration of the effect of the fixed water volume. If the number of equivalent capillaries per cross-section could be kept constant while changing the porosity, *n*, then the transmission coefficient, *k*, should be directly proportional to the square of the porosity. This condition is more or less fulfilled if the permeability is calculated from consolidation data as long as the applied pressures are too low to cause considerable plastic flow of the soil mass itself. Data obtained in this way by Winterkorn and Moorman (3) on homoionic Putnam soils are plotted in Fig. 1 against the square of the porosity. This figure shows that essentially a straight-line relationship exists between the permeabilities, at the higher porosity and lower pressure values, and the square of the porosity. Continuation of the straight[3] line through the abscissa leads to an intercept which physically has the meaning of the square of that portion of the liquid volume that has been fixed by the wall influence. However, since the degree of fixation is an inverse power function of the distance from the wall, the volume of immovable "water" decreases with increasing hydrostatic pressure. The data

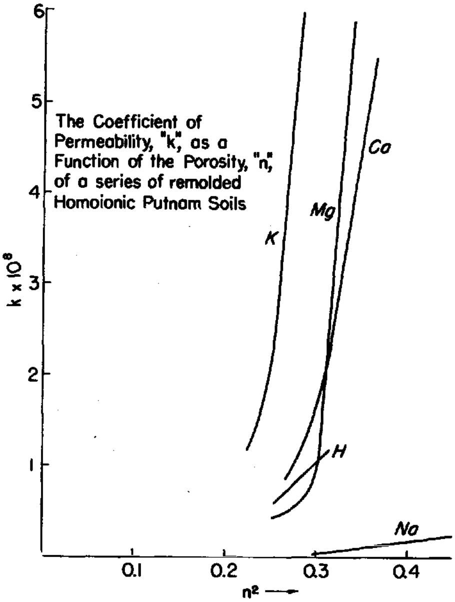

FIG. 1.—The Coefficient of Permeability, *k*, as a Function of the Porosity, *n*, of a Series of Remolded Homoionic Putnam Soils.

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[3] If the thickness of the fixed water layer remains constant while the porosity and pore sizes change, then the theoretical line obtained by Eq. 1 will not be entirely straight. There exist, however, good theoretical reasons backed by experimental evidence which indicate that the thickness of the fixed water layer is a function of the curvature of the solid surface to which it is attached. See paper by author on "Studies on the Surface Behavior of Bentonites and Clays," *Soil Science*, Vol. 41, No. 1 (1936). Undoubtedly, in the actual phenomenon, there are still other modifying factors involved.

TABLE I.—INTENSITY OF WATER FIXATION FORCES, KG PER SQ CM (5).

First molecular layer	25 000
Hygroscopic water	50
Permanent wilting point	12.5
Wilting point (dead water)	6.25
Vacuum moisture equivalent	0.55

TABLE II.—HYGROSCOPICITY, W_H, OF VARIOUS SOILS AND SILICATES.

HYGROSCOPICITY OF DIFFERENT SOIL TYPES, G H_2O PER 100 G SOIL (6)

Soil	Grain Size, mm	W_H	Soil	W_H
Quartz	2–1	0.055	Fine quartz sand	0.03
Sand	1–0.5	0.057	Sandy soil (Kummo)	1.06
	0.5–0.25	0.085	Loamy soil	1.40
	0.25–0.17	0.101	Sandy loam	2.09
	0.17–0.11	0.131	Silty loam	3.00
	0.11–0.07	0.168	Clay loam	6.54
	0.07–0.01	0.203	Low moor	18.42
			Heavy clay (Java)	23.81

HYGROSCOPICITY OF DIFFERENT ALUMINO SILICATES (7)

Mineral	W_H
H-Montmorillonite	32.7
H-Putnam clay	25.5
H-Davidson clay	21.4
H-Permutite	28.2

plotted in Fig. 1 were obtained by using in each case pressures from 1 to 8 atmos, the lower ones for the high porosity values and the higher ones for the low porosity values. Since plastic flow at the high pressures undoubtedly had some effect on the number of equivalent capillaries in these systems, the curved parts or lower ends of the k-lines are influenced by the plastic properties of both the entire system and the adsorbed water.

Including the volume effect, Eq 3 can be written for the higher porosities and lower pressure gradients as:

$$k = C_1 \frac{n^2}{\eta} - C_2 \quad \text{.........(5)}$$

The intensity of the fixation forces for different water conditions are indicated in Table I. The amounts of water held at the different stages characterized in Table I are functions mainly of the type and proportion of clay minerals and organic matter, and of the type and number of exchangeable cations (4). This is illustrated in Table II. The minimal water capacity of a soil, which is defined as the amount of water in grams held against gravity by the molecular (electrical field) forces of the particles of 100-g soil, can be calculated in first approximation from the hygroscopicity:

$$C_{\min} = 4.5W_H$$

This minimum water capacity is independent of the particle arrangement and of the menisci that are functions of particle shape and arrangement. Actual minimal water capacities of soils with different clay contents are shown in Table III.

In accordance with the experimental and theoretical evidence presented, the transmission of water through saturated hydrophilic systems can be expressed by an equation of the type of Eq 5. It must be understood, of course, that both the liquid viscosity, η, and the fixed volume function, C_2, decrease with increasing temperature. The constant, C_1, contains among other items the number of equivalent capillaries per cross-section, n_c. For most actual cases, n_c remains practically constant. The constants of a particular system are best obtained from experimental data plotted as in Fig. 1.

WATER MOVEMENT UNDER CAPILLARY SUCTION POTENTIALS

The volume of the fixed water plays a rôle not only when water is moved as a result of differences in hydraulic pressure but also when movement is the result of capillary and physicochemical suction forces. This became evident in previous studies of the problem of water attack on dry cohesive soil systems (8) and of water accumulation underneath pavements (9). These problems, therefore need not be treated here. However, one simple aspect of capillary rise appears worthy of mention. The simplest treatment of capillary rise is based on the force equilibrium equation applied to the condition at the end of the rise, namely:

$$\cos \alpha \, 2\pi r T_F = r^2 \pi h \gamma g$$

where:

α = angle of wetting between liquid and wall, in the case of water and soil usually assumed as zero,
r = radius of capillary,
T_F = surface tension of water,
h = height of capillary rise,
γ = density of water, and
g = gravity constant.

Making the usual assumption of a zero wetting angle, the following is obtained:

$$h = \frac{2T_F}{r\gamma g}$$

Now let us consider the problem from an energy point of view. Assume that the internal surface of the capillary has adsorbed its minimal water capacity and possesses essentially the surface energy (T_E × area) of an equal area of free water. The water of the reservoir is restrained from moving immediately on insertion of the capillary; it is released only after a zero wetting angle has formed at the inserted end of the capillary. Then the restraint is removed and the water permitted to rise in the capillary. After the rise has terminated, the meniscus with the zero wetting angle has been translocated by the height h. In addition, an amount of water $r^2\pi h$ of unit weight γ has been moved against the gravity g over an average distance of $\frac{h}{2}$; the free surface energy $2r\pi h T_E$ has been used up:

$$\text{Loss of free energy: } 2r\pi h T_E$$

$$\text{Gain of potential energy: } r^2\pi\gamma g \frac{h^2}{2}$$

TABLE III.—HYGROSCOPICITY AND MINIMAL WATER CAPACITY OF DIFFERENT SOILS.

Soil Type	Provenience	Size Composition, per cent					
		Sand		Silt	Clay	W_H	Minimal Water Capacity
		Coarse	Fine				
Sand	Greenlo, Holland	69.6	28.0	1.0	1.4	0.8	7.3
Moraine sand	Barlo, Holland	66.8	24.1	6.4	2.7	1.6	15.6
Loamy sand	Bahri, Egypt	57.3	25.7	8.8	9.0	2.2	16.3
Loam	Hedel, Holland	6.1	51.2	21.4	21.3	4.7	30.6
Loam	Onderdjik, Holland	4.6	29.8	27.9	37.7	7.2	59.3
Clay	Kafr Dimetun, Egypt	1.6	6.2	20.0	72.2	14.4	66.1

Since T_E is numerically equal to T_F and since the force equation holds true, only one half of the free energy involved in the process has been utilized. This equipartition of energy appears to be of fundamental importance and to hold true also in cases where the surface energy lost is, as in initially dry soils, considerably greater than that of an equal surface of water.

EFFECT OF TEMPERATURE POTENTIALS

At the beginning of this paper, it was pointed out that moist soil systems possess mechanical, thermal, and electrical properties. However, by keeping the temperature of the system constant, it is possible to treat moisture transmission under hydrostatic pressure potentials as a problem solely of fluid mechanics, reducing the interaction between solid surface and water molecules to a correction factor with respect to the liquid volume that takes part in the transmission. An analogous approach is possible in the case of thermal potentials applied to mineral-water systems, having no significant gas phase, in which the water is present in such proportions that even the molecules most distant from the solid surface are still under a certain, though relatively low, "fixation." It has been pointed out that such a restraint decreases the kinetic heat energy that can be stored by the water molecules and results in heat of wetting and hydration when liquid water molecules are placed in contact with dry, solid soil constituents.

In soils this heat of hydration has a negative temperature coefficient. If, therefore, a soil without significant air voids but possessing a uniform moisture content, in the above specified state of restraint (best in the vicinity of the plastic limit), is subjected to a temperature gradient, the latter will establish a difference in the specific heat and therefore in the total heat capacity between portions of the soil system that differ in temperature. If for a certain volume element at the higher temperature T_2, the total heat content:

$$\int_{T=0}^{T=T_2} q\,dt = U_2$$

and at the lower temperature T_1, the total heat content:

$$\int_{T=0}^{T=T_1} q\,dt = U_1$$

hen a certain amount of heat $U_2 - U_1 = Q$ would become available by transferring the volume element from T_2 to T_1, neglecting work involved in volume change. This heat is akin to a heat of fusion. This Q has an entropy of $\frac{Q}{T_2}$ at the higher temperature and one of $\frac{Q}{T_1}$ at the lower temperature; according to the second law of thermodynamics, the entropy of a system tends to decrease. Two physical possibilities for such decrease are available since the system can transmit both heat and water. Considering the water transmission as the predominant one for the moisture conditions envisaged, it can be stated in accordance with the second law of thermodynamics that a maximum amount of free energy, W_{max}, is available for the transfer of liquid from the location having the temperature T_2 to that having the temperature T_1:

$$W_{max} = \frac{(U_2 - U_1)(T_2 - T_1)}{T_2}$$

or:

$$dW_{max} = \frac{Q\,dt}{T}$$

If Q represents the latent heat involved in the change of restraint of 1 g of water incident to a temperature change of dt, then $W_{\max}$ is the free energy available for moving 1 g of water from a location at (T + dT) deg to one at T deg. The quotient of a free energy and the volume with which it is associated represents a pressure or a suction. Using the metric system and taking advantage of the fact that in this system the weight of a unit volume (1 cu cm) of water equals unity (1 g), the following is obtained for the maximum pressure or suction:

$$\mathrm{d}p_{\max} = \frac{\mathrm{d}W_{\max}}{1 \text{ cu cm}} = \frac{Q}{1}\left(\frac{\mathrm{d}T}{T}\right)$$

This suction value can be inserted into any Darcy type formula:

$$v = \frac{\mathrm{d}pkt}{\mathrm{d}l} = \frac{Q\,\mathrm{d}T}{T\,\mathrm{d}l}\,k_{\mathrm{D}}t \quad \ldots\ldots\ldots (6)$$

where:

v = volume of liquid transmitted in time t from location with temperature (T + dT) to one with temperature T,

k_{D} = Darcy coefficient of permeability in cgs system,

dl = distance in which temperature falls by dT, and

t = time in seconds.

If a temperature gradient, $\frac{\mathrm{d}T}{\mathrm{d}l} = 1$, is employed, and both sides of Eq 6 are divided by t, a thermo-osmotic transmission coefficient is obtained:

$$k_{\mathrm{T}} = \frac{v}{t} = \left(\frac{Q}{T}\right)k_{\mathrm{D}} \quad \ldots\ldots\ldots (7)$$

If k_{D} is known, then from an experimental determination of k_{T}, the value Q may be determined. It should be emphasized that in the analyzed process only a shift or rearrangement of water in the system is being dealt with, and this comes to an end when the entropy of the entire system has reached its minimum value for the prevailing conditions. The water involved is not free but restrained water. Only subsequent increase in temperature at the low-temperature points may change some of the shifted water into free water. This, however, can become an important feature in soils of low permeability under hydrostatic pressures (9). A special case of thermo-osmotic water transmission occurs in the formation of ice lenses in soils. This case has been treated in detail in a recent discussion (10).

WATER TRANSMISSION UNDER ELECTRICAL POTENTIALS

The highly electrical character of the mineral-water interaction phase renders soil-water susceptible to movement if an electrical potential is applied. The general and practical aspects of this phenomenon have been surveyed recently by Casagrande (11). The physicochemical factors that play a dominant rôle, especially in the case of moisture contents falling within the plastic range (water under restraint), have been treated theoretically and experimentally by the author (1, 2, 12). Because of this sufficient and recent coverage, the subject is not further discussed here. However, it should be pointed out that all experimental and theoretical evidence available proves the general correctness of the picture on soil-mineral-water relationships that has been developed over the last 25 years by the soil physicists and colloid chemists. Thus, in systems of low moisture content in which all water is under constraint, well defined minimum voltages must be applied before water can be moved out of the system. These threshold potentials correspond to "yield" pressures in plastic systems (4).

ELECTRICAL CONSEQUENCES DERIVING FROM APPLICATION OF A THERMAL POTENTIAL TO MOIST HYDROPHILIC SYSTEMS

In an electrical system, such as a moist clay soil, in which the electrical charges are asymmetrically distributed between the predominantly negatively charged internal solid surface and the predominantly positively charged water phases, any interference can be expected to have electrical consequences. The latter may, on their part, set up mechanical forces. Hence, it was certain that application of a thermal potential on a moist soil would result in the establishment of an electrical potential (2, 12). This phenomenon appears to be of definite importance in normal and abnormal plant and animal physiology and in thermal methods for curing of disease; it is only of secondary importance for the present Symposium. More information on this subject is found in reference 2.

SIMULTANEOUS ACTION OF SEVERAL POTENTIALS

It is obvious in the light of the evidence presented that a general equation for water transmission through porous soils must have mechanical, thermodynamic, and electrical factors. In first approximation, and this seems to be the best that one can do at the present time, a simple superposition of the effects of these factors can be employed. Instead of doing this in one equation, it is generally preferable to evaluate separately the effect of each factor. In the absence of a thermal potential and of an applied electrical potential when flow occurs as the result of hydrostatic pressure difference, the effect of the resulting electrical flow potential can be included in the volume factor C_2 of Eq 5. On the other hand, electrical drainage is used in soils of low hydraulic permeability, and for practical purposes the main flow effect is due to the applied electrical potential. However, at low moisture contents, the Joule heat developed by the passing current often results in a thermal potential superimposed on the electrical. In such cases the effects of both the electrica and thermal potentials must be considered. The simplest general equation is:

$$v = \left(k_D i + k_e \frac{dE}{dl} + k_T \frac{dT}{dl}\right) t \ldots . (8)$$

where:

v = volume of water moved through unit cross-section,
k_D = Darcy transmission coefficient,
i = hydraulic gradient $\frac{dp}{dl}$,
k_e = electro-osmotic transmission coefficient according to Winterkorn (12),
dE = increment in electrical potential,
dl = increment in distance normal to the cross-section,
dT = increment in temperature,
k_T = thermo-osmotic transmission coefficient, and
t = time.

Although Eq 8 is correct in form and sufficiently accurate for practical purposes, the purist, who understands the derivation of the thermal and electrical members, is disturbed by the fact that in this derivation implied use has been made of the Poiseuille equation, whereas the actual flow picture is different from that postulated by Poiseuille. The engineer, however, need not be concerned, since k_e and k_T as well as k_D are and should be obtained experimentally; and the scientists should know that here, as in many other cases, the final form of the equation is more important than the original picture on which it is based. The classic example of this is, of course, the Carnot equation which is still valid, although its author believed that heat was a material substance.

CONCLUSIONS

In all types of moisture transmission through fine-grained, cohesive soils, the interaction of the internal soil surface with the water phase is of definite and often decisive importance. This interaction has been studied and well elucidated by soil physicists and colloid chemists. There is no valid cause for continued neglect by the soil engineer of the available information. It is realized that much of this information is in a form that is not immediately usable by the soils engineer; the necessary transformation is, however, a job he must do himself.

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DISCUSSION

MR. S. J. ROSCH.[1]—Frankly, I was somewhat disappointed because I had anticipated getting more information from this paper or possibly during its presentation. We are constantly confronted with the fact that the soils throughout the country are radically different. When we consider that we have soils containing extremely dry sand, comparatively wet sand, clays, and combinations of these, we know that these soils may compact differently and yield different values of thermal conductivity.

Therefore I was wondering whether a method has been adopted for determining thermal conductivity of soils, and if so, whether any values have been standardized for the different types of soil. I have studied the reference material in the paper but fail to find this type of information. It is certainly needed for the computation of current-carrying capacity of cables buried directly in the soil, since the thermal conductivity of the soil has an important bearing on this very important problem.

A MEMBER.—I do not get the significance of measuring soil thermal conductivity. Various results are secured depending on the investigator, varying in ratio of two to one. That makes a lot of difference.

MR. HANS F. WINTERKORN (*author's closure*).—The paper just presented treated in a very limited time the theoretical basis of water flow through porous hydrophilic systems under several types of potentials. While flow under thermal potentials was included in the treatment and while the problem of heat conductivity is closely interrelated with that of thermo-osmosis, it was impossible to discuss, within the set frame of this paper, the problem of heat conductivity in soils in such a manner and in such detail as to furnish the engineering information wanted by Mr. Rosch.

With respect to the desired information on heat conductivity, I should like to refer Mr. Rosch to the excellent works of W. O. Smith.[2, 3, 4] Also, I would like to say that I am well aware of Mr. Rosch's practical problem in which thermo-osmosis plays as great a rôle as does straight heat conductivity. The practical solution of this very important problem will require a great deal of scientific and engineering research, which can, however, be based profitably on the theoretical foundations which have already been laid.

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[1] Consulting Cable Engineer, Anaconda Wire and Cable Co., Hastings-on-Hudson, N. Y.

[2] W. O. Smith, "Thermal Transfer of Moisture in Soils," *Transactions*, Am. Geophysical Union, Vol. 25, pp. 511–523 (1944).

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A LOW-HEAD PERMEAMETER FOR TESTING GRANULAR MATERIALS

By E. G. Yemington[1]

Highway drainage designs generally have been based upon empirical rules, and only in recent years has the importance of obtaining permeability data on granular materials been stressed for the design of sand drains and blanket courses used in embankments for assisting in the removal of water by consolidation of the subsoils. In 1952, the Bureau of Public Roads published a report[2] on highway subdrainage. The report did not establish design criteria but it did present test methods, a new type of low-head drainage-lag permeameter, and data on the permeability and drainability of soils and their applications to highway subdrainage.

This paper presents additional test data and information on the above-mentioned drainage-lag permeameter for the measurement of permeability coefficients of open-graded granular materials. Comparisons are made between the coefficients of permeability obtained by this device and an ordinary falling-head permeameter for eight gradings of granular material. The results of these tests indicate that the drainage-lag type of permeameter minimizes turbulence in permeability testing and can be used to measure higher permeability values than the ordinary falling-head type of permeameter.

DRAINAGE-LAG PERMEAMETER

A schematic drawing of this permeameter developed for the measurement of coefficients of permeability for open-graded granular materials is shown in Fig. 1. The outer cylinder, or metal tank, is closed at one end and is equipped with a quick-opening valve near the bottom. An inner cylinder, 6 in. in diameter, is used to hold the test sample, and has a wire mesh attached to the bottom to support the sample. Generally, No. 10 wire mesh is used with No. 200 wire mesh inserted above. The sample cylinder is supported on a base ring which has perforated walls. Provision is made, by means of an Ames dial and a hook gage, to measure accurately the height of the water level in the smaller cylinder at the start and completion of the permeability test. Dimensional characteristics are such that a hydraulic gradient of approximately 0.2 can be readily maintained during testing.

The 6-in. cylinder with wire mesh attached is placed on a solid plate; then a representative sample of the granular material to be tested is placed in the cylinder and compacted to the desired density by either an impact or static method. The cylinder containing the compacted test sample is attached to the base ring in the metal tank and the hook gage assembly attached, as shown in Fig. 1. Water is added to the tank slowly until the sample is completely inundated.

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[1] Highway Physical Research Engineer, Physical Research Branch, Bureau of Public Roads, Washington, D. C.

[2] E. S. Barber and C. L. Sawyer, "Highway Subdrainage," *Public Roads*, Vol. 26, No. 12, February, 1952.

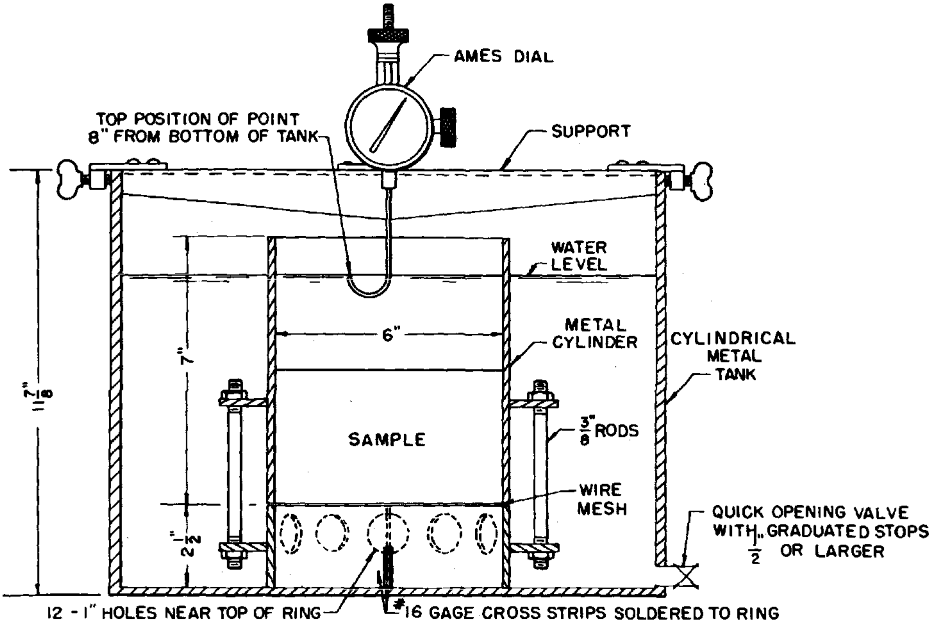

FIG. 1.—Drainage-Lag Permeameter.

After coming to an equilibrium, the water level is determined with a hook gage. The hook gage is then lowered an arbitrarily selected amount, *h*. After measuring the temperature of the water, the quick-opening valve at the bottom of the tank is opened and the outflow caught. A stopwatch is used to determine the time required for the inner water level to reach the hook gage, at which time the watch is stopped and the valve closed simultaneously. The effective head is a variable, $H - h$, where H is the change in water level in the metal tank during the test. The time, discharge, change in water level inside the sample cylinder, and dimensions of the sample and apparatus are used to determine the coefficient of permeability, *k*. This determination is facilitated by the use of Fig. 2. The formula for *k* in Fig. 2 was derived[3] by integration, assuming a con-

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[3] Flow from tank, $\frac{Q}{t}\,dt = S\,dh + a\,d(H - h)$

Flow through sample, $A\,\frac{dh}{dt} = kA\,\frac{H - h}{d}$

Continuing by eliminating dh,

$$dt = \frac{a\,d(H - h)}{Q/t - Sk(H - h)\,d}$$

Integrating, $t = 0$ when $H - h = 0$,

$$t = \frac{ad}{Sk} \ln \frac{1}{1 - Sk(H - h)t/Qd}$$

Substituting $H - h = (Q - Sh)a$,

$$\frac{Stk}{ad} = \ln \frac{1}{1 - Stk(1 - hS/Q)/ad}$$

Letting $F = Stk(1 - hS/Q)/ad$,

$$k = \frac{F}{1 - hS/Q}\,\frac{ad}{St},$$

where F is defined by $hS/Q = 1 - F/\ln 1/(1 - F)$

stant rate of discharge, from the equation:

$$q = \frac{kAh_d}{d}$$

where:

q = volume of flow per unit time,

k = coefficient of permeability,

A = gross area of soil sample perpendicular to direction of flow, and

h_d = head loss through a distance, d, of the soil in the direction of flow.

Since the permeability depends upon the viscosity of the water, which is a function of temperature, the calculated permeability at an arbitrary temperature (68 F (or 20 C)) is often reported as $k_{68} = Ck$, where C is the viscosity of water at the test temperature divided by its viscosity at 68 F. Values of C are plotted in Fig. 3.

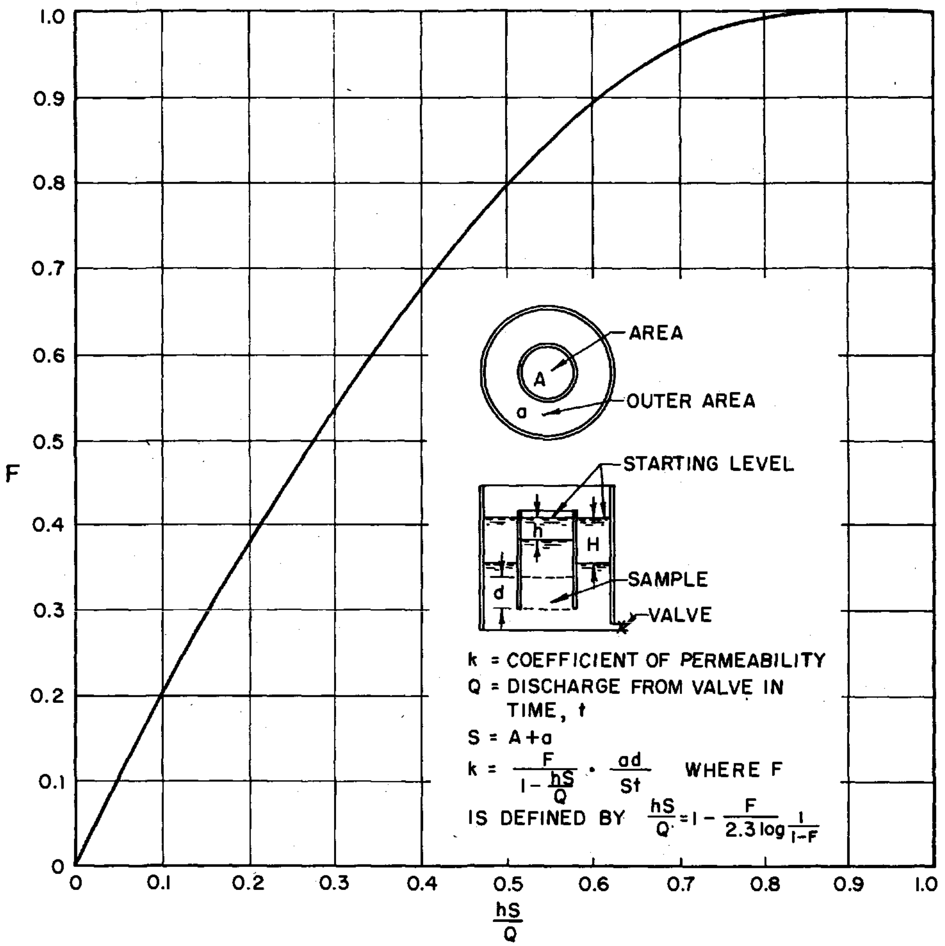

FIG. 2.—Permeability from Drainage-Lag Device.

FALLING-HEAD PERMEAMETER

The ordinary falling-head permeameter used for comparison purposes, shown in Fig. 4, was operated with an hydraulic gradient of approximately 2.0. The co-

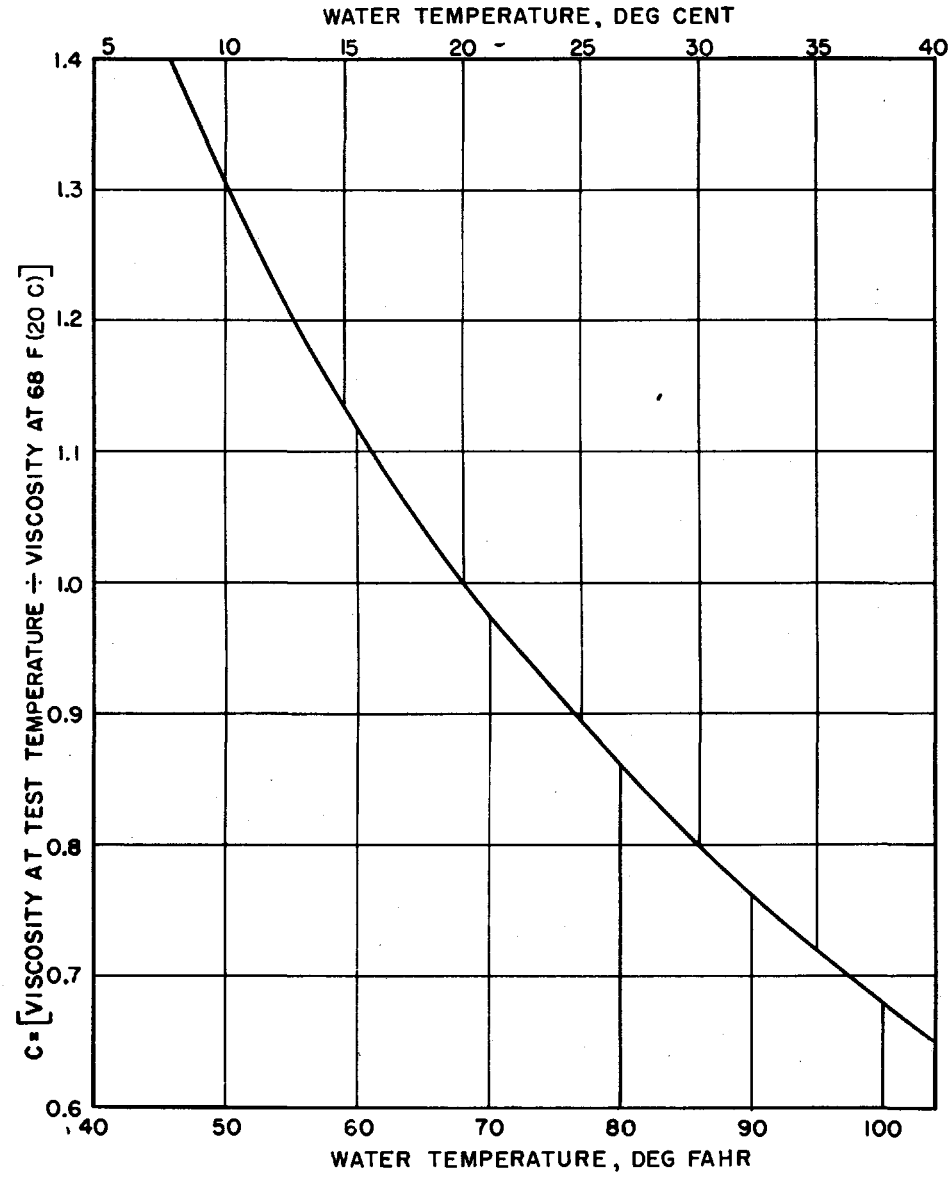

FIG. 3.—Effect of Temperature of Water on the Viscosity Ratio.

efficient of permeability is determined from the formula:

$$k = \left(\frac{2.3ad}{At}\right) \log \left(\frac{h_1}{h_2}\right)$$

where:

a = area of standpipe,
d = thickness of sample,
A = area of sample,
t = time,
h_1 = initial head, and
h_2 = final head.

Since it was necessary to pour water rapidly into the device to build up h_1, a baffle was placed above the sample.

TEST RESULTS

To simulate field conditions, it is often necessary to measure the coefficients of permeability of unsaturated materials. In this research to compare two test methods, it is desirable to eliminate the variable of air because of the difficulty of controlling and measuring the degree of saturation. Removal of the air was accomplished by placing each immersed sample in a large tank and subjecting it to a vacuum. In the drainage-lag device, the vacuum can be applied through a lid fitting the top of the metal tank.

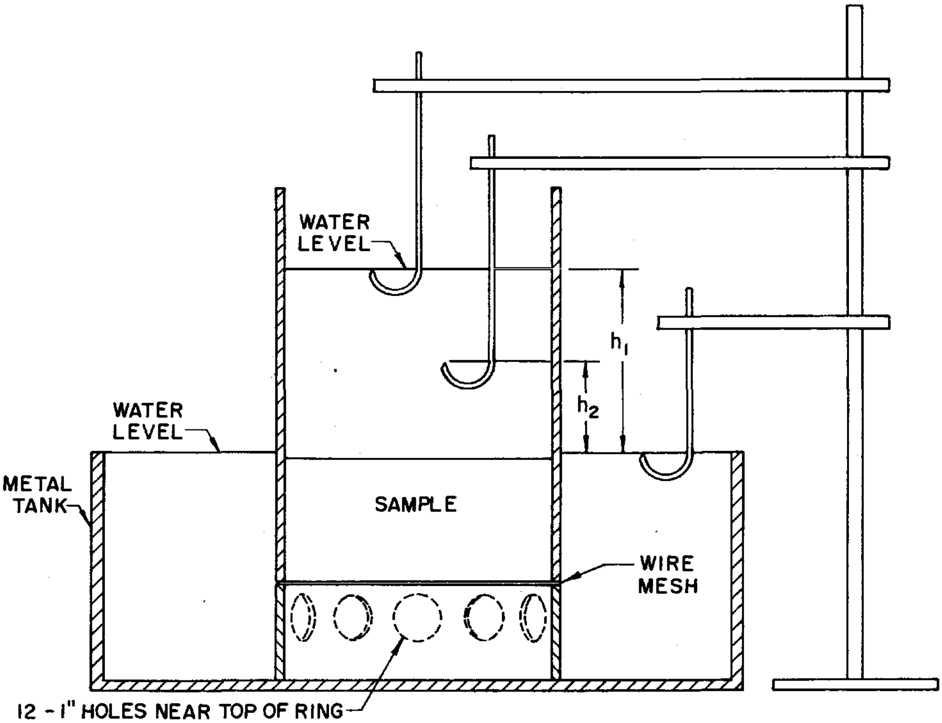

FIG. 4.—Ordinary Falling-Head Permeameter.

Compacted samples of each of eight materials were tested alternately by each method. Table I shows the gradations of the materials tested and their coefficients of permeability as determined through use of the above-mentioned devices. It is evident that the coefficients of permeability of samples Nos. 1, 2, and 3 are not influenced by the type of testing apparatus used. The coefficients of permeability for samples Nos. 4 and 5 vary considerably for the two types of testing apparatus. The higher values as determined by the drainage-lag apparatus are due to less turbulence than that occurring when the ordinary falling-head device is used.

TABLE I.—GRADATION OF MATERIALS TESTED AND THEIR COEFFICIENTS OF PERMEABILITY AS DETERMINED BY TWO TYPES OF PERMEAMETERS.

	Sample No. 1	Sample No. 2	Sample No. 3	Sample No. 4	Sample No. 5	Sample No. 6	Sample No. 7	Sample No. 8
Percentage passing:								
1½-in. sieve	...	...	...	...	...	...	...	100
1-in. sieve	...	...	...	...	...	...	...	0
½-in. sieve	...	...	...	...	...	100	100	...
⅜-in. sieve	...	...	...	...	...	96	0	...
No. 4 sieve	...	...	...	...	100	20	...	...
No. 10 sieve	100	100	100	100	2	1	...	...
No. 40 sieve	80	72	1	1	0	0	...	...
No. 200 sieve	14	1	0	0	...	...	...	...
Dry density, lb per cu ft	90	113	113	102	102	101	101	104
Coefficient of permeability, ft per day:								
Ordinary falling-head permeameter	18	129	624	1936	6380	[a]	[a]	[a]
Drainage-lag permeameter	18	129	629	2466	7775	9660	81 808[b]	105 625[b]

[a] Unable to determine permeability, fall too rapid.
[b] No. 200 wire mesh removed.

Between 6000 and 7000 ft per day appeared to be the maximum values that could be measured with the ordinary falling-head permeameter, whereas much higher values were measurable with the drainage-lag permeameter. For samples Nos. 6, 7, and 8, it was necessary to increase the porosity of the ring supporting the sample container.

Several tests were run with the No. 200 wire mesh removed to measure the effect of gradients on the movement of fines in poorly graded materials. It was observed that the greatest loss of fines occurred during the immersion of the sample. Additional loss of fines during testing was considerably less in the drainage-lag permeameter than in the ordinary falling-head permeameter. Movement after immersion was minimized in the drainage-lag device by pouring water into the tank at a constant rate to give an upward gradient.

CONCLUSION

The drainage-lag device provides a means of measuring the permeability of very coarse materials up to 100,000 ft per day. The effect of turbulence and the movement of fines is minimized by the use of small gradients comparable to those occurring in the field.

PERMEABILITY TEST FOR SANDS

By T. Y. Chu,[1] D. T. Davidson,[1] and A. E. Wickstrom[1]

Synopsis

Information on the permeability of sand is often required in its utilization as an engineering material. This paper presents a modification of Barber's falling head permeability test for porous granular materials. The modified method is simple to use and gives reproducible results.

The modifications recommended include: (1) a procedure for loading and compacting the sample in the permeameter tube, (2) a carbon dioxide treatment to remove air from the sample, and (3) the repeated testing of the loaded sample until permeability values obtained are within a recommended range of variation.

The suggested permeability test was used to measure the permeability of several Iowa sands at various densities. Compositional information pertinent to the permeability of the sands is also presented.

Barber's falling head permeability test (**1**)[2] has been widely used for the laboratory determination of the permeability of sands.[3] This method is simple to use and does not require complicated equipment. However, test results obtained may not be readily reproducible, mainly because of the lack of a specific procedure for loading the sample in the permeability tube. This paper presents a modification of Barber's test method as set forth in complete form in the Appendix, which method is easy to use and which gives reproducible results.

In developing the modified permeability test for sands, experiments were made for the following purposes: (1) to evaluate different methods of loading and compacting the sample in the permeameter tube, (2) to determine the effect of air entrapped in the loaded sample on permeability, (3) to determine the effect of soaking the loaded sample in water for varying periods of time prior to testing, and (4) to compare the permeability values obtained by repeated tests on the same loaded sample.

Loading and Compaction:

In Barber's permeability test, the loading is done by pouring an air-dried sample into a 2-in. diameter permeameter tube to a depth of 6 in. Since the tube is usually 48 in. long, the pouring of dry sand in this manner may cause appreciable segregation of coarse and fine particles. Segregation can be avoided by

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[2] The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 50.

[3] For definition, see Definition of the Term Sand (C 58–28), 1952 Book of ASTM Standards, Part 3, p. 988.

loading the sample in a moist condition, but it may be difficult to compact the moist sand to a desired density.

A comparison was made of different techniques for loading dry sand in the permeameter tube. The procedure called the "inverted method" was found to be most satisfactory. In this method, the permeameter tube is inverted and the sample is loaded in what is then the upper 6 in. of the tube. After loading, the tube is turned right side up for the permeability test. The details of the loading procedure are given in the Appendix.

TABLE I.—COMPARATIVE REPRODUCIBILITY OF PERMEABILITY TEST RESULTS OBTAINED WITH SAMPLES[a] LOADED BY TWO DIFFERENT METHODS.

Method of Loading	Experiment	Coefficient of Permeability, ft per day
Inverted method	No. 1	28.2
	No. 2	29.2
	No. 3	29.4
	No. 4	30.9
Sample poured from top to bottom of permeameter tube	No. 1	16.8
	No. 2	25.3
	No. 3	12.0
	No. 4	19.6

[a] The data are for sample No. 77-S in Table III.

The typical test data given in Table I illustrate the high degree of reproducibility in test results obtainable by the inverted method of loading. The low reproducibility of results obtained with similar samples loaded by the method of pouring the sample from top to bottom of the permeameter tube are shown for comparison. All samples were tested for permeability according to Barber's procedure.

The density of a sample loaded by the inverted method is close to the compact unit weight.[4] Higher densities may be obtained by compacting the sample with a cylindrical weight as described in the Appendix. Sands can be easily compacted to Standard Proctor density by this method.

Carbon Dioxide Treatment:

The permeability test can be performed with the sample in either a completely saturated condition or a partially saturated condition. The condition of testing will usually depend upon the purpose of the test. The simplest way to obtain results on a comparable basis is to use completely saturated samples. The permeability test for sands given in the Appendix was developed primarily for testing samples in the condition of complete saturation. However, the procedure for loading and compaction may also be used for preparing samples for testing in the partially saturated condition.

To obtain complete saturation, the sample should be free from entrapped air before being tested for permeability, and no air should be introduced into the sample from the percolating water during testing. The entrainment of air in the test sample can be prevented by evacuation methods (2) or by passing carbon dioxide gas[5] through the air-dry sample after it has been loaded in the permeameter tube and before it is compacted. The latter method is comparatively simple to use and was found satisfactory by Christiansen (3) and others in permeability tests with many types of soil. It was the only method studied in the present investigation.

The two variables involved in the carbon dioxide treatment are the rate of flow and the duration of flow of carbon dioxide passed through the sample. Experiments were performed with sand samples to determine the satisfactory

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[4] Method of Test for Unit Weight of Aggregate (C 29 – 42), 1952 Book of ASTM Standards, Part 3, p. 975.

[5] The air in the sample is displaced by carbon dioxide which will be quickly dissolved by percolating water during the permeability test.

quantities of the gas to be used. The values recommended in the test method in the Appendix seem adequate.

De-aired water may be used in permeability tests to avoid the release of dissolved air as the water percolates through the test sample. The use of distilled water at a temperature 5 to 10 F higher than room temperature, as recommended in Barber's method, also serves to avoid the release of dissolved air. Because of its simplicity this method is recommended.

TABLE II.—TYPICAL DATA SHOWING EFFECT OF CARBON DIOXIDE TREATMENT ON THE PERMEABILITY OF SANDS.[a]

Sample	Porosity, per cent	Carbon Dioxide Treatment	Coefficient of Permeability,[b] ft per day	Relative Permeability,[c] per cent
No. 77-S	39.6	No	29.4	76
		Yes	38.8	100
	36.6	No	12.0	49
		Yes	24.6	100
No. 89-S	33.8	No	150.2	89
		Yes	168.7	100

[a] Properties of samples used are given in Table III.

[b] Data are the average of at least two tests.

[c] Relative permeabilities are computed by taking the coefficient of permeability obtained with carbon dioxide-treated samples as 100 per cent.

Experiments were made to determine the effect of air removal by the carbon dioxide treatment on the permeability of several sand samples. The samples were loaded in the permeameter tube by the inverted method (see Appendix) and were compacted to various densities prior to permeability determinations by Barber's procedure. Test data showed that the carbon dioxide treatment results in a higher permeability regardless of the type of sand and the degree of compaction. The data also indicated that the lower the permeability of the untreated sample the greater the increase in permeability due to the carbon dioxide treatment. These observations are illustrated by the typical data given in Table II.

Soaking:

After the sample has been loaded in the permeameter tube, treated with carbon dioxide and compacted to the desired density, the tube is immersed in distilled water in a water bath (see Appendix). The permeability determination can be started either immediately after the water in the tube has risen nearly to the water level in the bath or after the sample has been soaked for an additional period of time. Experimental results indicate that additional soaking up to 48 hr does not appreciably change the permeability value. For this reason, a soaking period prior to the permeability determination is not considered necessary.

Change in Permeability During Testing:

In Barber's method of test, the permeability is determined on the basis of the time required for the water in the permeameter tube to drop from level *A* to level *C* as shown in Fig. 8 in the Appendix. Any change in the permeability of the sample during this time cannot be detected, since only one coefficient of permeability is computed. If an intermediate level, level *B* in Fig. 8, is marked on the permeameter tube and the time intervals required for the water to drop from level *A* to level *B* and from level *B* to level *C* are recorded, two coefficients of permeability can be computed. If this is done, any change in the permeability of a sample during testing is indicated by the difference between the two coefficients of permeability.

To facilitate the comparison of the two coefficients of permeability, an intermediate level can be selected to conform to the following relationship:

$$\frac{h_a}{h_b} = \frac{h_b}{h_c} \quad \text{.............} \quad (1)$$

where:

h_a = height[6] of level A,

h_b = height of the intermediate level (level B), and

h_c = height of level C.

As shown in Fig. 8, h_a = 36 in., h_b = $20\frac{25}{32}$ in., and h_c = 12 in. Equation 1 and the formula for computing the coefficient of permeability given in the Appendix show that, if the permeability of the sample is constant during the test, the time intervals required for the water in the tube to drop from level A to level B and from level B to level C will be the same. For simplicity the time required for the water to drop from level A to level B will be referred to as the first reading, and the time from level B to level C the second reading. Any difference between the first and the second readings is an indication of a change in the permeability of the sample during testing. Test results indicate that, when a single permeability determination is made on a sample, the first reading is usually different from the second. It was also found that the deviation between the two readings often decreases with repeated permeability determinations on the same loaded sample. This is illustrated in Fig. 1. The deviation curve shows that after several successive permeability determinations the amount of deviation between the two readings becomes relatively small and remains nearly constant.

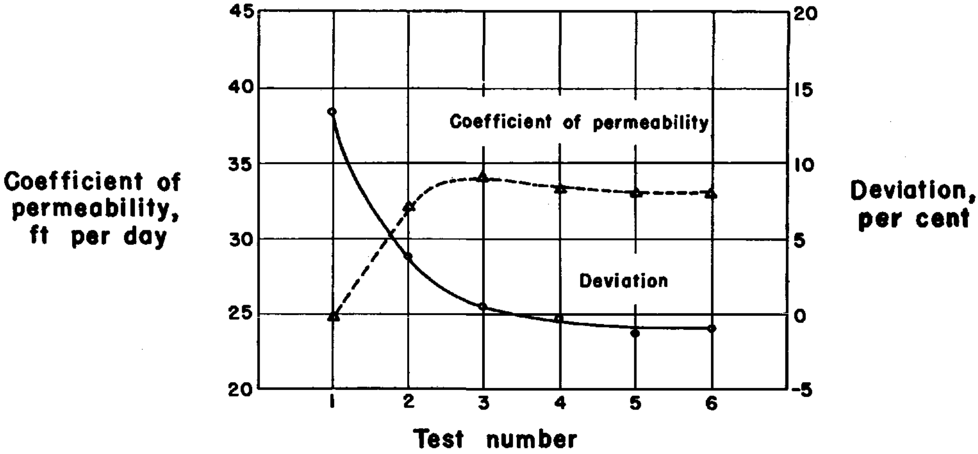

FIG. 1.—Typical Curves Showing Variation in Permeability Test Results Obtained by Repeated Testing of a Sand Sample (Sample No. 92-S in Table III).

$$\text{Per cent of deviation} = \frac{2(T_1 - T_2)}{T_1 + T_2} \times 100$$

where:

T_1 = time for water to drop from level A to level B, and

T_2 = time for water to drop from level B to level C.

Figure 1 further shows that the permeability increases with repeated testing but approaches a uniform value as the deviation curve flattens. The permeabilities obtained from tests having small deviations between first and second readings are, probably, more representative of the test sample than the permeabilities obtained from tests having large deviations. For this reason, it seems de-

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[6] All heights are measured from the water level in the bath.

TABLE III.—MECHANICAL COMPOSITION OF FOUR IOWA SANDS.

Sample	Textural Composition[a]			Sorting Coefficient, S_0	Specific Gravity	Average Sphericity	Surface Texture[b]		Aggregation Characteristics		
	Sand, per cent	Silt, per cent	Clay, per cent				Markings	Coatings	Amount	Size, mm	Cement
No. 77-S.........	87.6	9.6	2.8	1.6	2.66	0.71	Dull and rough (faceted)	Partially coated by calcareous clay	Common	1–3	Calcareous clay
No. 79-S.........	84.7	11.0	4.3	1.4	2.65	0.73	Dull and smooth	Partially coated by ferruginous clay	Abundant	½–1½	Ferruginous clay
No. 89-S.........	98.1	1.6	0.3	1.2	2.67	0.75	Dull and rough	Partially coated by iron oxide	Absent	...	...
No. 92-S.........	92.9	4.3	2.8	1.3	2.68	0.61	Dull and rough (pitted)	Completely coated by slightly calcareous clay	Common	1–2	Slightly calcareous clay

[a] Sand—2 to 0.074 mm; silt—0.074 to 0.005 mm; clay—less than 0.005 mm.

[b] Descriptions apply to most sand-size particles in the sample.

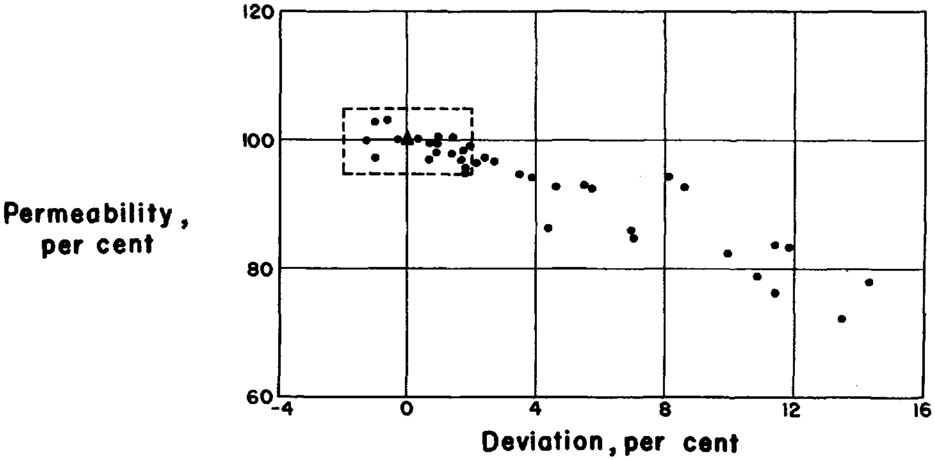

FIG. 2.—Relationship Between Permeability Value and Deviation Between First and Second Readings.

$$\text{Per cent of permeability} = \frac{k_a}{k_0} \times 100$$

where:

k_a = coefficient of permeability corresponding to a deviation greater than ±0.5 per cent, and
k_0 = coefficient of permeability of the same test sample corresponding to a deviation less than ±0.5 per cent.
Per cent of deviation is defined in Fig. 1.

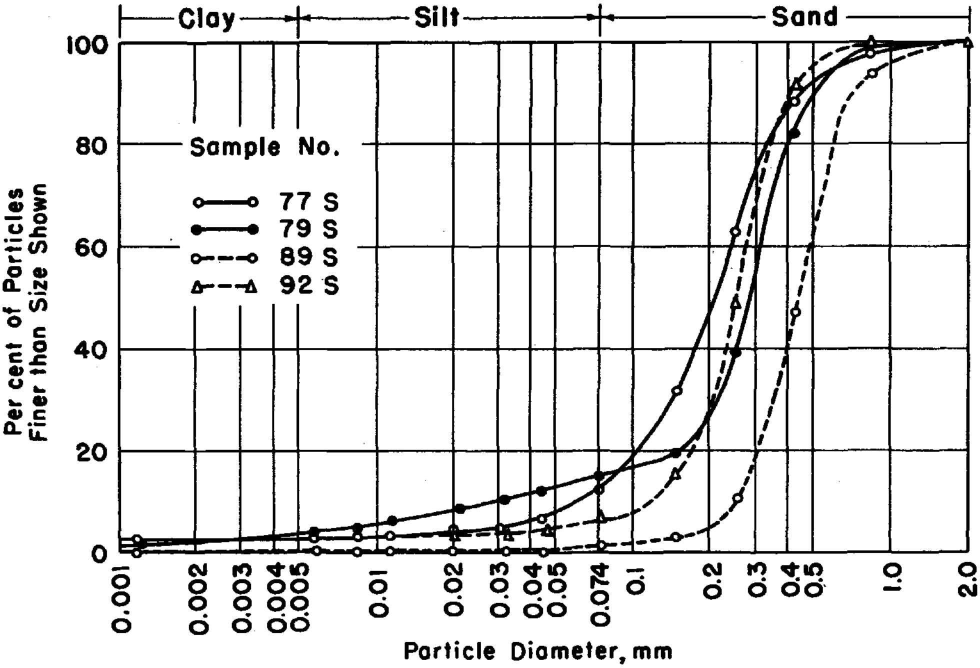

FIG. 3.—Particle-Size Accumulation Curves for the Four Iowa Sands.

sirable in routine permeability tests to repeat the determination until the deviation between the first and second readings is small and then to report the coefficient of permeability obtained by this determination.

To determine the permissible deviation of first and second readings, an analysis was made of permeability and deviation data for the four sand samples (see Table III) at various densities. Figure 2 shows the relationship between permeability values and deviations. The permeability values are expressed on a percentage basis. The coefficient of permeability corresponding to a deviation less than ±0.5 per cent is taken as 100 per cent permeability and is represented by the small triangle in the upper left of the figure. The per cent of permeability corresponding to deviations of more than ±0.5 per cent is computed by the equation given in Fig. 2. The computed values for the four sand samples are plotted as dots in the figure; each dot represents one permeability determination. As shown by the dots within the rectangular dashed-line area, when the deviation is less than ±2.0 per cent, the permeability value is within about ±5 per cent of the 100 per cent permeability value. Since a 5 per cent variation in the permeability value seems to be a reasonable tolerance, ±2.0 per cent is recommended as an allowable limit of deviation between first and second readings. In many tests, this requirement was met in the second permeability determination.

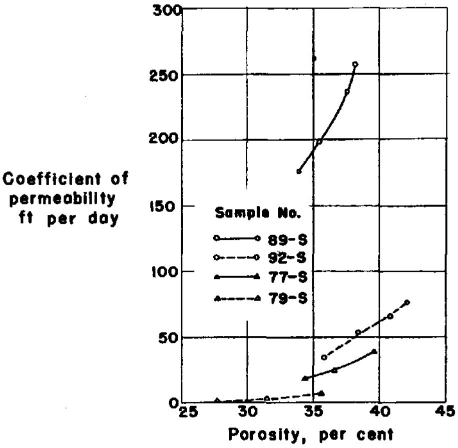

FIG. 4.—Permeabilities of the Four Iowa Sands at Various Porosities.

PERMEABILITY OF FOUR IOWA SANDS

The test method presented in the Appendix was used in determining the permeability of four Iowa sands. The source and properties of the sands were reported in a previous paper (4). Compositional information pertinent to their permeability is given in Fig. 3 and in Table III. The sorting coefficient, S_0 in the table, is computed by the formula (5):

TABLE IV.—COEFFICIENTS OF PERMEABILITY OF FOUR IOWA SANDS AT VARIOUS DENSITIES.

Sample	Dry Density		Porosity, per cent	Coefficient of Permeability, ft per day
	lb per cu ft	Per cent of Standard Proctor Density		
No. 77-S	100	92	39.6	39.5
	105	97	36.6	24.5
	109	100	34.5	18.4
No. 79-S	106	90	35.6	6.9
	113	96	31.5	2.4
	119	101	27.8	1.4
No. 89-S	103	96	38.1	258.5
	104	97	37.5	237.5
	108	100	35.5	199.4
	110	102	34.0	175.4
No. 92-S	97	90	42.0	77.1
	99	92	40.8	66.3
	103	96	38.3	54.5
	107	100	35.7	34.1

$$S_0 = \sqrt{\frac{Q_a}{Q_b}}$$

The values of Q_a and Q_b are determined from the particle-size accumulation curve: Q_a is the maximum diameter of the smallest 75 per cent by weight of the soil particles, and Q_b is the maximum diameter of the smallest 25 per cent. A well graded sand will have a higher sorting coefficient and, usually, a lower porosity than a poorly graded sand. The average sphericity values are for sand-size particles in the sample, as determined by the Rittenhouse chart (6). A perfect sphere has a sphericity of 1.00. The predominant kind of clay mineral in each of the four sands is illite.

Permeability test results for the four sands at various densities are summarized in Table IV and plotted in Fig. 4. The data show that the sands differ greatly in their coefficients of permeability and the coefficient varies with the density for each sand.

The differences in permeability of the sands are related to their void characteristics. The term "void characteristics" as used here refers not only to the amount of voids in a sample but also to other related variables, such as the size distribution and continuity of the voids. The number of voids in a sample can be determined easily and is commonly expressed in terms of the porosity of the sample. The other variables are difficult to measure quantitatively, but they can be evaluated qualitatively from compositional information.

Each curve in Fig. 4 shows the decrease in permeability that accompanies a decrease in porosity. The figure also illustrates that porosity is not the only important factor affecting permeability. For example, sample No. 79-S at a porosity of 35 per cent has a coefficient of permeability of about 6 ft per day, but sample No. 89-S tested at a similar porosity has a coefficient of about 200 ft per day. This great difference in permeability is largely due to the differences in mechanical composition as shown in Fig. 3 and Table III.

Acknowledgment:

The subject matter of this paper was obtained as part of the research being done under Project 283-S of the Iowa Engineering Experiment Station of Iowa State College. This project, entitled "The Loess and Glacial Till Materials of Iowa: An Investigation of Their Physical and Chemical Properties and Techniques for Processing Them to Increase Their All-Weather Stability for Road Construction," is being carried on under contract with the Iowa State Highway Commission and under the sponsorship of the Iowa Highway Research Board. The project is supported by funds supplied by the Commission and the U. S. Bureau of Public Roads.

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APPENDIX

METHOD OF TEST FOR PERMEABILITY OF SANDS

Scope:

This method describes test procedures for determining the coefficient of permeability of sands as defined in the ASTM Standard Definition of the Term Sand (C 58).[3]

Apparatus:

The apparatus used in this test consists of the following:

Permeameter Tube.—A rigid transparent plastic tube approximately 48 in. in length and about 2 in. in inside diameter. The tube is marked as shown in Fig. 8.

Brass Screens.—Two circular brass screens. One is of No. 200 sieve cloth, the other of No. 10 sieve cloth. The diameters of the screens are the same as the outside diameter of the permeameter tube.

Screen Holder.—A holder to attach the screens tightly to the permeameter tube.

Supporting Device.—A supporting device made of rubber stoppers, a brass tube, etc., as shown in Fig. 5.

Loading Funnel.—A special funnel as shown in Fig. 5.

Tamping Rod.—A straight $\frac{1}{8}$-in. round metal rod, approximately 8 in. long.

Attachment for Carbon Dioxide Treatment.—An attachment made of a rubber sleeve, rubber stopper, etc., as shown in Fig. 5.

Bottle of Carbon Dioxide and Flow Meter.—A 50-lb tank of liquid carbon dioxide with a flow meter, accurate to 0.01 cu ft per min, attached.

Cylindrical Weight.—A 4-lb steel or brass weight having a diameter of about $1\frac{7}{8}$ in.

Balance.—A balance of 1 kg capacity and accurate to 0.1 g.

Ruler.—A ruler reading to $\frac{1}{32}$ in.

Thermometer.—A thermometer accurate to 1 F or 0.5 C.

Stop Watch.—A stop watch reading to $\frac{1}{5}$ sec.

Water Bath.—A water bath with attachments for holding permeameter tube as shown in Fig. 8.

Loading of Test Sample:

A representative air-dry sample of sufficient quantity, usually 500 g, is prepared and its weight is recorded. The sample is mixed until uniform. To obtain the oven-dry weight of the sample, the air-dry weight is corrected for hygroscopic moisture.

The sample is loaded in the permeameter tube in four layers, each approximately 1.5 in. thick. The loading funnel and supporting device shown in Fig. 5 are used in loading the sample by the following procedure:

1. Invert the permeameter tube, and assemble loading apparatus as shown in Fig. 5. For loading the first layer, the stem of the funnel rests on the top of the supporting device.
2. Pour a portion of the sample sufficient to make a layer approximately 1.5 in. thick into the stoppered funnel. If segregation occurs as the sample is being poured, remix to a uniform condition in the funnel.
3. Pull the stopper and allow the sand to flow down into the stem of the funnel. The small hole in the neck of the funnel permits the escape of air from the stem.
4. Carefully remove the funnel. The resulting layer will usually be free from segregation.
5. Use the tamping rod to rod the layer five times. The rodding is distributed uniformly over the layer and extends through it.

The above procedure is followed in loading each of the other three layers. After all four layers are loaded, any excess sand above the top of the permeameter tube is removed and saved for weighing. The weight of sand in the permeameter tube is determined by subtracting the weight of all excess sand from the initial weight of the sample.

Two brass screens are attached tightly to the top of the permeameter tube, the one of No. 200 sieve cloth being in direct contact with the loaded sample. The permeameter tube is then turned right side up and the supporting device removed. In removing the supporting device, the following procedure is used:

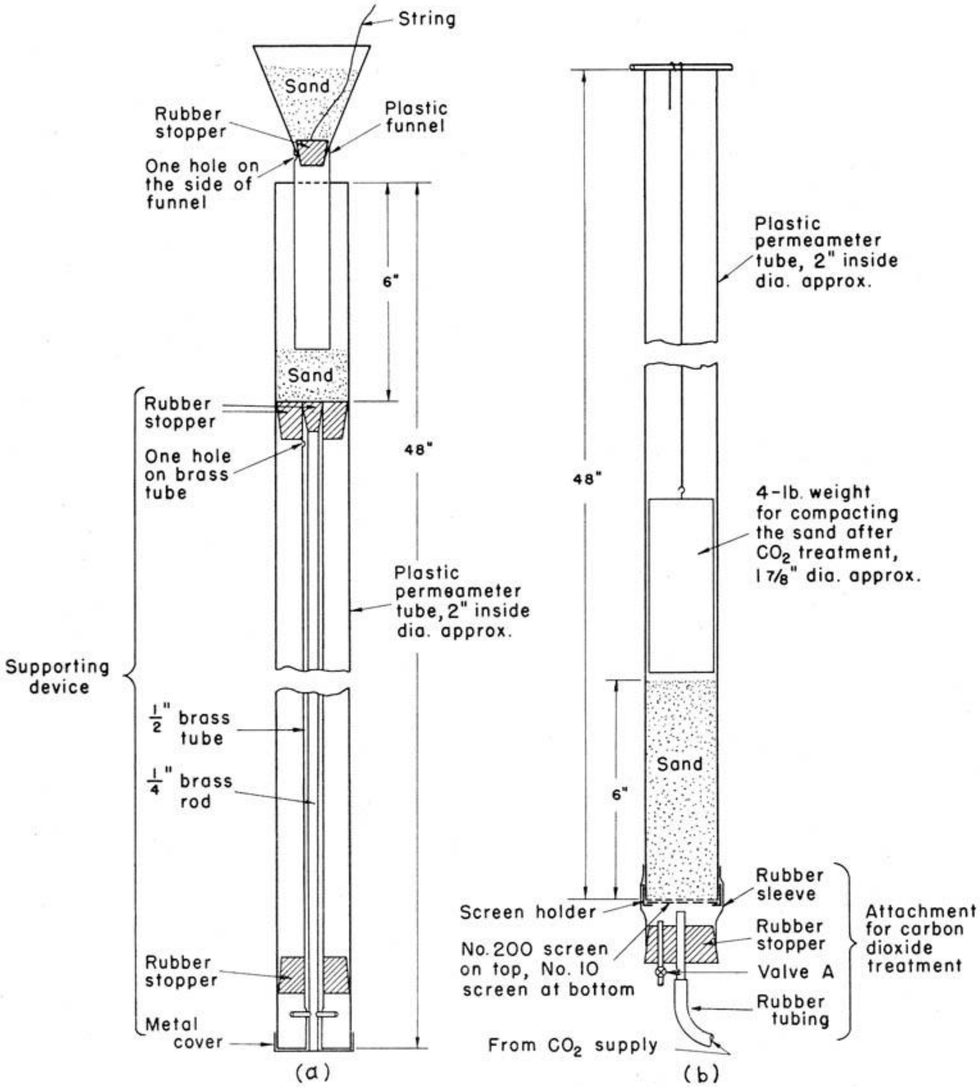

FIG. 5.—Apparatus for Preparing Sample for Permeability Determination.
(*a*) Apparatus for loading sample in permeameter tube.
(*b*) Apparatus for carbon dioxide treatment.

1. Lift the rod inside the brass tube until the short cross bar is clear of the slots, and

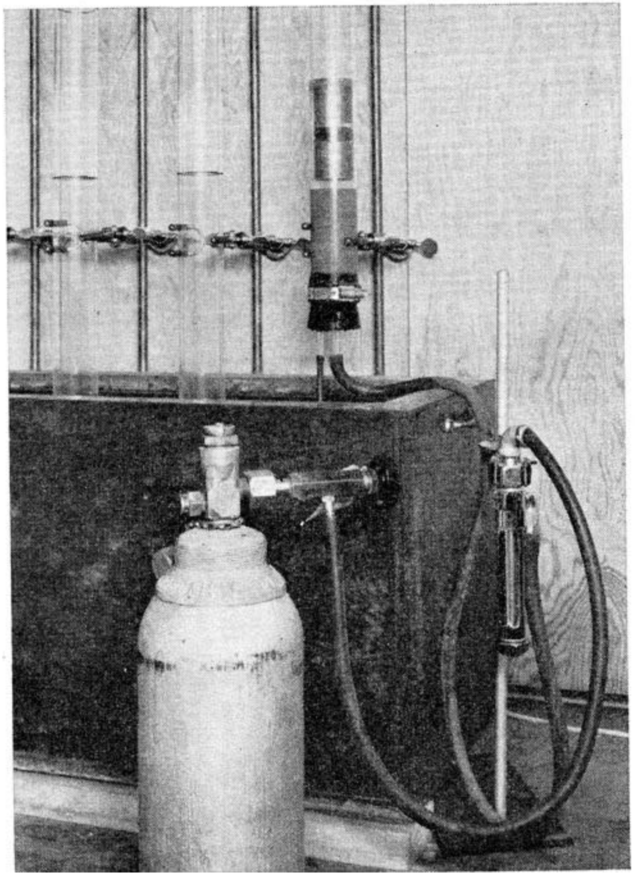

FIG. 6.—Carbon Dioxide Treatment of Sand Samples.

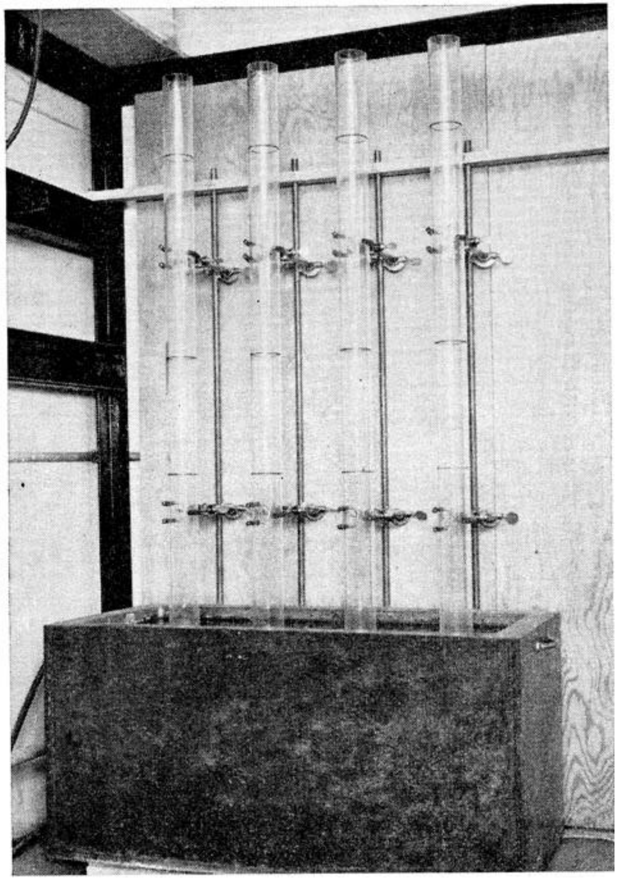

FIG. 7.—Arrangement of Apparatus for Testing a Group of Samples.

then rotate it about 90 deg until the cross bar can be rested on the end of the brass tube. This step is necessary to prevent disturbance of the sample during removal of the supporting device.

2. With the permeameter tube in a vertical position, remove the whole supporting device slowly from it.

Carbon Dioxide Treatment:

The sample in the permeameter tube is treated with carbon dioxide gas in the manner shown in Figs. 5 and 6. The rate at which the carbon dioxide gas is passed through the sample is 0.03 cu ft per min. During the first ½ min of the carbon dioxide treatment, valve *A* is left open to permit the escape of air from the chamber below the sample. The valve is then closed, and the treatment is continued for an additional 15 min. The amount of carbon dioxide passed through the sample is about 0.45 cu ft.

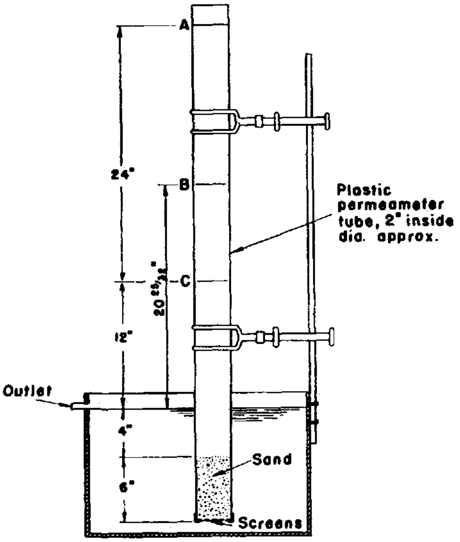

FIG. 8.—Arrangement of Apparatus for Permeability Determination.

Compaction (Optional):

After the carbon dioxide treatment, the sample is compacted, if necessary, to the desired density before being tested for permeability. In compacting the sand, the 4-lb cylindrical weight is placed on the sample in the permeameter tube. The portion of the tube containing the sample then is patted with the palm of the hand until the desired density is obtained.

NOTE.—Since both the weight of the sample in the tube and the inside diameter of the tube are known, the depth to which the sample is compacted can be computed from the desired density.

Permeability Determination:

The sample in the permeameter tube is immersed in distilled water at room temperature by one of the following procedures:

Method A.—For samples loaded in the permeameter tube without additional compaction.—After the carbon dioxide treatment, the sample is gently immersed in the water bath. The permeameter tube is positioned as shown in Fig. 8.

Method B.—For samples loaded and further compacted in the permeameter tube.—The sample is very gently immersed in the water bath with the weight used for compaction on top of the sample to prevent expansion during immersion. The weight is removed when the rising water nears the top of the sample. The position of the permeameter tube in the water bath is as shown in Fig. 8.

After the water in the permeameter tube has risen to the level of the water bath, the tube is filled with distilled water at a temperaturen betwee 5 and 10 F above room temperature, without disturbing the sample in the tube. The time intervals required for the water in the tube to drop from level *A* to level *B* and from level *B* to level *C* are recorded with a stop watch.

If the difference between the first and second time intervals is more than 2.0 per cent of the average value, the preceding step is repeated one or more times until the difference is less than 2.0 per cent. The depth and the temperature of the sample in the permeameter tube are then measured.

NOTE.—Figure 7 illustrates the assembly of apparatus for testing a group of samples simultaneously.

Calculation:

The coefficient of permeability, k, in feet per day, is calculated as follows:

$$k = \frac{276Cd}{t} \log \frac{h_a}{h_c}$$

where:

C = temperature correction (viscosity of water at the temperature measured in the sample after testing divided by viscosity of water at 68 F),

d = depth of sample in inches measured after the test,

h_a = height of level A from the water level in the bath,

h_c = height of level C from the water level in the bath, and

t = time in minutes required for water to drop from level A to level C during the last permeability determination.

NOTE.—For h_a = 36 in. and h_c = 12 in., the above formula can be simplified to:

$$k = \frac{131.7Cd}{t}$$

THE PERMEABILITY OF FINE-GRAINED SOILS

By T. William Lambe[1]

Importance of Permeability Measurements:

The permeability of a soil is one of its most fundamental and important properties. It enters into nearly all seepage, settlement, and stability problems confronting the soil engineer. The amount of leakage through and under dams, the rate at which a building settles, and the rate at which the strength of a deposit increases after it has been subjected to a consolidating pressure are typical of the many problems in which the permeability of a soil can be a critical factor.

The importance of evaluating the permeability of a pervious soil has been long recognized and test techniques for measuring it have been well developed and are widely used. The permeability of fine-grained soils, however, has not received extensive study. Soils with permeabilities of less than 1 μ per sec are often considered "impervious" and are not subjected to permeability testing.

Soil permeabilities less than 1 μ per sec are becoming increasingly important to soil technologists from both practical and theoretical considerations. More use is being made of "impervious" soil to line canals and reservoirs and to construct cores for earth dams. Research workers are studying the permeability of fine-grained soils to learn more of the nature of particle surfaces and the thickness of adsorbed water **(7)**.[2] A look at some of the factors influencing the permeability of fine-grained soils, therefore, is timely.

Theoretical Permeability Equations:

From a comparison of flow through soils with flow through capillary tubes, the following equation **(8)** was developed:

$$k = D_s^2 \frac{\gamma}{\mu} \frac{e^3}{1 + e} C \quad \ldots\ldots\ldots (1)$$

where:

k = the Darcy coefficient of permeability, or simply permeability,

D_S = some effective particle diameter,

γ = unit weight of permeant,

μ = viscosity of permeant,

e = void ratio, and

C = shape factor.

An expression for the permeability of porous media, proposed by Kozeny **(2)** and improved by Carman **(1)** is:

$$k = \frac{1}{k_0 S^2} \frac{\gamma}{\mu} \frac{e^3}{1 + e} \quad \ldots\ldots\ldots\ldots (2)$$

where:

k_0 = constant depending on pore shape and ratio of length of actual flow path to soil bed thickness, and

S = specific surface area.

Since D_s is defined as the diameter of particle having specific surface S, Eq 1 may be considered a simplification or extension of the Kozeny-Carman equation.

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[2] The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 67.

These equations have been found to express accurately the permeability characteristics of saturated sand. Laboratory tests suggest that a plot of degree of saturation *versus* permeability of sand approximates a straight line (4). The various factors affecting the permeability of cohesionless soils are, therefore, relatively well known.

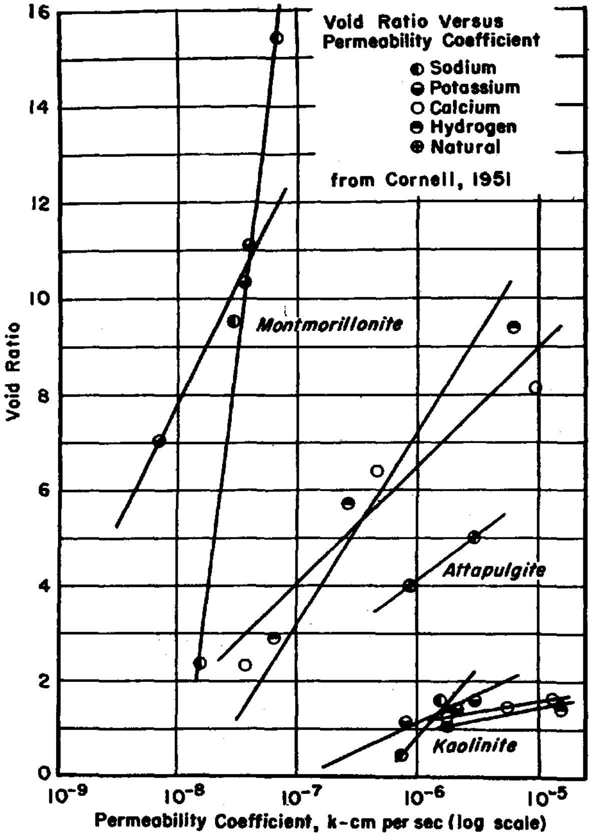

FIG. 1.—Void Ratio *versus* Permeability. From Cornell University report (12).

The permeability equations are of very limited use to the soil engineer for fine-grained soils for two reasons: (1) the difficulty of selecting the effective "constants" and soil characteristics, and (2) the fact that these various terms are not independent, but interrelated in a very complex manner. One can well argue for discarding the equations when considering fine-grained soils; one can also argue that the equations are sound but that the knowledge of soils is not extensive enough to permit proper interpretation of the equations.

The purpose of this paper is to present and discuss briefly the major factors affecting the permeability of fine-grained soils and to show that these factors are not covered in a correct, or at least an interpretable, manner by the Carman-Kozeny equation. The factors to be examined in this paper are:

1. Soil composition.
2. Permeant characteristics.
3. Void ratio.
4. Structure.
5. Degree of saturation.

EFFECT OF SOIL COMPOSITION

The influence of soil composition on permeability is generally of little importance (mica and organic matter are two of the exceptions) with silts, sands, and gravels; it is of major importance with clays. Clay "composition" includes minerals, exchangeable ions, and impurities (free iron oxides and organic matter, for example).

At Cornell University consolidometer test specimens were subjected to variable head permeability measurements and the data in Fig. 1 obtained. The permeabilities of the various ionic forms of montmorillonite at a void ratio of 8 vary as:

$$Ca > H > Na > K$$

At a void ratio of 1.5 for kaolinite:

$$H > Ca > K > Na$$

A comparison of permeabilities at constant consolidation pressure is probably more indicative than one at constant void ratio. Such a comparison can be approximated from the coefficients of consolidation listed by the Cornell report for kaolinite, namely,

$$H > Fe > Mg > Ca > Na$$

From the Cornell and other data (for example, Winterkorn (10) and Winterkorn and Moorman (11)), the following order for permeability at a given consolidation pressure appears to be usually true:

$$Ca > Mg > K > Na$$

Figure 1 shows that the permeabilities of the three minerals are in the following order: montmorillonite < attapulgite < kaolinite. It is seen from Fig. 1 that, at a void ratio as high as 15, sodium montmorillonite had a permeability less than 10^{-7} cm per sec. Sodium montmorillonite is one of the least permeable soil minerals and is therefore widely used by the engineer as an impermeabilizing additive to other soils.

The magnitude of permeability variation with soil composition ranges widely. Figure 1 shows the ratio of permeability of calcium montmorillonite to that of potassium montmorillonite at a void ratio of 7 is approximately 300. It further shows that the permeability of kaolinite can be 1000 times that of montmorillonite. The lower the ion exchange capacity of a soil, the lower, of course, the effect of exchangeable ion on permeability.

There is no term[3] in the permeability equation that directly covers composition or indirectly covers exchangeable ions. A composition term with a value range for each soil mineral group could be added to the equations. Since the mineralogy of a soil is seldom known, the use of such a term might be limited.

EFFECT OF PERMEANT

Michaels and Lin (7) studied the permeability of saturated kaolinite to various fluids. They plotted their results in terms of the absolute permeability, which is equal to the Darcy coefficient of permeability times viscosity over density. If the permeability equations were correct, the absolute permeability of the kaolinite at a given void ratio should be the same for all fluids. Figure 2 clearly shows that this is indeed not true: the permeability of kaolinite is considerably different for the various fluids.

The data in Fig. 2 were obtained from tests in which the kaolinite was molded in the liquid which was to be used as the

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[3] The specific surface reflects, to some extent, composition. The effective surface area is more nearly represented by the total area available for hydration, as indicated by glycol adsorption, than by the external area measured by nitrogen adsorption.

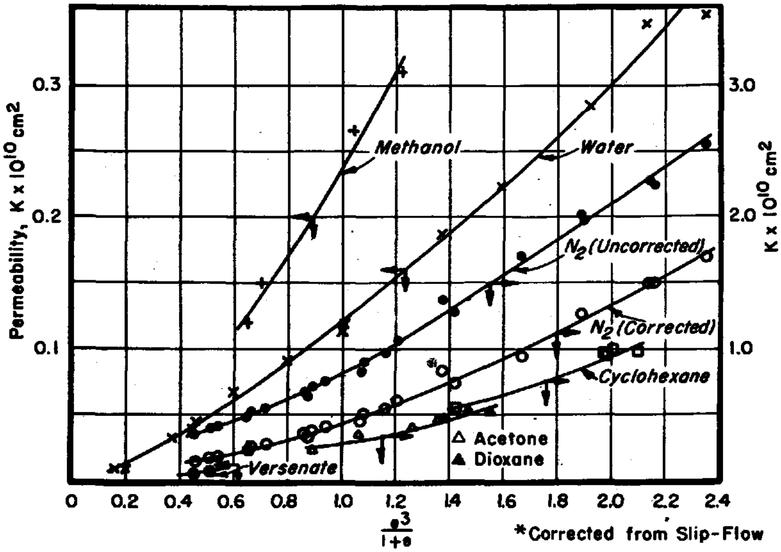

Fig. 2.—Permeability of Kaolinite to Various Fluids as a Function of $\frac{e^3}{1 + e}$ (e = void ratio). From Michaels and Lin (7).

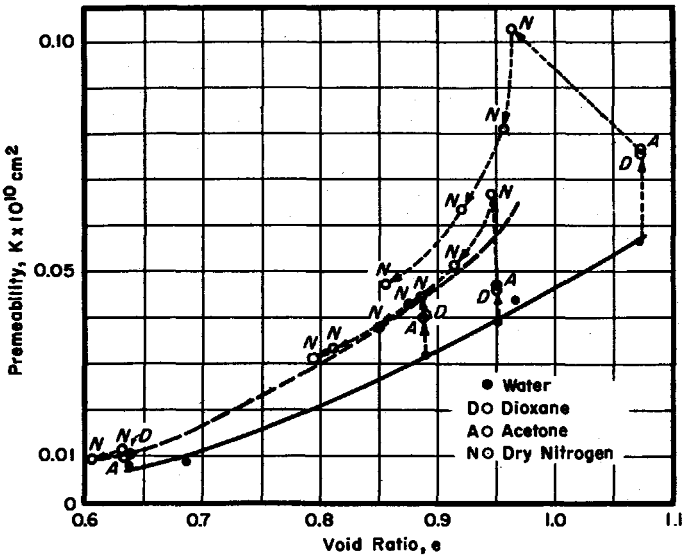

Fig. 3.—Effect of Desolvation on Kaolinite Permeability; Initial Permeant, Water. From Michaels and Lin (7).

permeant. Figure 3 presents results from tests in which water was used as the molding fluid and initial permeant; each succeeding permeant displaced the previous one. Figure 3 shows that, although different permeabilities were obtained for different permeants, the differences are smaller than those in Fig. 2.

Michaels and Lin's work shows that the major effect of permeant on permeability (corrected for permeant density and viscosity) is caused by differences in soil structure (see section on Effect of Structure on Permeability). If the remaining permeant effect were attributed to a layer of immobilized fluid, the kaolinite had an effective thickness of adsorbed water of 50 to 100 Å. Most of this effective thickness was more properly explained by electro-osmotic counterflow.

One must conclude that viscosity and density are not the only permeant characteristics, as indicated by the theoretical equations, that influence the permeability of fine-grained soils. Since the electro-osmotic backflow and thickness of immobilized fluid increase with fluid polarity, some measure of polarity might well be included in the equations. Further research is required to determine whether such improvement in the equations is feasible.

EFFECT OF VOID RATIO

Figure 2 shows that k *versus* $\frac{e^3}{1 + e}$ is not a straight line as the equations indicate. The normal soil testing procedures consider adsorbed fluid as normal pore fluid rather than part of the soil particle. Since the absolute amount of immobilized fluid probably depends on pore size and water content (among other things), the commonly measured void ratio is not equal to the effective one nor is it a constant percentage of it.

Considerable data have shown that generally the plot of void ratio *versus* log permeability approximates a straight line. This relationship holds, of course, only when all other soil characteristics are kept constant, since void ratio is a dependent variable. Data have been presented to show that the nature of the molding fluid has an effect on permeability; data are presented in the next section of this paper to show the influence of amount of molding fluid. In other words, permeability depends not only on void ratio but also on the method by which it is obtained.

EFFECT OF STRUCTURE

Permeability depends to a considerable extent on the arrangement of soil particles, or "structure." The importance of structure on almost all soil properties has been recognized, and theoretical explanations for the role of structure on behavior have been proposed (5). Data on the relationships between permeability and structure have been obtained and are presented here.

Compaction-Permeability Test:

To obtain permeability data on compacted soils, the apparatus shown in Fig. 4 was constructed. The following test procedure was developed and employed:

1. The soil sample was brought to desired water content, mixed with a standard amount of work, and allowed to equilibrate for at least 24 hr.
2. It was compacted in a Harvard Miniature Compaction size mold, and then weighed.
3. The mold was mounted in the apparatus (Fig. 4), the permeant chamber filled, and desired gas pressure applied.
4. Time and flow measurements were taken at various times until a constant rate of flow was reached.
5. The apparatus was disassembled and the sample swell or shrinkage measured (the measurement must be made as soon as possible after the pressure has been released to minimize the effect of the expansion due to pressure release).
6. The specimen was extruded, weighed, dried, and reweighed.

The above procedure has proved to be

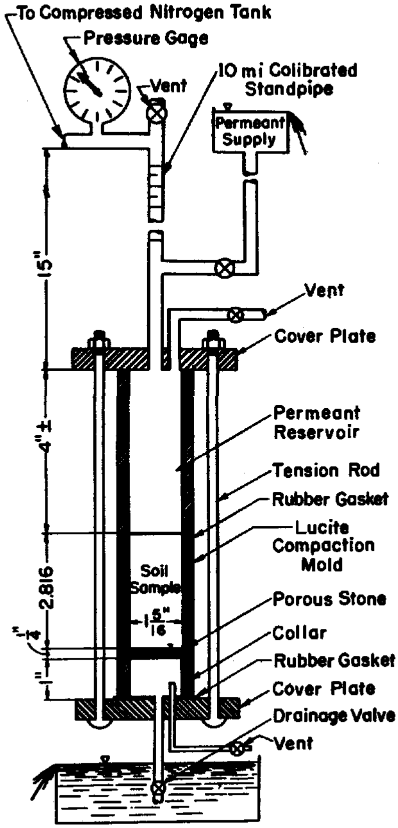

FIG. 4.—Permeability Test Set-Up.

a very rapid and dependable test to measure the compaction and permeability characteristics of a fine-grained soil. The main objections are that deaired water is not used (the use of gas pressure precludes it) and the small sample size leads to data scatter. These disadvantages of the test are considerably more than outweighed by its convenience and general dependability. Unless otherwise noted, it was employed to obtain the data presented and discussed in the remaining portion of this paper.

Influence of Molding Water Content:

Figure 5 presents compacted density and permeability *versus* molding water content for two soils from New Castle, Pa. For soil No. 1 (a silty sand), the minimum permeability occurs at water content at, or slightly greater than, the optimum for compaction. This relationship, rather than the reverse exhibited by soil No. 2 (a sandy silt), is most common (see Wilson (9) and Lambe (3)). The data in Fig. 5 are the molding water content, the as-molded density, and the permeability reached after permeation. Figures 6 and 7 present data on three soils which show how the water content, density, degree of saturation, and permeability change with permeation.

Samples compacted drier than the optimum (Fig. 6) picked up moisture, swelled, and became more nearly saturated upon permeation; samples compacted near optimum showed little change; samples wetter than optimum did not behave consistently. Permeation caused a reduction in permeability for all samples in which fines were not washed out (Fig. 7). Figures 8 and 9, in which data are cross-plotted, show the moisture and density changes caused by permeation.

Figure 6 illustrates a fact not fully appreciated by many experimenters, namely, that the degree of saturation after permeation is usually considerably below 100 per cent.

Sample Mixing:

In general, the more nearly homogeneous an isotropic soil, the lower its permeability. The fines in a soil have a higher impermeabilizing effect (excluding the case of lenses of fines normal to flow) if they are well distributed so they can most effectively plug voids among the larger particles. Mechanical mixing distributes fines.

Mixing also breaks down some of the soil aggregates ("mechanical dispersion"),[4] thereby supplying fines for void

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[4] Remolding a Boston blue clay sample caused a reduction of permeability to a value 1/200 of that of the undisturbed value.

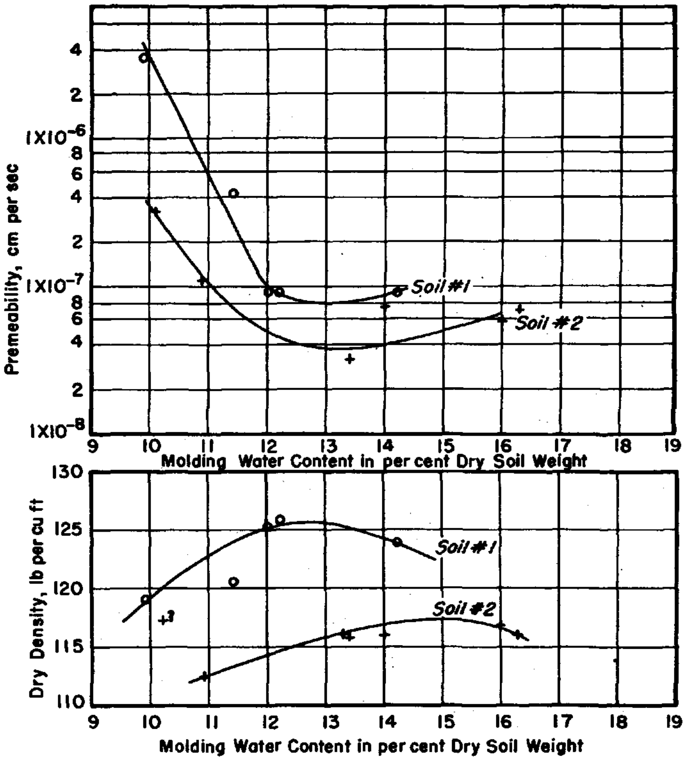

FIG. 5.—Density and Permeability as a Function of Molding Water Content.

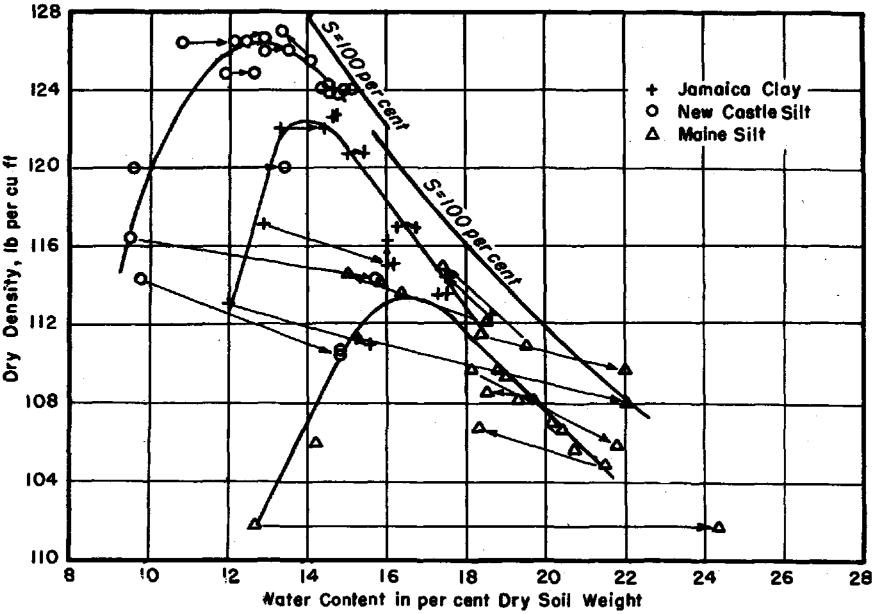

FIG. 6.—Changes in Density and Water Content Caused by Permeation.

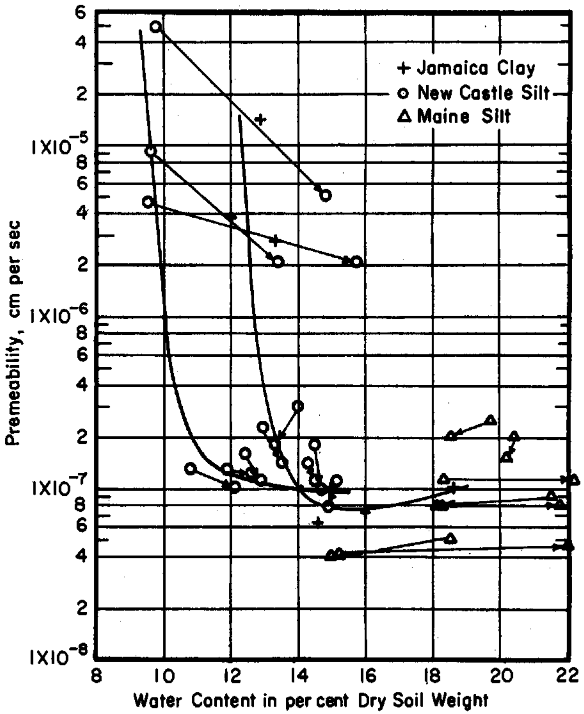

Fig. 7.—Changes in Permeability Caused by Permeation.

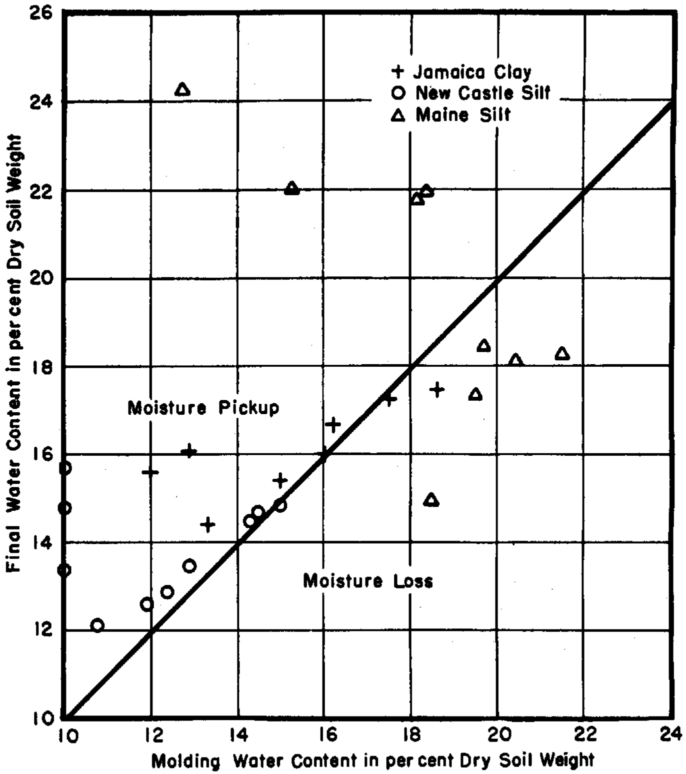

Fig. 8.—Water Content Change Caused by Permeation.

plugging and destroying large voids. Mechanical mixing, because of its blending and dispersing, effects a reduction in soil permeability. Figure 10 illustrates the pronounced influence mixing can have on permeability. The data in the lower curve were obtained on a sample that was initially mixed thoroughly, then reused from test to test. The upper curve was obtained from tests in which a different sample was employed for each determination and each sample was given the same amount of mixing time.

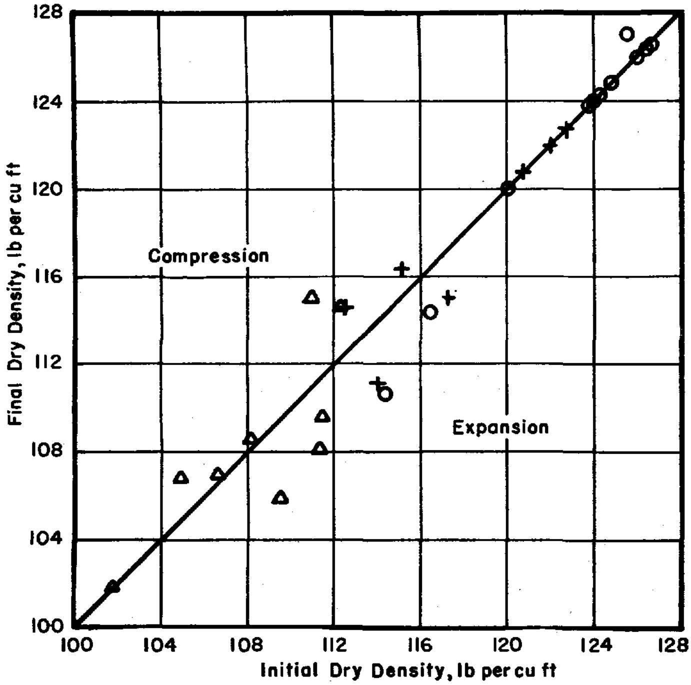

FIG. 9.—Density Change Caused by Permeation.

Effect of Dispersion and Aggregation:

The importance of mechanical dispersion has already been pointed out. Data have been presented elsewhere (3) to show that the minimum permeability of a soil compacted in the presence of a chemical dispersant can be less than one tenth of the minimum of the natural soil. The curves in Fig. 11 show that dispersion reduced the permeability of a soil containing only 8 per cent by weight finer than 0.07 mm.

Since dispersion causes a permeability reduction, aggregation should cause an increase. Figure 12 shows that such an increase does, indeed, occur. The data in Fig. 12 were obtained from two series of tests, one series on an untreated sandy clay and one on the same clay to which 0.05 per cent (of clay dry weight) of sodium polyacrylate had been added. Only one sample was used for each series. Since the sample was densified in the permeameter by vibration, the alteration of structure with densification was minor.

Discussion of Data:

The data presented in this section as well as that in Effect of Permeant on Permeability emphasize the importance

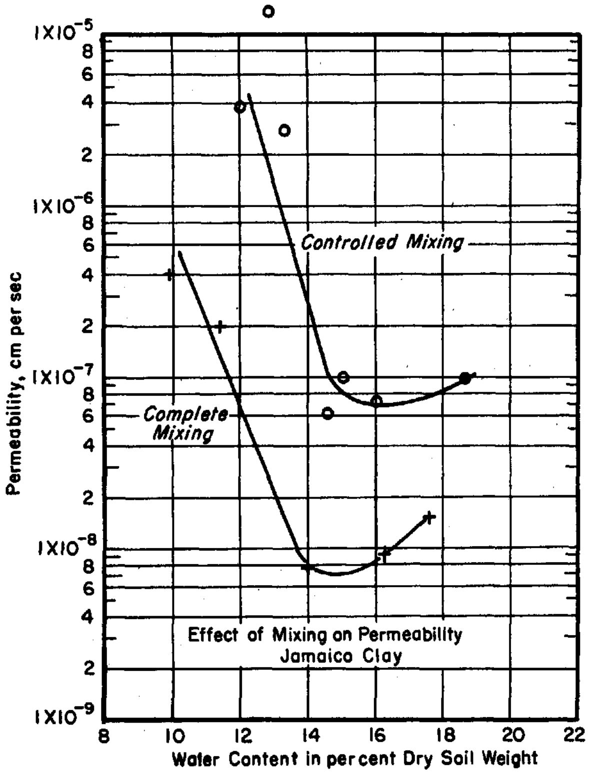

Fig. 10.—Effect of Mixing on Permeability. Jamaica clay.

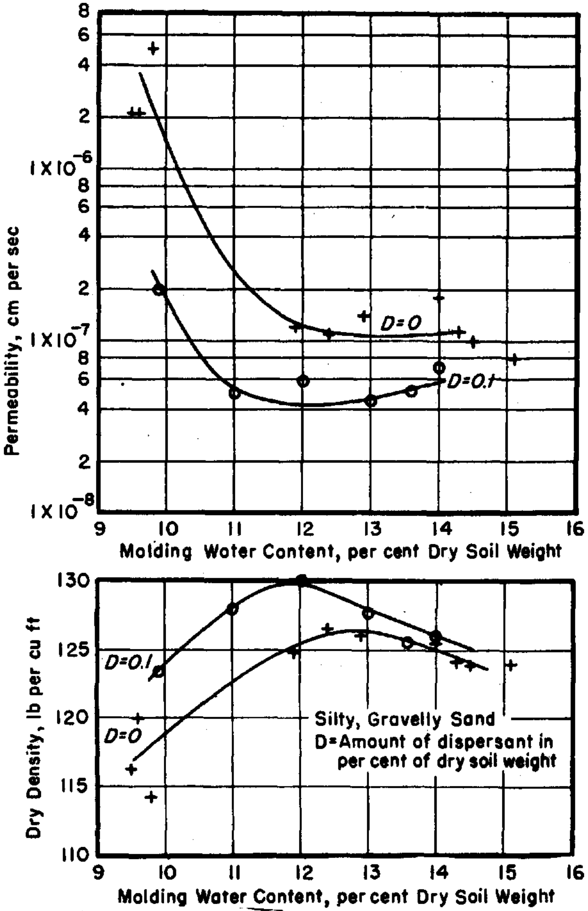

Fig. 11.—Effect of Dispersion on Permeability.

of structure, as the following comparisons illustrate:

Soil	Dry Density or Void Ratio	Degree of Saturation	Permeability, cm per sec	Figure Furnishing Data
Jamaican clay	115 lb per cu ft	Approximately same	4×10^{-6}	6
	116 lb per cu ft		7×10^{-8}	
Virginia sandy clay	1.3	100%	2.7×10^{-4}	12
	1.3	100%	1×10^{-3}	

The first comparison, between a sample compacted dry of optimum and one wet of optimum, shows two samples at essentially the same void ratio and degree of saturation having a permeability ratio of approximately 60. The second comparison, also between samples at the same void ratio and degree of saturation, shows a permeability ratio of greater than 3.

The reduction in permeability that occurs with permeation (Fig. 7) is caused by a change in structure. As flow through a soil occurs, particles tend to move to positions of greater stability to seepage forces. This particle shifting always results in lower permeability, if particles are not washed out of the soil. While this particle shifting can result in either an increase or decrease in density or saturation degree, it usually causes a densification and higher degree of saturation. The permeability reduction has to be explained, therefore, by changes in structure rather than incidental changes in density or degree of saturation.

To evaluate directly a "structure" term for the permeability equations will be exceedingly difficult. Attempts to measure the extent of aggregation have been made (for example, see Michaels and Lambe (6)), but no simple way of giving soil a number to indicate accurately its position in a structure scale has been developed. One is not likely to be developed because of the complex nature of structure. The concept of a scale ranging from 0 for complete dispersion to 100 for complete aggregation is emphasized to some extent at present, even though the best method of determining such numbers has yet to be established.

The permeability of fine-grained soils varies as some power of this "structure coefficient."

EFFECT OF DEGREE OF SATURATION

Soil pores filled with entrapped air are not serving as channels for flowing water. A partially saturated soil is not, therefore, transmitting its maximum amount of liquid during permeation. While this reasoning is commonly accepted, data correlating the degree of saturation with the permeability of fine-grained soils are not readily available.

The influence of degree of saturation on permeability is relatively minor in comparison with composition, structure, and void ratio. Thus, while there are test data available, the effects of degree of saturation are masked, as the data in Figs. 6 and 7 well illustrate. For example, the sample of Maine silt compacted at 20.4 per cent moisture decreased in permeability during permeation even though a *decrease* in density and an *increase* in degree of saturation occurred. The permeability decrease caused by the alteration of structure more than outweighed the combined effects of degree of saturation and density alteration.

SUMMARY

The major factors influencing the permeability of fine-grained soils are: (1)

soil composition, (2) permeant characteristics, (3) void ratio, (4) structure, and (5) degree of saturation. Data are presented to indicate the magnitude of the influence each of these variables can have. In their present state, the theoretical equations showing the relationships between permeability and the characteristics of soil and permeant are of limited practical use in fine-grained soils. These equations can be improved by the addition of terms and by reinterpretation and evaluation of the present equation terms. Considerable research is required to accomplish this goal.

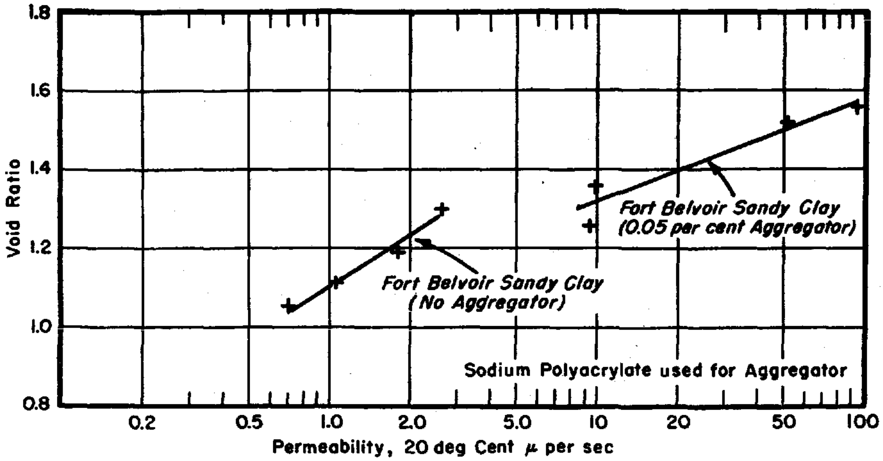

FIG. 12.—Effect of Aggregation on Permeability.

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THE PERMEABILITY AND SETTLEMENT OF LABORATORY SPECIMENS OF SAND AND SAND-GRAVEL MIXTURES

By Chester W. Jones[1]

Synopsis

This paper describes the procedure and results of laboratory permeability-settlement tests on samples from a river deposit containing about half sand and half gravel. The tests were conducted (1) on specimens containing the sand fraction passing the No. 4 sieve, (2) on specimens containing the sand fraction plus various percentages of gravel with ¾-in. maximum particle size, and (3) on specimens containing the sand fraction plus various percentages of gravel with 3-in. maximum particle size. The tests of (1) and (2) above were made on specimens of 8-in. diameter and 5-in. height, whereas those of (3) were made on large-scale specimens of 19-in. diameter and 9-in. height. The specimens were placed at different densities, and settlements were measured during a definite loading schedule during the permeability test.

The results show that the permeability of both the small and large specimens containing 20 to 50 per cent gravel was considerably less than the permeability of the sand fraction alone. In addition, the permeability of specimens containing the ¾-in. material was somewhat greater than those containing the 3-in. material. The permeability decreased with increase in placement density and, with the exception of the specimens containing ¾-in. material, decreased with increasing load. The settlements increased with load and generally increased with gravel content.

The study shows that the differences between the results of the small and large-scale tests are sufficient to justify the use of the latter where a reasonably close value of the permeability is required.

It is often good engineering practice to use soil-gravel mixtures in earth structures where such mixtures occur in deposits near the structure because of the high stability of these materials. In some cases, permeability and settlement are important functions of the structure, and values of these properties are needed for design and construction purposes. The determination of the permeability and the associated settlement of a loaded specimen can be found by laboratory tests. In Bureau of Reclamation work prior to 1940, permeability testing was conducted only on the soil fraction (minus No. 4 material) and, when soil-gravel mixtures were encountered, the effect of the presence of the gravel was either disregarded or at best roughly estimated.

About 1940, the Bureau laboratories constructed some large-scale laboratory apparatus for conducting loaded perme-

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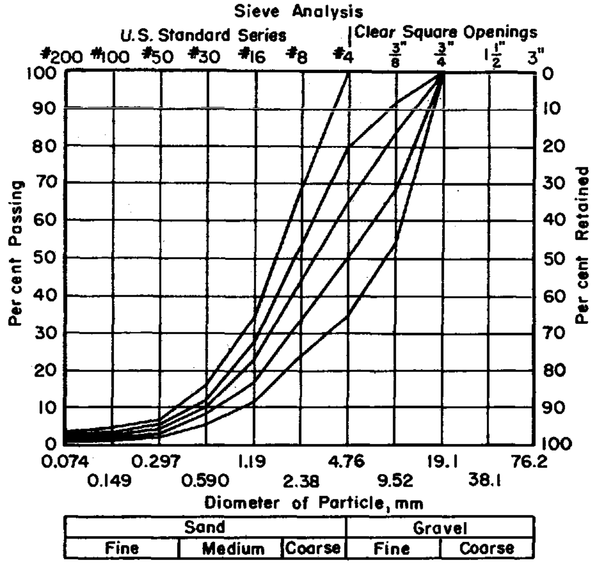

FIG. 1.—Gradation of ¾-in. Maximum Size Material Used in Small Permeability-Settlement Test Specimens.

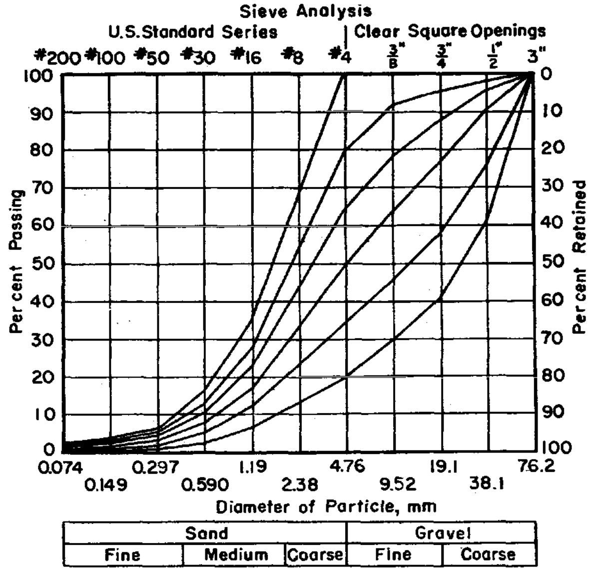

FIG. 2.—Gradation of 3-in. Maximum Size Material Used in Large Permeability-Settlement Test Specimens.

ability tests on specimens containing soil-gravel mixtures, and the apparatus has since been in nearly constant use.[2] Large-scale types of tests such as these are necessarily more expensive to make than smaller tests, and questions have often arisen as to the differences in results of the two types of tests and whether or not any correlation between them exists.

This paper describes the first phase of a Bureau of Reclamation research program involving the conducting of permeability-settlement tests on soil and soil-gravel mixtures in small- and large-scale laboratory apparatus. The materials used were predominantly sand and sand-gravel mixtures and resulted in tests of high permeability. Such material would find common use in the pervious zones of an earth dam or in drainage filters. The ultimate program will include similar tests on other soil-gravel types. The permeability-settlement tests reported herein can be conveniently divided into several series as follows:

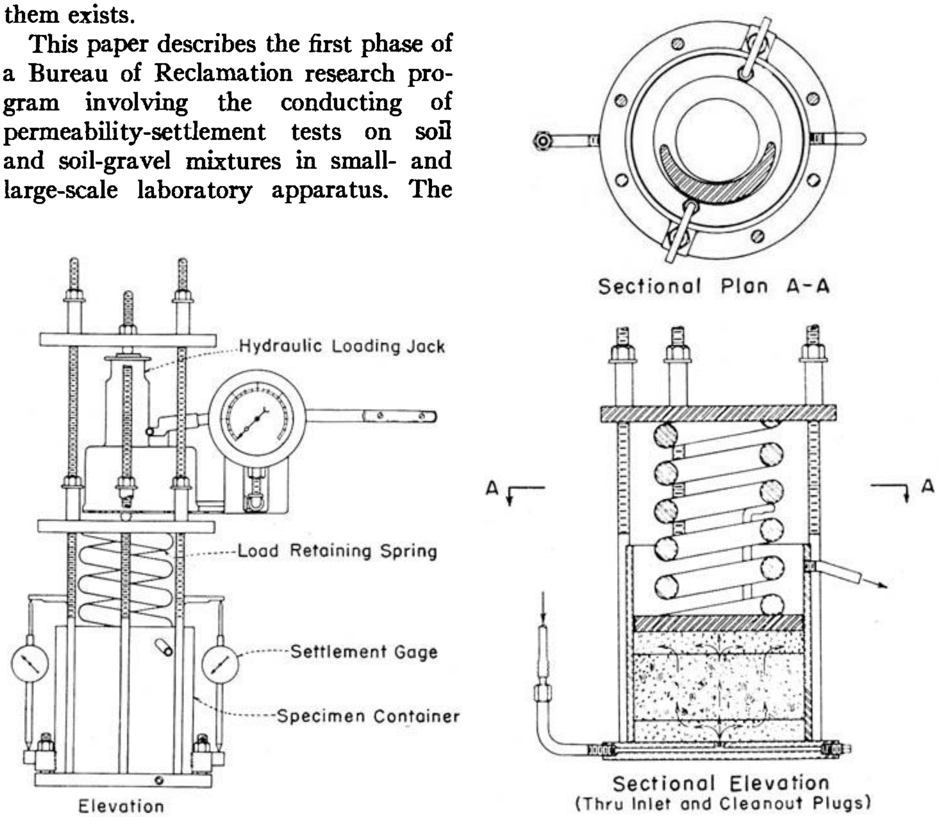

FIG. 3.—8-in. Diameter Laboratory Permeability Apparatus.

1. Tests on the sand fraction passing a No. 4 sieve in specimens of 8-in. diameter and 5-in. height.

2. Tests on the sand fraction plus various percentages of gravel of $\frac{3}{4}$-in. maximum particle size in specimens of 8-in. diameter and 5-in. height.

3. Tests on the sand fraction plus various percentages of gravel of 3-in.

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[2] W. G. Holtz and V. S. Meissner, "Suggested Method of Test for Permeability and Settlement of Earth Materials Containing Particles Up to Three Inches in Size," Procedures for Testing Soils, Am. Soc. Testing Mats., July, 1950, p. 184.

maximum particle size in specimens of 19-in. diameter and 9-in. height.

Each of the above tests was performed at three different density conditions, with all specimens in each test series being placed at similar relative densities. The specimens were subjected to a definite system of increasing loads during the permeability tests. The resulting recorded settlements are presented.

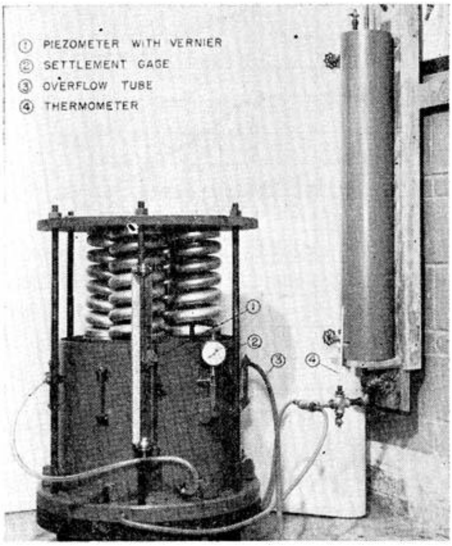

FIG. 4.—Large 19-in. Diameter Laboratory Permeability Apparatus.

MATERIALS TESTED

The source of the materials used was a deposit at a commercial sand and gravel plant located on the South Platte River in Denver, Colo. The sand and gravel particles were generally of igneous types. The particles were subangular to subrounded with the angularity becoming more pronounced with the degree of fineness.

The grading of the sand fraction, together with the gradings of the materials used in the $\frac{3}{4}$-in. series of tests, is shown in Fig. 1. The grading of the materials used in the 3-in. series of tests is shown in Fig. 2. In both series the gravel content was varied by percentage increments of 0, 20, 35, 50, and 65. In a few instances of the larger specimens, 80 per cent gravel contents were used.

TEST APPARATUS

A drawing of the permeability-settlement apparatus for the 8-in. diameter specimens is shown in Fig. 3, and a photograph of the large-scale apparatus for 19-in. diameter specimens is seen in Fig. 4. Both tests are identical in principle of operation, but they differ in design to provide the necessary capacity for maximum particle size and of water flow to each size of specimen. The water flow through the specimen is determined by timing the discharge from a volumetrically calibrated head tank which operates on the mariotte principle to provide a constant head of water for the permeability specimen. Practically constant loads were applied to the specimens during the permeability tests by the springs which could be compressed to give the desired loading.

Since the materials tested were relatively pervious, it was necessary to eliminate excess piping of water between the specimen and the inside cylinder wall. After trying various methods of overcoming this, a liner of sponge rubber cemented to the cylinder was found to be most convenient and most successful. Also because of the perviousness of the soil, accurate measurements of low water heads were necessary and, at times, a piezometer with a vernier gage (shown in the photograph of the large-scale apparatus of Fig. 4) was used to enable readings to be made to the nearest 0.01 in.

TEST PROCEDURE

Density Control:

The minus No. 4 material used in these tests was too granular to yield a regular-shaped moisture-density curve from a standard compaction method. For this reason the relative density method of control for placement densities was used. This method involves the determination of maximum and minimum densities[3] by definite laboratory procedures and the computation and placement of desired relative densities between the two limits. The maximum density is found by vibrating to constant density the saturated material in a container of known volume and determining the dry unit weight. The minimum density is found by determining the dry unit weight of material poured in a dry condition into a container of known volume. The results of these tests are shown in the plot of Fig. 5.

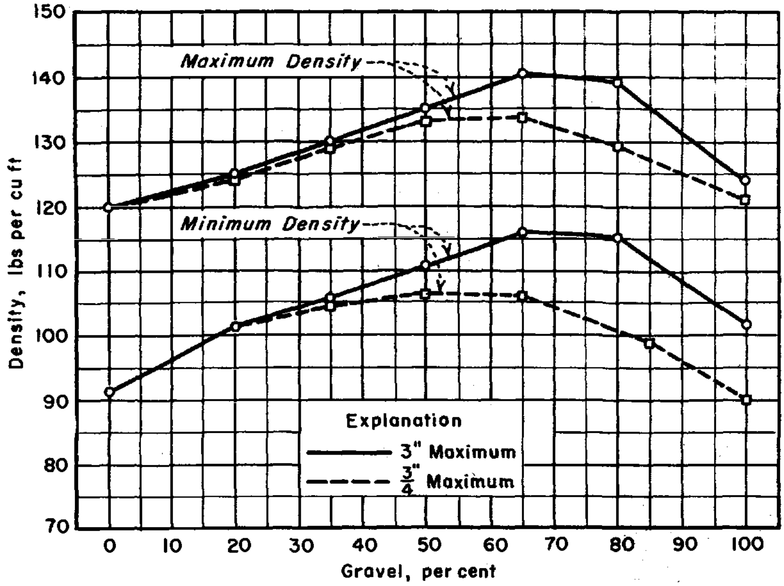

FIG. 5.—Maximum and Minimum Densities of Sand and Sand-Gravel Mixtures.

TABLE I.—SOIL PLACEMENT DENSITIES AND VOID RATIOS OF PERMEABILITY TEST SPECIMENS.

Gravel, per cent	¾-in. Material						3-in. Material					
	50% Relative Density		60% Relative Density		70% Relative Density		50% Relative Density		60% Relative Density		70% Relative Density	
	Density[a]	Void Ratio	Density	Void Ratio	Density	Void Ratio	Density	Void Ratio	Density	Void Ratio	Density	Void Ratio
0.......	103.8	0.575	106.6	0.534	109.7	0.490	103.8	0.575	106.6	0.534	109.7	0.490
20.......	111.7	0.464	114.0	0.435	116.3	0.406	111.9	0.462	114.2	0.433	116.7	0.401
35.......	115.4	0.416	117.8	0.387	120.4	0.359	117.2	0.395	119.5	0.368	122.0	0.340
50.......	118.8	0.376	121.5	0.346	124.2	0.316	121.7	0.344	124.2	0.316	126.8	0.289
65.......	118.3	0.381	121.0	0.351	123.9	0.319	127.2	0.285	129.7	0.261	132.4	0.235
80.......	112.2	0.458	115.2	0.418	118.4	0.381	125.8	0.300	128.2	0.276	130.8	0.250

[a] Density in pounds per cubic foot.

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[3] Designation E-11, U. S. Bureau of Reclamation *Earth Manual* (1951). A manual on the use of earth materials for foundation and construction purposes.

The placement densities used for the test specimens were 50, 60, and 70 per cent relative density as computed from the following formula:

$$\gamma = \frac{(\gamma_{max})(\gamma_{min})}{\gamma_{max} - D_r(\gamma_{max} - \gamma_{min})}$$

where:

γ = placement density,
γ_{max} = maximum density,
γ_{min} = minimum density, and
D_r = percentage relative density desired (expressed as a decimal).

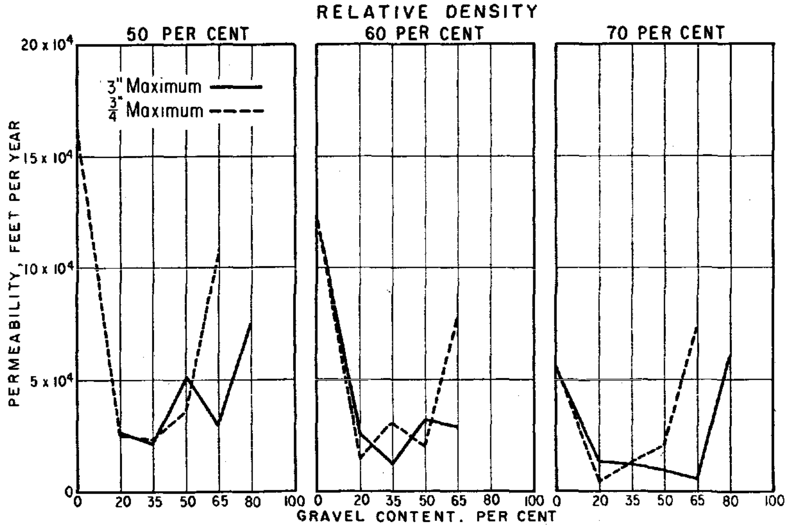

FIG. 6.—Relationship of Permeability to Gravel Content for Specimens with Variation in Density.

The specimen placement densities computed as shown above and the corresponding void ratios are listed in Table I. These densities are in the same range of values as those one could reasonably expect of similar material in a compacted earth structure.

Specimen Preparation:

For each specimen of sand or sand-gravel mixture, material sufficient to make the desired density and volume was first weighed. The gravel, which was kept separate until time for specimen placement, was soaked in pans of water overnight and then surfaced-dried prior to combination with the sand (minus No. 4 fraction). The sand fraction was moistened sufficiently to facilitate specimen compaction. The sand and gravel fractions were combined and tamped to desired density in layers in the permeability cylinders. The specimens containing no gravel were placed in five 1-in. layers, those with ¾-in. maximum size gravel in three 1.67-in. layers, and those with 3-in. maximum size gravel in two 4.5-in. layers.

Specimen Loading:

The 8-in. diameter permeability specimens were loaded by exerting a force on the springs over the specimen by means of a hydraulic jack between the plates above the specimen (see Fig. 3). The load was maintained by tightening the nuts on the tension rods against the steel plate between the jack and the springs. The 19-in. diameter permeability specimens were loaded in a large universal testing machine.

The sequence of loads for the specimens during the permeability test was representative of 20, 40, 80, and 160 ft of overburden. For this purpose the density of overburden was assumed to be 130 lb per cu ft; thus the overburden figures are approximately equal to the load expressed in pound per square inch.

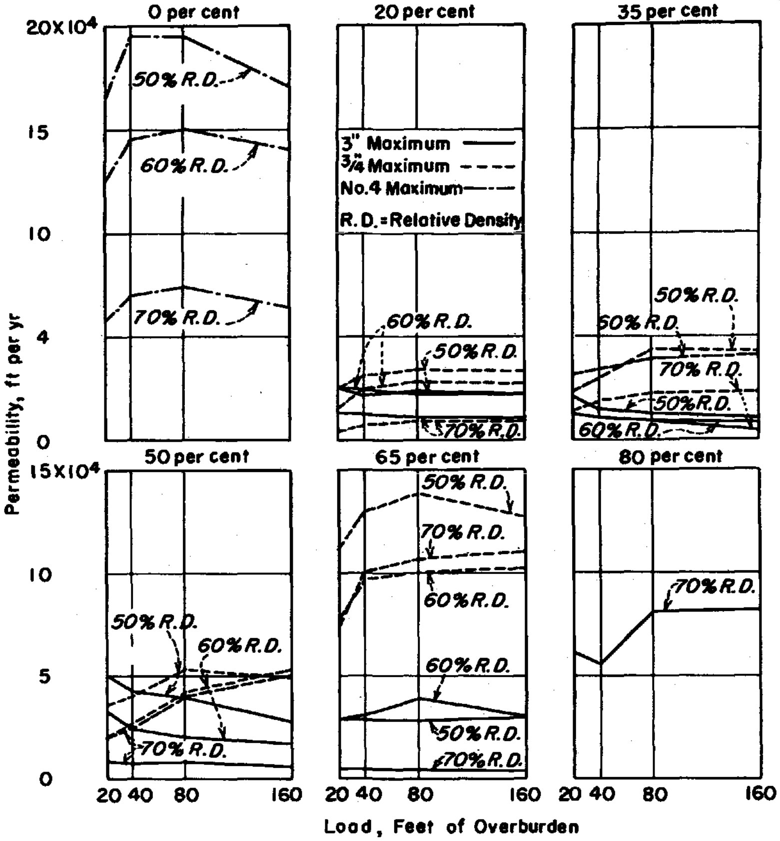

FIG. 7.—Relationship of Permeability to Load for Specimens with Variation in Gravel Content and Density.

Permeability Testing:

The permeability test was continued under each loading from one to several weeks until the flow of water became reasonably constant. Settlement readings were made at intervals during the permeability test by means of dial gages between reference points located on opposite sides of the permeability cylinder (see Figs. 3 and 4).

The water used for the permeability tests was taken from the cold tap of the Denver city water system. The water for the tests was drawn into a reservoir located near the laboratory ceiling and was allowed to remain there sufficiently long so that the water temperature was slightly above the temperature of the soil specimens which were located near the laboratory floor. This procedure was followed to cause a slightly falling temperature gradient through the soil specimen to avoid deposition of air in the soil voids from that dissolved in the water.

TEST RESULTS

Permeability Tests:

The range of permeabilities for the tests reported herein extend from about 5000 to 200,000 ft per year. In all cases duplicate specimens placed under as nearly similar conditions as possible were tested, and the results shown in Figs. 6 and 7 are the averages of the duplicate specimens. The percentage difference of permeability values of individual specimens from the average of duplicates varied from 0 to 72 per cent with the over-all difference of 18 per cent. There is a slight tendency for this percentage difference to decrease with increase in load but no definite tendency for it to increase or decrease with the specimen density or gravel content. In permeability determinations on material as pervious as that used in the tests reported herein, some anomalies are certain to occur. A slightly different arrangement of the particles composing specimens may easily be reflected in permeability and settlement. Therefore, only general trends of the tests are discussed.

The variation of the permeability with gravel content for specimens placed at 50, 60, and 70 per cent relative densities and subjected to a 20-ft overburden load are shown in Fig. 6. The most significant fact apparent is the comparatively lower values of permeability for the specimens containing 20, 35, and 50 per cent gravel compared to those of the specimens containing no gravel. For both the specimens with ¾-in. maximum size gravel and the specimens with 3-in. maximum size gravel, the permeability was reduced to about 10 to 20 per cent of the sand permeability by the addition of 20 per cent gravel. The permeability of the specimens with the ¾-in. maximum size material containing 35 and 50 per cent gravel generally increased somewhat over those containing 20 per cent gravel, then increased greatly for the specimens containing 65 per cent gravel. The permeability of the specimens with the 3-in. maximum size material containing 35 per cent gravel decreased somewhat from those containing 20 per cent gravel and then, with the exception of the specimens placed at 70 per cent relative density, increased considerably for the specimens containing gravel percentages over 35 per cent. For the specimens containing 3-in. particles and placed at 70 per cent relative density, the permeability decreased uniformly with the increase in gravel from 20 to 65 per cent, then increased greatly with those containing 80 per cent gravel.

The variation of permeability with relative placement density for specimens having different gravel contents is shown in Fig. 6. In nearly all cases the permeability of specimens decreased with increase in density and this was particularly noticeable for the specimens containing no gravel.

The variation of permeability with load for specimens containing different gravel contents and placement densities is shown in Fig. 7. The permeability of the specimens containing 3-in. maximum size gravel showed a slight tendency to decrease with increases in load applied during the permeability. However, for the smaller specimens, both for the specimens without gravel as well as those with the ¾-in. maximum size gravel, this tendency was not in evidence. The smaller specimens showed a tendency for the permeability to increase somewhat for an increase in load from 20- to 40- and 80-ft loads and then to decrease for the 160-ft loads, but the permeability at the highest load was almost invariably higher than that for the lowest load.

Specimen Settlement:

The settlement of loaded permeability specimens made from the granular pervious material tested was comparatively low. This fact may explain why the permeability of some of the specimens increased with increase in load, as mentioned in the preceding paragraph. The decrease in void content of the specimens was so small that even a slight rearrangement of the particles in the specimens resulted in an increase in permeability, rather than a decrease as might be expected.

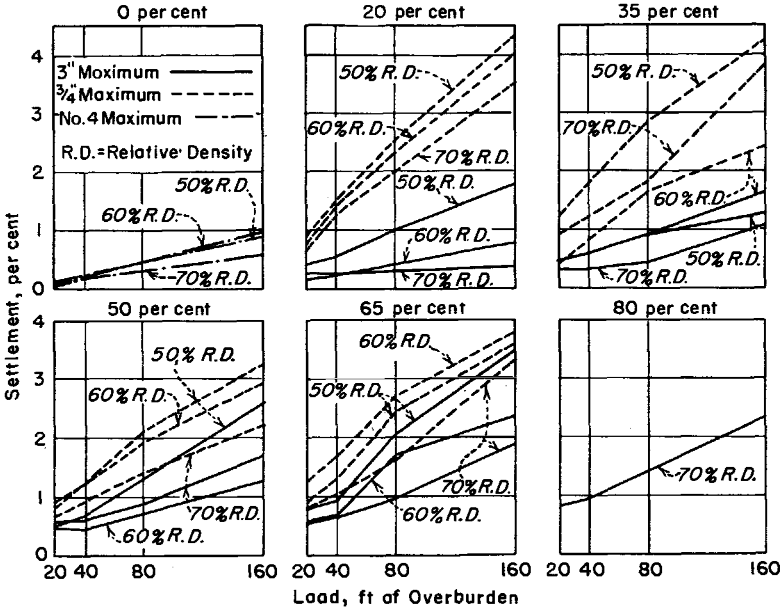

FIG. 8.—Relationship of Settlement to Load for Specimens with Variation in Gravel Content and Density.

The plot in Fig. 8 shows the variation of settlement to load for specimens with different gravel contents and densities. The settlement increased approximately linearly with increase in load. The settlement of the specimens with $\frac{3}{4}$-in. maximum size gravel was greater than (in some cases about twice) that of the specimens with 3-in. maximum size gravel.

In Fig. 9 the plot shows the relationship of settlement to gravel content for specimens varying in load and density. This shows that the specimens containing no gravel settled considerably less than those containing gravel. In general, for the specimens containing $\frac{3}{4}$-in. gravel, the highest settlements occurred in the specimens containing 20 and 35 per cent gravel. For the specimens containing 3-in. gravel, the settlements generally increased with the gravel content up to the highest content used.

A previous study[4] on consolidation of sand-gravel mixtures made in the Bureau

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[4] H. J. Gibbs, "The Effect of Rock Content and Placement Density on Consolidation and Related Pore Pressure in Embankment Construction," *Proceedings*, Am. Soc. Testing Mats., Vol. 50, p. 1343 (1950).

of Reclamation laboratory showed relationships of 1-in. maximum size material to 3-in. maximum size. These comparisons were made at the same placement density for a given percentage of gravel. The 3-in. maximum size material consolidated a greater amount because it was better graded in the coarse particle sizes and was more adapted to densification. Although the settlement for the present study showed greater consolidation for ¾-in. maximum than for the 3-in. maximum, the feature of better adaptability to densification of the better graded 3-in. maximum material is also shown by the higher resulting compacted density for a given per cent of relative density in Fig. 5. Therefore, this basic feature of the two studies has relationship when consideration is given to the different methods of initial placement control. There was considerable difference in the degree of consolidation obtained on the two studies because the previous study involved a plastic soil matrix, whereas the present study involved granular soil only. In the previous study, therefore, gravel had a considerable effect in reducing consolidation, whereas the present study which involved only granular soil showed less effect of gravel content and even resulted in some increase in consolidation with gravel content.

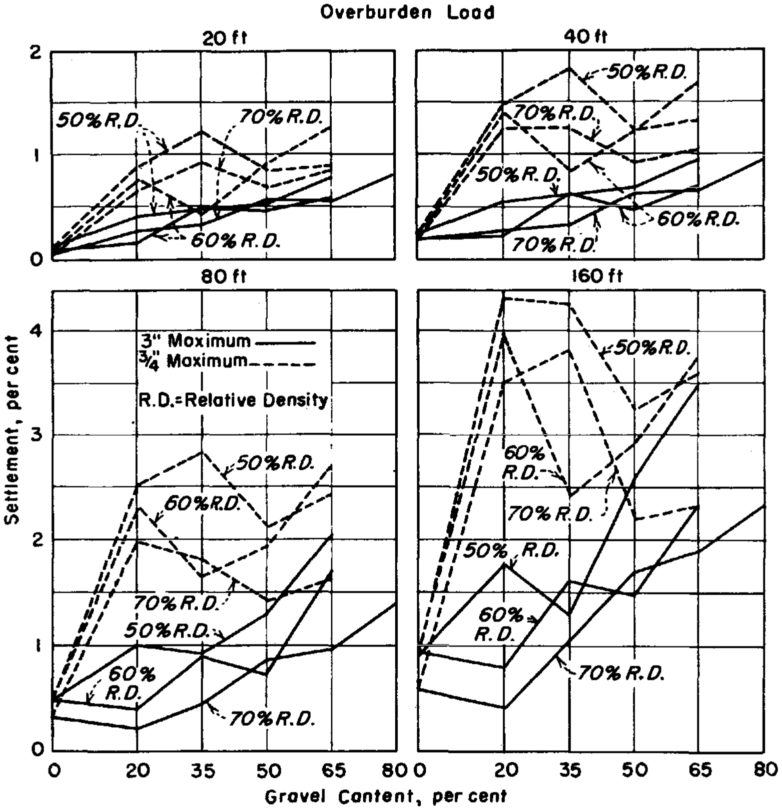

FIG. 9.—Relationship of Settlement to Gravel Content for Specimens with Variation in Load and Density.

CONCLUSIONS

From the results of the laboratory permeability-settlement tests made on the pervious South Platte River sand and sand-gravel mixtures, the following conclusions can be drawn:

1. The permeability of sand-gravel mixtures containing 20 to 50 per cent gravel is considerably lower than that for the sand fraction (minus No. 4 material) alone.

2. The permeability of sand-gravel mixtures containing 3-in. maximum size material in large 19-in. diameter specimens was approximated by small 8-in. diameter specimens containing ¾-in. maximum size material for 20 per cent gravel, but for the higher gravel contents the difference between the two types of specimens was generally large.

3. The permeability of sand and sand-gravel mixtures decreases with increase in placement density in a more or less linear fashion.

4. The permeability of specimens with 3-in. gravel showed a slight tendency to decrease with increase in load. The permeability of small specimens, both with and without gravel, showed a tendency to increase in permeability with load.

5. The effect of the proportion of gravel on the permeability of the sand-gravel mixtures can be stated in the following general terms. For 0 to about 20 per cent gravel, the gravel particles do not interfere with one another to any extent, but they cause the mixture to become better graded and greatly reduce permeability. For about 20 to 65 per cent gravel, although the grading is improved, there is a variable interference among the coarser particles, and the permeability may either increase or decrease a moderate amount. For gravel contents over about 65 per cent, there is major interference of the coarser particles, the density decreases, and the permeability rapidly increases.

6. The settlements of the permeability specimens increased approximately linearly with load, with the settlement of the specimens containing ¾-in. gravel higher, in most cases, than the settlements of the specimens containing 3-in. gravel.

7. The settlement of specimens containing 3-in. gravel increased more or less uniformly with gravel content, whereas, for the specimens containing ¾-in. gravel the settlement was highest for the 20 and 35 per cent gravel.

8. For determinations of permeability and settlement of soils containing gravel, the use of the 3-in. maximum size gravel in 19-in. diameter specimens is justified and recommended. The use of the 8-in. diameter specimens from which material larger than ¾-in. maximum size gravel has been removed will generally result in values of permeability and settlement which are too high.

DISCUSSION

MR. DON KIRKHAM.[1]—The curves of the relationship of permeability to gravel content for mixtures of sand and gravel (Fig. 6) are, at least in part, of the shape one would expect them to have on theoretical grounds. For 100 per cent sand the permeability is a maximum. With the addition of impermeable pieces of gravel, the permeability then falls off until about 35 to 65 per cent of the impermeable pieces of gravel have been added. So far the curves are in accordance with the hydraulic analogue of a theorem in electricity which states that "any increase ... in the specific resistance of any part of a conductor is accompanied by an increase ... of the resistance of the conductor as a whole."[2] As further impermeable pieces of gravel are added to the conducting sand, it should be expected that the permeability would further decrease, whereas, actually, the experiments now show an increase in the permeability. This apparent discrepancy from theoretical expectations may be explained on two grounds. (1) At the higher gravel contents the conducting medium (the sand) did not completely—or at least not so densely as originally—fill the voids between the gravel. (2) The theorem referred to applies to components of a composite medium on a volume basis, whereas the components in the experimental medium here are given on a weight basis.

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[1] Professor of Soils and Physics, Iowa State College, Ames, Iowa.

[2] See, for example, Sir James Jeans, "The Mathematical Theory of Electricity and Magnetism," Cambridge University Press, Cambridge, England, Fifth Edition, p. 349 (1933).

MEASUREMENT OF THE HYDRAULIC CONDUCTIVITY OF SOIL IN PLACE*

By Don Kirkham[1]

Synopsis

Several recently developed and two proposed methods for measuring the hydraulic conductivity of soil in place are described. Particular attention is given to the so-called auger hole, piezometer and tube methods. In the auger hole method an auger hole is bored to a depth below the water table and the water in it is pumped out. After the hole has been emptied, the ensuing rate of rise of water in it is determined and this rate is then converted to the hydraulic conductivity of the soil by use of a suitable formula. In the piezometer and tube methods the procedure is the same except that the auger hole is now effectively cased, and there is only a small cavity at the base of the cased hole into which the rate of water entry is noted. The latter methods have the advantage that they permit the measurement of the hydraulic conductivity at virtually a point in the soil. The auger hole method, on the other hand, gives an average conductivity for the whole length of the hole. Results from the auger hole method are less variable. The problem of soil anisotropy is discussed, and it is pointed out that the anisotropic components of the hydraulic conductivity may be determined by use of the methods.

A knowledge of the hydraulic conductivity (**1**)[2, 3] of soil in place is important in many activities. In agriculture, a knowledge of this soil coefficient is needed to predict rationally the proper depth and spacing of farm drain tiles and drainage ditches; to determine the amount of seepage in and around soil conservation dams; and to predict whether a site for a farm pond will be suitable. More money has been spent on land drainage in Iowa alone than was spent to build the Panama Canal. In activities not necessarily agricultural, questions like the following are asked. How rapidly will an aquifer produce water? How fast will water drain through the subsoil below a proposed highway or a proposed airfield? At what rate should ground water be disposed of about a foundation or in an embankment? The answers to all the above questions depend on a knowledge of the hydraulic conductivity of the soil.

It has been common practice, in getting answers to the above questions, to take soil cores or bulk soil to the labora-

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* Journal Paper No. J-2505 of the Iowa Agricultural Experiment Station, Ames, Iowa, Project 998.

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[2] The boldface numbers in parenthese refer to the list of references appended to this paper, see p. 96.

[3] The term "hydraulic conductivity" is used, rather than "permeability," etc., in keeping with a recommendation of a committee of the Soil Science Society of America (**1**). The term hydraulic conductivity may be defined as the coefficient k in Darcy's law, $v = ki$, v being the velocity of seepage and i the hydraulic gradient. Values of k depend on properties of the fluid, as well as of the porous medium, and reflect any interactions of the fluid with the porous medium such as swelling of soil and its attendant reduced porosity. The term permeability is reserved to apply to that (linear) impedance characteristic of porous media which is independent of the viscosity, etc., of the fluids.

tory for permeameter measurements. Such measurements, because of the disturbed nature of the samples, may not indicate the true value of the hydraulic conductivity of the soil in place. In the present paper, methods for measuring the hydraulic conductivity of soil in place are presented which are intended to obviate problems associated with core and bulk samples, such as:

1. Core and bulk samples are small compared to the mass of soil of interest. Large numbers of the samples must be taken for any intended true representation of conditions. The smallness of core samples may be especially significant in soils with much clay. In these soils, drainage may occur largely through cracks and channels (2) whereas core samples, since they are small, might not include the cracks and channels at all. In fact, workers often discard core samples with cracks as being unrepresentative of true soil conditions.
2. Core samples may be compressed when taken, thereby lowering the true hydraulic conductivity of the soil as found in place.
3. If the soil is too dry, core samples will shatter. The natural, water-conducting channels in the soil will no longer exist.
4. If the sample is taken too wet, the soil will puddle. Natural channels will be sealed.
5. Even if a "good" core is taken—a core obtained when the soil is not too wet or too dry—a conductivity determination made in the laboratory may still not represent the water-saturated, field condition because air may become trapped in the laboratory samples to result in the explosive (3) break down of natural channels in the soil.
6. The trapped air in laboratory samples, aside from, or in addition to, the explosive effects just mentioned, may block channels and greatly increase the impedance of the tube to flow, or block the flow almost completely (4). Variability in hydraulic conductivity measurements, to the extent of 3000 per cent, has been attributed to the presence of trapped air in cores (5).
7. The chemical nature of the water used in the laboratory may differ from that of the ground water of the soil in place. The resulting deflocculation or flocculation of soil material may produce channel sizes in the cores, quite different from the sizes in the natural soil (6, 7, 8).

The above list, although not complete, shows that there is a real place for field methods. In the following, the so-called auger hole, piezometer and tube field methods will receive major attention. These methods may all be designated as single-cavity methods. A recent two-cavity (two-well) method will be described. Mention will be made of some older field methods. A proposed four-well method and a proposed single-well method, both as yet untried experimentally, but of possible interest for further study and development, will be described; a dry auger hole method and some methods depending on the discharge of drain tubes or ditches will be noted.

SINGLE-CAVITY METHODS

The auger hole method, the piezometer method, and the tube method are similar in that the following procedure applies to all of them (9).

A cavity is bored into the soil to a depth below the water table. The water seeps into the cavity and rises to an equilibrium level, which is the water table. This level may be reached in a few minutes or a number of hours, depending on the tightness of the soil. The equilibrium level is recorded. The cavity is pumped out a few times to permit any puddled-over pores along the wall of the cavity to be flushed out by the in-seeping ground water. After a last pumping (2 or 3 pumpings generally suffice) the time for the water to rise a certain distance in the cavity is observed. The time and the distance of rise are used finally in a suitable formula to yield the hydraulic conductivity of the soil in place.

The three single-cavity methods differ in several respects. In the auger hole method the cavity is uncased. In the piezometer and tube methods, the side walls of the cavity are partially or completely cased. For the piezometer method a 4-in. long cavity at the base of a cased hole 2 in. in diameter is generally used. For the tube method the side walls of the cavity are completely cased. Water can seep in only at the base of the cavity. Since flow can occur along the whole length of the side walls of the auger hole, the auger hole method yields an average value of the hydraulic conductivity for the depth of the soil profile below the water table. The piezometer and tube methods yield essentially "point" values of the hydraulic conductivity in the soil. If the soil is anisotropic, the tube method tends to measure the vertical component of the hydraulic conductivity. The auger hole and piezometer methods tend to measure the horizontal component of the hydraulic conductivity.

It is emphasized that all three cavity methods will work only if there exists a water table, that is, a ground water surface where the pressure is atmospheric. This point, which is not at all trivial, has been enunciated in the form of a law by L. A. Richards (10). The existence of a water table is a necessary but not sufficient condition. Luthin and Day (11) have shown experimentally that water in water-saturated soil will not flow into an auger hole penetrating the soil, if the water table moves vertically downward at a velocity equal to the quotient of the hydraulic conductivity and the soil porosity. The hydraulic gradient for this condition is unity and the soil water pressure just atmospheric.

Aside from the need of a water table, there is another, more immediately apparent, limitation on the cavity methods: the soils in which the methods are used must support a cavity. That is, the cavity must be fairly well defined. The cavity walls may not break down. In sandy soils, unless the cavities have a screened liner, or its equivalent, the methods may fail. It has been found that a number of sandy soils have sufficient clay to support a cavity without the use of liners. Cavities will not tend to break down if they are only partially pumped out. It is not necessary to pump out the cavities completely in order for the methods to work. Hvorslev (12) has pointed out some problems associated with cavities.

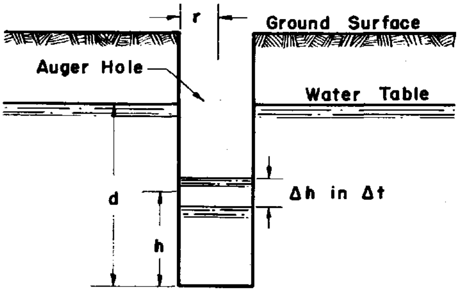

FIG. 1.—Auger Hole Method for Determining Hydraulic Conductivity of Soil.

AUGER HOLE METHOD

The simplest method one can probably imagine for measuring the hydraulic conductivity of soil in place is to bore a post hole into the soil below the water table and observe the rate at which ground water will enter the hole. Diserens (13) was apparently the first to report this method. Hooghoudt (14) used it further and gave an approximate analysis of the problem. His name is associated with the method in the Netherlands, where the method is extensively used. Hooghoudt assumed in his analysis that water entered the hole horizontally from the sides and vertically from below. The assumptions obviously do not represent the truth. Nevertheless, they yielded approximately correct results.

Figure 1 illustrates the auger hole method. The hole is bored to a depth d below the water table. The time Δt needed for the water to rise a distance Δh in the hole, and the average level h of the water in the hole for the time interval Δt, are recorded. The hydraulic conductivity k is then determined from the relation (15):

$$k = 0.617 \frac{r}{Sd} \frac{\Delta h}{\Delta t} \qquad \ldots\ldots\ldots\ (1)$$

where S is a geometrical function given by

$$\begin{aligned} S = {} & \cos \frac{\pi h}{2d} \cdot \frac{K_1(\pi r/2d)}{K_0(\pi r/2d)} \\ & - \frac{1}{3^2} \cos \frac{3\pi h}{2d} \cdot \frac{K_1(3\pi r/2d)}{K_0(3\pi r/2d)} \\ & + \frac{1}{5^2} \cos \frac{5\pi h}{2d} \cdot \frac{K_1(5\pi r/2d)}{K_0(5\pi r/2d)} - \cdots \qquad ..(2) \end{aligned}$$

in which $K_0(\pi r/2d)$ and $K_1(\pi r/2d)$, etc., are Bessel functions defined by the infinite series (16):

$$\begin{aligned} K_0(x) = {} & -\left\{\left[0.5772 + \log_e \frac{x}{2}\right]\right. \\ & \left.\left[1 + \frac{x^2}{2^2(1)^2} + \frac{x^4}{2^4(1\cdot2)^2} + \frac{x^6}{2^6(1\cdot2\cdot3)^2} + \cdots\right]\right\} \\ & + \left\{\frac{1}{(1)^2}\frac{x^2}{2^2}(1)\right\} \\ & + \left\{\frac{1}{(1\cdot2)^2}\frac{x^4}{2^4}\left(1 + \frac{1}{2}\right)\right\} \\ & + \left\{\frac{1}{(1\cdot2\cdot3)^2}\frac{x^6}{2^6}\left(1 + \frac{1}{2} + \frac{1}{3}\right)\right\} + \cdots \qquad ..(3a) \end{aligned}$$

$$\begin{aligned} K_1(x) = {} & \left\{\left[0.5772 + \log_e \frac{x}{2}\right]\right. \\ & \left[\frac{x}{2} + \frac{x^3}{2^3 1\cdot1\cdot2} + \frac{x^5}{2^5 1\cdot2\cdot1\cdot2\cdot3}\right. \\ & \left.\left.+ \frac{x^7}{2^7 1\cdot2\cdot3\cdot1\cdot2\cdot3\cdot4} + \cdots\right]\right\} \\ & + \frac{1}{x} - \left\{\frac{x}{2}\left(1 - \frac{1}{2}\right)\right\} \\ & - \left\{\frac{1}{1\cdot1\cdot2}\left(\frac{x}{2}\right)^3\left(1 + \frac{1}{2} - \frac{1}{4}\right)\right\} \\ & - \left\{\frac{1}{1\cdot2\cdot1\cdot2\cdot3}\left(\frac{x}{2}\right)^5\left(1 + \frac{1}{2} + \frac{1}{3} - \frac{1}{6}\right)\right\} \\ & - \left\{\frac{1}{1\cdot2\cdot3\cdot1\cdot2\cdot3\cdot4}\left(\frac{x}{2}\right)^7\right. \\ & \left.\left(1 + \frac{1}{2} + \frac{1}{3} + \frac{1}{4} - \frac{1}{8}\right)\right\} - \cdots \qquad (3b) \end{aligned}$$

It has seemed worthwhile to present the above Eqs. *3a* and *3b*, as the author knows of no place in the literature where these functions, except in compressed notation, are displayed. Only the three terms shown in the right-hand side of Eq 2 are needed for practical work.

To aid in the practical use of Eq 1, Spangler (17), utilizing values of $K_0(x)$ and $K_1(x)$ tabulated in the British Association Tables (16), has prepared a chart of values of S. This chart is reproduced as Fig. 2.

Strictly speaking, Eq 1 is valid only if an impervious layer coincides with the bottom of the auger hole. If such a layer does not so coincide, then Eq 1 yields an approximate value of the hydraulic conductivity, the approximation becoming better as the ratio d/r becomes greater. If the distance s of the impermeable layer below the base of the auger hole is known, then S in Eq 1 should be replaced by a geometrical factor S'

which would be a function of s, as well as of r, d, and h. Unfortunately this factor S' cannot now be given except for a few values of r/d, h/d, and s/d. Values which can be given are based on work with electric models (18)[4] and are therefore subject to experimental error. When further values of S' are available (theoretical values would be preferable as they are not subject to experimental error), charts like Fig. 1 for $s/d = 0.25$, 0.5, 1, 2, and ∞, should be completed. Figure 1 covers the case $s/d = 0$. The case $s/d = \infty$ should not depart much from that of $s/d = 2$. L. F. Ernst of the Soil Science Inst., T.N.O., Groningen, The Netherlands, has prepared some unpublished charts from which some values of S' could be obtained.

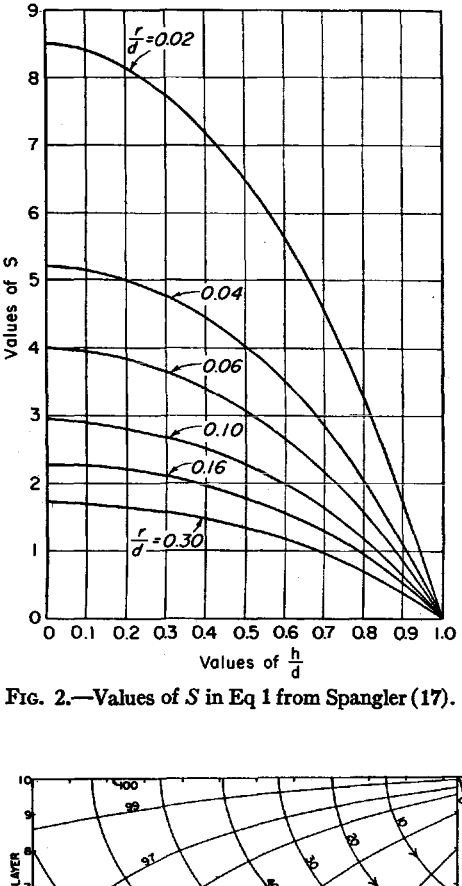

FIG. 2.—Values of S in Eq 1 from Spangler (17).

Figures 3 and 4 show flow nets for the

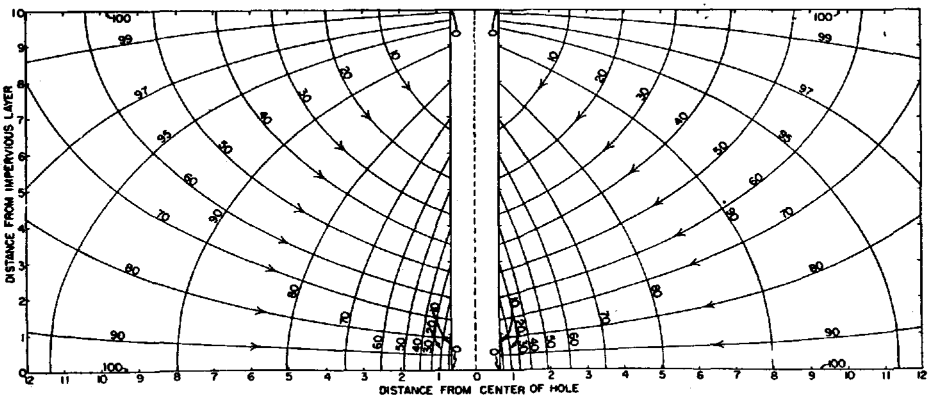

FIG. 3.—Lines of Equal Hydraulic Head (Equipotentials) and Streamlines in Soil about an Emptied Auger Hole.

The hole extends from a water table to an impervious layer, the soil extending horizontally to infinity. The hydraulic head, referred to the level of the impervious layer, is taken 0 at the bottom of the hole and 100 at the water table. The zero-streamline (point) is taken at the top of the hole and the 100-streamline (above which 100 per cent of the water enters the hole) at the impervious layer. Distances shown are in arbitrary units. From Kirkham and van Bavel (15).

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[4] To obtain values of S' from reference (18), multiply values of A given in Fig. 4 there by $(\pi/16)(1 - h/d)(A/5)$. If the worker using the auger hole method will agree to keep the value of h/d in Fig. 1 less than 0.2 and will also agree to use an auger hole of 4-in. diameter, then the effect of an impermeable layer can be taken into account by utilization of a chart prepared by Johnson, Frevert, and Evans (19, Fig. 4). The A in this chart is the A just noted.

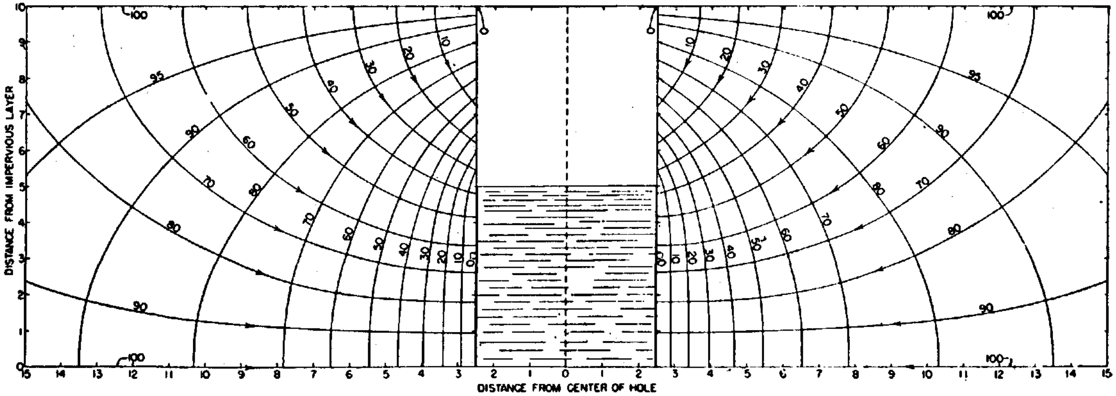

FIG. 4.—Equipotentials and Streamlines, as in Fig. 3 Except Hole is Wider and Half Full of Water. From Kirkham and van Bavel (15).

auger hole problem. In Fig. 3 the hole has been pumped out completely; in Fig. 4 it has been pumped out to half its depth. The flow nets show at a glance how water enters the hole and how the driving hydraulic head is dissipated. The streamlines (arrows) show the paths of water flow. The equipotentials, the lines everywhere perpendicular to the streamlines, show how the hydraulic head is dissipated. With the aid of these flow nets it has been possible to show that if Δh or the sum of several Δh's in Fig. 1 is kept such that h stays less than $\frac{1}{4}$ d, then the effect of a cone of depression (which will always develop to a greater or lesser extent about the top of the auger hole as the hole fills) will have negligible influence on the validity of Eq 1.

Aside from the effect of the cone of depression to introduce error in Eq 1, there is a further, theoretically more obvious, error in the equation, namely, use of $\Delta h/\Delta t$ for dh/dt. But calculations show that very little error, less than 2 or 3 per cent, will result through use of finite values of Δh and Δt, instead of the limiting form dh/dt, provided Δh is kept less than the value $\frac{1}{4}$ d, as described above.

PIEZOMETER METHOD

It has been pointed out that the auger hole method gives an average value of the hydraulic conductivity of the soil over the length of the hole below the water table. Many soils are stratified. One stratum may be highly permeable compared to the others. For this reason the auger hole method will fail to give a correct picture of the way the hydraulic conductivity varies with depth. The piezometer method (Fig. 5) overcomes the difficulty. In this method one obtains, as has been mentioned, essentially the hydraulic conductivity of the soil at a point. This is possible because the hole which is bored into the soil for the measurement is cased, except for a small cavity at its end. The rate of entry into this cavity is a measure of the hydraulic conductivity of the soil immediately around the cavity—not over the whole length of the hole.

A specific example of the piezometer method is of interest. After trash and the surface sod have been removed from the location in question, a hole $1\frac{15}{16}$ in. in diameter is bored to a depth of 6 in. below the ground surface. A length of 2-in. inside diameter, thin-walled, electrical conduit, hereafter called the piezometer, sharpened on one end, is driven into the hole to a depth of 5 in. with light blows from a maul. A driving head is used on the top of the piezometer to prevent its damage. The $1\frac{15}{16}$-in. soil auger is then inserted into the piezometer and the soil is removed to a depth 6 in. below its sharpened end. The piezometer is again driven 5 in. deeper. This procedure is continued until the sharpened end of the piezometer is at the desired depth of measurement. The procedure thus far is designed to prevent compaction of the soil. At the same time, because of the differential diameters of auger and piezometer, the necessary tight fit of the piezometer with the soil is obtained. Leaks along the outside of the piezometer would vitiate the method.

A cavity 4 in. in length and $1\frac{15}{16}$ in. in diameter is now carefully augered below the bottom end of the piezometer. A stop on the auger is provided to make this 4-in. depth precise. The auger is also provided with a hollow shaft through which air can enter to prevent the development of suction at the base of the auger. Suction, if not released, would, upon withdrawal of the auger, pull the soil on the auger thread back into the cavity. Also, without the release of suction, a larger cavity than the intended one could result. It should be recognized here that, when boring is done below the water table, atmospheric air cannot enter the bored out cavity, except by artificial means.

After the cavity has been prepared, water seeping into it is pumped out two or three times to remove the effect of puddled soil on the cavity walls. A small pitcher pump with a flexible hose to go to the tube bottom is used. When, after a pumping, the rate of rise of water in the conduit is the same as for a preceding pumping, the effect of puddling is assumed to be absent.

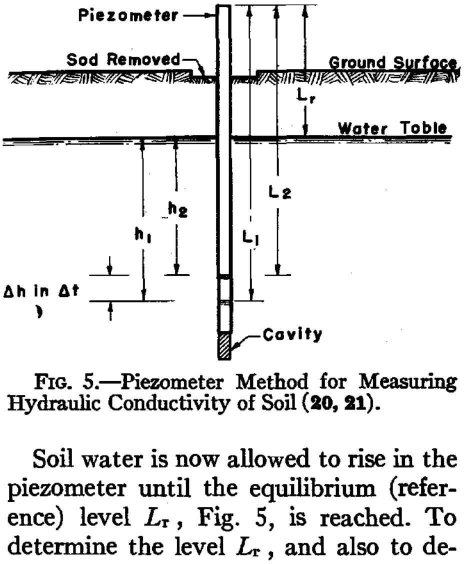

FIG. 5.—Piezometer Method for Measuring Hydraulic Conductivity of Soil (**20, 21**).

Soil water is now allowed to rise in the piezometer until the equilibrium (reference) level L_r, Fig. 5, is reached. To determine the level L_r, and also to determine required values of L_1 and L_2 (Fig. 5), a reel-type electric probe similar to the one described by Luthin (22) is used.

The electric probe (Fig. 6) consists of a rubber-insulated, copper wire wound upon a clutch type (Pflueger Sal-Trout No. 1558) fishing reel secured to a post on a stand, the stand in turn resting stably on the piezometer. An inverted cup arrangement on the bottom of the stand fits over the top of the stand to provide for the stability. The end of the insulated copper wire leading from the reel into the piezometer is encased, for weight, in a sleeve of lead, prepared by pouring lead inside a further encasing, insulating, plastic cylinder. The other end of the wire makes contact, through the metal reel clutch, with one side of a current-limiting resistor. The other side of the resistor is series-connected to a milliammeter, a 33-v hearing aid battery, and to "ground." Ground can be the inverted (brass) cup, since it makes contact with the top of the piezometer. When the weighted end of the reel wire touches the water, current flows through the circuit, and the needle of the milliammeter, serving as a "yes-no" meter, deflects. The depth of the probe is then established by means of two scales. One scale is marked on the insulated copper wire at foot intervals. The other scale, which is a 1-ft section of a steel surveyor's scale and is mounted on the reel post, provides for reading the former scale to $\frac{1}{100}$ ft.

Thin, flat, flexible wire for the reel has been found preferable to round wire, mainly because a numbered scale can be engraved on it. Flat 300 ohm television wire is particularly suitable for this purpose.

A battery of fairly high voltage is used with the probe, so that in the case of relatively salt free soils, the value of the resistance through the water part of the circuit will be small compared to that of the limiting resistor. Also use of the 33-v battery makes increased resistances in the circuit due to polarization negligible. Use of a $1\frac{1}{2}$-v battery has not proved generally satisfactory. A hearing aid battery is used because it is light.

At the same time that the electric probe is used to determine the levels L_1 and L_2 for Fig. 5, a stop watch is used to obtain the corresponding $t_2 - t_1$. The formula for converting the measured quantities shown in Fig. 5 to hydraulic conductivity, for a 4 in. long, $1\frac{5}{16}$-in. diameter soil cavity at the base of a 2-in. inside diameter cavity, is:

$$k = \frac{665 \log_e [(L_1 - L_r)/(L_2 - L_r)]}{t_2 - t_1}. \quad (4)$$

where k will be in inches per hour if t_2 and t_1 are in seconds, and L_1, L_2, and

L_r are each in the same units, inches; the factor 665 has been obtained with the aid of reference 21, Fig. 2.

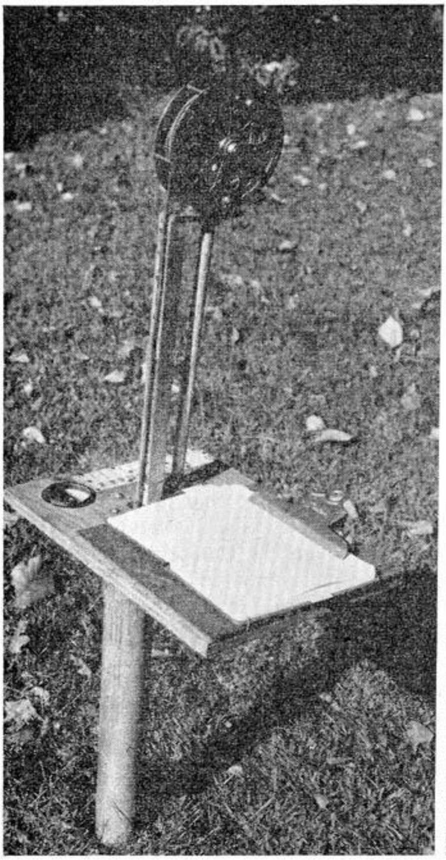

FIG. 6.—Clutched, Fishing Reel Type, Electric Probe for Determining Water Levels in Piezometer Method. After Luthin (22).

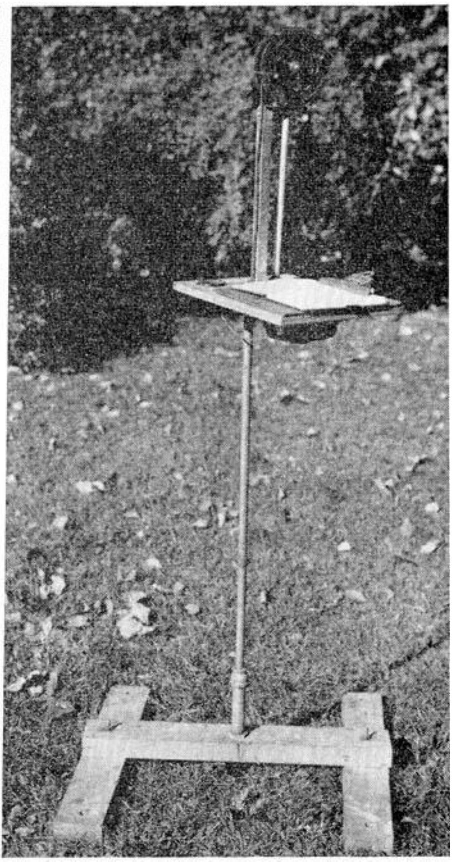

FIG. 7.—Electric Probe of Piezometer Method as Adopted for Reading Water Levels in Auger Holes.

Equation 4 is more complex than it need be for most field work. A simpler, very approximate expression, to be derived below, which is valid for almost all practical cases is, with units as in Eq 4:

$$k = \frac{1350(L_1 - L_2)}{(t_2 - t_1)(L_1 + L_2 - 2L_r)} \quad \text{(in. per hr)} \ldots (5)$$

or in more easily remembered form, since $(L_1 + L_2 - 2L_r)/2$ is the average head,

$$k = 665 \frac{\Delta h}{\Delta t} \frac{1}{\text{average head}} \ldots\ldots (6)$$

Even if the quantity $\frac{\Delta h}{\text{average head}}$ in Eq 6 is as much as $\frac{7}{10}$, use of Eq 6 results in an error of only 10.4 per cent.

After measurements have been made at the desired number of depths, the piezometer may be pulled out by a Veihmeyer soil tube jack or by a block and tackle on a tripod or by the hydraulic lift of a tractor.

It has been repeated that the piezometer method yields essentially the hydraulic conductivity at a point in the soil. This implies that a highly permeable layer or very impermeable layer near the piezometer cavity should have negligible effect on a determination, and this is, in fact, so. If an end of the cavity is closer than its length to such a layer, the error may be ±8 per cent for a cavity 1 in. in diameter and 4 in. long. For layers, at further distances, which are highly permeable or very impermeable, the error will be less than 8 per cent (21).

Figure 7 shows the electric probe for the piezometer method as adapted for use with auger holes. A pointed rod to be pushed into the soil for a "ground" is not shown in the figure.

Equation 5 remains to be derived. From the notation in Fig. 5:

$$\log_e \frac{L_1 - L_r}{L_2 - L_r} = \log_e \frac{h_1}{h_2}.$$

But the identity can be written:

$$\log_e \frac{h_1}{h_2} = \log_e \frac{1 + (h_1 - h_2)/(h_1 + h_2)}{1 - (h_1 - h_2)/(h_1 + h_2)}.$$

Also from calculus:

$$\log_e \frac{1+x}{1-x} = 2\left(x + \frac{x^3}{3} + \frac{x^5}{5} + \cdots\right)$$

so that taking $x = (h_1 - h_2)/(h_1 + h_2)$ and neglecting the terms in x^3, x^5, etc.:

$$\log_e \frac{h_1}{h_2} = 2\,\frac{h_1 - h_2}{h_1 + h_2}, \text{ approx.;}$$

or since $(h_1 + h_2)/2$ is the average head, and $h_1 - h_2$ is Δh:

$$\log_e \frac{h_1}{h_2} = \frac{\Delta h}{\text{average head}}, \text{ approx.;}$$

or in terms of L_1, L_2, and L_r:

$$\log_e \frac{L_1 - L_r}{L_2 - L_r} = 2\,\frac{L_1 - L_2}{(L_1 + L_2 - 2L_r)}, \text{ approx.,}$$

which when put in Eq 4 yields Eq 5.

Since the approximation $\log_e h_1/h_2 = \Delta h/(\text{average head})$ can be used, not only in the piezometer method but also in the tube method and in a number of other cavity problems, for example, those catalogued by Hoorslev (12, p. 31), it is desirable to compute a few values of the errors associated with the approximation. Let $\Delta h/(\text{average head}) = f$. Then, in the following pairs of numbers, the first number is f and the second number is the per cent error: 0.1, 0.09; 0.2, 0.4; 0.3, 1.3; 0.4, 2.0; 0.5, 3.8; 0.6, 6.4; 0.7, 10.4.

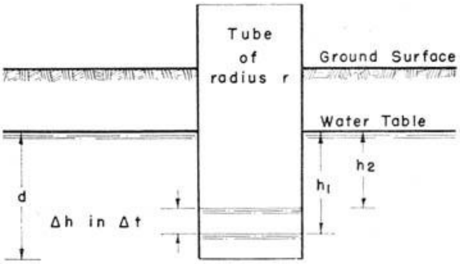

FIG. 8.—Tube Method for Measuring Hydraulic Conductivity of Soil (23).

TUBE METHOD

A special case of the piezometer method, where the piezometer is of large diameter and the length of the cavity at the piezometer base is zero, has, as indicated earlier, been called the tube method. This method, illustrated in Fig. 8, has been used primarily with tubes of 8-in. diameter, but theoretically any diameter tube could be used. As with the piezometer method, one measures (Fig. 8) the levels h_1 and h_2 and the time intervals $t_2 - t_1$ corresponding to $h_1 - h_2$. The formula for hydraulic conductivity is (23):

$$k = \frac{\pi r^2 \log_e h_1/h_2}{E(t_2 - t_1)} \times 60 \ldots\ldots (7)$$

where k is in inches per hour; r, h_1, h_2, and E in inches; and $t_2 - t_1$ in minutes. The time is measured in minutes becaus of the inherently slower time of water rise associated with the method due to the smaller ratio of entry area to tube cross-section.

The factor E in Eq 7 is a shape factor depending on the depth and diameter of the tube; numerical values for it may be taken conveniently from a table by Spangler (17).

As an example of use of Eq 7 for a tube 8 in. in diameter and 16 in. below the water table, $E = 20.8$ in. Equation 7 becomes:

$$k = \frac{16\pi \log_e h_1/h_2}{20.8(t_2 - t_1)} \times 60 \text{ (in. per hour)}$$

or putting $t_2 - t_1 = \Delta t$, and $h_1 - h_2 = \Delta h$ and using the approximation derived at the end of the last section:

$$k = 14.5 \frac{\Delta h}{\Delta t} \frac{1}{\text{average head}}, \text{ approx.}$$

A limitation on the tube method for 8-in. diameter tubes is that it can be used with presently developed equipment only to a depth of about 30 in. below the soil surface. The reason that the tube method tends to measure the vertical component of the hydraulic conductivity is that all the streamlines which enter the tube must do so in a vertically upward direction.

COMPARISON OF THE AUGER HOLE, PIEZOMETER, AND TUBE METHODS

If the soils for the three above described methods are homogeneous and isotropic, the methods should all yield the same values of k. If the soils are heterogeneous and anisotropic, the results will not be the same. Ignoring, however, the effect of anisotropy for the moment, it seems clear that the results for any one of the methods should be the more reproducible, in heterogeneous soil, the larger the soil sample.

Size of Samples:

To determine the size of samples associated with the three methods, one may arbitrarily take, as the size of the sample, that volume of soil about the cavities through which 80 per cent of the available hydraulic head is dissipated. If this is done, it may be computed for a 2-in. diameter piezometer cavity 4 in. long, for an 8-in. diameter tube, and for a 4-in. diameter auger hole, all penetrating to a common depth 30 in. below the water table, that the sample sizes will be respectively in the proportions 100 to 270 to 1400. Therefore, it may be estimated that the sampling errors due to soil heterogeneity for the three methods will be as $100^{-1/2}$ to $270^{-1/2}$ to $1400^{-1/2} = 1$ to 0.6 to 0.4. Field practice bears out this proportionality in a general way. But it should be remembered that in uniform isotropic soils the errors associated with the methods, other than those due to soil heterogeneity, should normally be about the same, the auger hole method being perhaps somewhat more subject to inherent errors than the others.

Reproducibility of Results:

Data have been obtained which indicate the reproducibility of determinations for each method by itself. For the auger hole method the following values of k in centimeters per day have been reported (19): 403 ± 13, 488 ± 12, 98.5 ± 3.9, 207 ± 11, 206 ± 5.2. The errors ±13, ±12, etc., when reduced to coefficients of variation are respectively, 3.2, 2.5, 4.0, 5.3, 2.5, the average being 3.5 per cent. This 3.5 per cent is a lower coefficient of variation than one might expect. The soil at the sampling sites here reported was unusually uniform. On less uniform sites, coefficients of variation of 10.7 and 14.4 have been reported (9).

Some rather complete experimental data (21) (see Table I) are presented for the piezometer method for three highly permeable Iowa prairie soils and one Iowa clay soil. The prairie soils are highly permeable, presumably because of

TABLE I.—HYDRAULIC CONDUCTIVITY MEASUREMENTS WITH PIEZOMETERS (21).

Site[a] and Soil	Inside Pipe Diameter, in.	Depth of Water Table, ft	Depth of Sample,[b] ft	Hydraulic Conductivity, in. per day							
				Pipe 1	Pipe 2	Pipe 3	Pipe 4	Pipe 5	Pipe 6	Average, in. per day	Coefficient of Variation
No. 1 Webster silty clay loam	0.756	1.0	2	725	514	586	648			618 ± 90	13
	0.756	1.0	3	1150	784	1222	1069			1056 ± 192	5.5
	0.756	1.0	4	1144	1569	1355	1081			1287 ± 221	5.8
No. 2 Webster silt loam	1.50	1.1	2	788	532	622	860	767	748	728 ± 116	6.3
	1.50	1.1	3	878	1140	888	955	878	813	925 ± 114	8.1
	1.50	1.1	4	840	810	814	862	1218	1150	949 ± 207	4.6
No. 3 Luton clay (not typical)	1.0	0.6	3	1.79	1.78	1.57	1.92	1.92	1.97	1.82 ± 0.147	8.1
	1.0	0.6	4	61.8	64.2	61.8				62.6 ± 1.3	2.1
No. 4 Marion silty clay loam	2.0	0.4	2	159	153	103	95			127 ± 33	26
	2.0	0.4	3	232	103	114	78			132 ± 68	52
	1.0	0.4	3	134	159	80				124 ± 40	32

[a] The locations, except site No. 3, are on highly permeable prairie soils.
[b] From surface of soil to top of cavity. Length of cavity is 4 in.

cracks, root channels, worm holes, and inherently good agricultural structure. The clay soil is of relatively poor structure. Notice that the inside diameters of the piezometers varied from soil to soil, so that comparison of diameter effects cannot be assessed from this table. The worst coefficient of variation in the table is 52; the least, 2.1.

Ayers (24), using piezometers with cavities of 1-in. diameter and of 4-in. length, reports a number of values of hydraulic conductivities, together with their standard deviations, for a glacial outwash area in Canada. From his data one computes an average coefficient of variation of 50. At one depth on one site he found a standard deviation in excess of the mean (the coefficient of variation being 115). He attributes this to "variation in permeability within the test area." In considering a coefficient or variation such as 115, one should remember, besides Ayers' comment, the fact that values of measurements of hydraulic conductivity by common (core) methods often deviate by more than 1000 per cent (5).

For the tube method, reproducibility of results has been found to be good, if measurements are made below the surface 1-ft layer of soil (23). Considerable variability is found at about the plow-layer depth. Some sample coefficients of variation for 6-, 12-, and 18-in. depth for an Iowa peat soil are respectively 47, 10, and 12. For this particular field of soil it was computed that 32.3, 1.1, and 1.5 tubes (that is, about 32, 1, and 2 tubes) would be needed at the 6-, 12-, and 18-in. depths, respectively, to obtain an average value of the hydraulic conductivity of the soil which would be within ±10 per cent of the true value two thirds the times that these sets of tubes would be used in an extensive program of determinations.

The data given above on reproducibility of results refer to measurements obtained on small test sites, of areas generally much smaller than an acre. On large test sites variability may be large. On a 30-acre field, considered to be uniform, measurements (25) of the hydraulic conductivity varied from about 2 to 10 in. per day at 18-in. depth and from about 2 to 6 in. per day at 30-in. depth. Sand or clay lenses in a soil can of course cause extremely large variations in measured values.

Man Hours for the Three Methods:

Some figures for the man hours for the piezometer and tube method, but not for the auger hole method, are available. The auger hole method is faster than the others.

For the piezometer method (21), one man working in loam soil with 1-in. piezometers can install four piezometers at 2, 3, and 4 ft; obtain readings at each of the depths; and remove the piezometers, all in 6 hr. On heavy clay soil the time is doubled. If two men, one unskilled, work together the number of readings in a given time can be doubled. Experience shows that the times for 1-in. piezometers apply equally well to 2-in. ones.

For the tube method, under suitable conditions, two men working 8 hr can make determinations at 6, 12, 18, 24, and 30-in. depths with 12 tubes, each tube at each depth. The tube method thus appears to require considerably less labor than the piezometer method. But note that these figures for the tube method are for depths to only 30 in.

Soil Anisotropy:

It has already been stated that the auger hole and piezometer methods tend to accentuate the horizontal hydraulic conductivity of the soil. The possibility of sorting out the horizontal and vertical hydraulic conductivities by use of the methods was pointed out by Reeve and Kirkham. Childs has considered the problem in some analytical detail (26). M. Maasland and the author have done further work on the problem and have obtained experimentally some, as yet unpublished, values of the shape factors E needed for anisotropic cases. Some approximate values of E for anisotropic cases can be obtained from the paper by Hvorslev.[5]

Use of any of the methods described above for determining anistropy should ordinarily yield higher values of horizontal hydraulic conductivity than the vertical (27). As an exception to the rule, H. P. Johnson[6] found vertical values higher than horizontal, for a certain loessal soil. The general subject of soil anisotropy, as it is related to seepage problems, has been considered by Dutch investigators. M. Maasland[6] has gathered this work together, as well as that of Austrian and German investigators, into the English language.

Effect of Roof Channels, etc.:

N. Kadir[7] considers the effects of root channels and worms holes on the methods, also the effect of the use of impervious disks at the base of piezometer cavities. He concludes that a cone of draw-down, rather than worm or root channels, accounts for anomalies sometimes observed in the methods and that impervious disks placed at the bases of cavities have relatively small effects.

FURTHER FIELD METHODS

Excluding the three methods described above (which were not yet developed), Wenzel (28), in 1942, described a number of field methods for determining "permeability." Two of the most common methods are based on discharge and draw-down measurements for a single well. These methods are well known and will not be mentioned here.

Child's Two-Well Method:

Quite recently Childs(26) has proposed, and he and co-workers (29) have utilized, a method employing two wells rather than one. The two wells are of equal diameter and penetrate to the same depth below the water table, preferably to an impermeable layer, if one exists. Water is pumped at a steady rate out of one well and carried by a hose into the other. A small but distinct hydraulic head difference is thereby created between the levels of water in the wells. If Q is the pumping rate, Δh the hydraulic head difference, L the length of each well, r the radius of each well, and d the distance between their axes, then, the hydraulic conductivity k is given by:

$$k = \frac{Q}{\pi L \Delta h} \cosh^{-1} \frac{d}{2r} \ldots\ldots\ldots (8)$$

The units of k will be in inches per hour if Q is in cubic inches per hour and L, d, r, and Δh are all in inches. Equation 8 is correct only if the wells penetrate to an impermeable layer. If an impermeable layer is not reached, a correction is to be applied.

Some field results obtained with Eq 8 have been given. It is not clear from the data whether more than one pair of wells was established at any one site. Therefore reproducibility of results from pairs of wells cannot be ascertained. There is good agreement for results ob-

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[5] For Hvorslev's case No. 4, p. 31 of reference (**12**), it is believed the factor 2.75 D is probably too large; a better value may be 2.45 D. [See Fig. 3 of reference (**23**), noting that the last two plotted points, both of value A or E = 19.40 in., pertain to an 8-in. diameter tube; thus, $E = (19.40/8)\ D = 2.45\ D$. See also Fig. 3 of reference (**21**) where the same result, 2.45 D, was obtained independently.]

[6] M.S. thesis on file in the library of Iowa State College, Ames, Iowa.

[7] Ph.D. thesis on file in the library of Utah State Agricultural College, Logan, Utah.

tained by interchanging the pumped-out and pumped-into wells.

Labor involved in making the two-well measurements is not reported. Quite a little labor may be involved, as each well is cased with a screen and gravel liner.

The size of soil sample for the two-well measurement will be quite large. This volume was computed for a pair of wells, each of 6-in. diameter, and extending to an impermeable layer 30 in. below the water table. The calculation shows that the size of sample, in which 80 per cent of the head will be lost, will be approximately equal to the corresponding volume of soil for a single auger hole 4 in. in diameter and 30 in. deep. The calculation thus raises the question as to whether a single well, to which the auger hole method would be applied, might not yield results as good as, or better than, the two wells. But note that if an horizontal anisotropy is in question the two wells might better be used in either event.

In the two-well method, the well into which the water is pumped will always be subject to a certain amount of clogging by suspended material carried into the soil pores. Frevert and Kirkham found that when a head was applied to a cavity to force water into the soil rather than *vice versa*, the measured hydraulic conductivity was reduced by a factor of five. But Childs *et al.* worked primarily in sandy soils. Also the screen and gravel liners, which they used, must have reduced the clogging effect. In the auger hole, piezometer, and tube methods, the seepage is from the soil into the hole; accordingly clogging of the pores by suspended material is not a problem.

A Proposed Four-Well Method:

It has appeared that clogging of pores can be a problem in a two-well method, especially if sand liners are not used. By placing two more cased wells of the piezometer type between the two outer wells, a measurement of hydraulic conductivity should be obtainable which, in theory, would be independent of any clogging effect at any of the four "wells." This four-well method would be entirely analogous to the four electrode method for measuring the electrical conductivity of soil (30, 31, 32, 33). The rate of water movement between the outer two wells and the difference in head between the inner two wells would be measured. The ratio of the rate of water movement to the difference in head between the inner two wells would then be, to within a determinable geometric factor, the hydraulic conductivity.

A Proposed Single-Well Method:

The four-well method would probably have practical application only in confined artesian aquifers of well-known geometry. The following proposed single-well procedure might be of more general use. Bore a cavity, as with a post hole auger, into the soil a foot or two below the water table. After puddling effects have been removed and the true ground water level has been established, maintain a small, pumped down distance Δh in the well and record the pumping rate Q. Let E be the appropriate shape factor for the well. Then,

$$Q = kE\Delta h \quad \text{(9)}$$

in which all quantities but k, and hence k itself, will be known.

The method will be clearer if it is assumed that the well is of hemispherical shape, its flat surface to coincide with the water table as it exists before pumping. When the pumping is underway the streamlines will be essentially horizontal, because it was agreed to make Δh small compared to the depth or radius r of the well. Therefore, the flow rate into the well will be (34, pp. 258–263; note especially near bottom p. 262):

$$Q = k(2\pi r)\Delta h \qquad (10)$$

Comparing Eqs 9 and 10, it is seen that the shape factor for the hemispherical well is $E = 2\pi r$. For a post hole, E will involve, besides the hole's radius r, also its depth d.

Notice that the streamlines for the proposed single-well method will all (except for a few at the top of the well, which exist when pumping is begun) originate theoretically at infinity. This is an important observation in the method because, at infinity, the hydraulic head will always be Δh higher than it will be in the well because the supply of water at infinity is infinite and hence cannot be drawn down by a finite amount of pumping. Hence it is seen that Q will remain constant, as given by Eq 10, to within less than $[2\pi r\Delta h/(2\pi r^2 - 2\pi r\Delta h)] \times 100$ per cent.

Notice further that the proposed single-well method, as described, will not work if the well penetrates to an impermeable layer. In that case, the flow rate would be (34, p. 153):

$$Q = \frac{2\pi kh\Delta h}{\log_e r_e/r} \qquad (11)$$

where h is the depth to the impermeable layer, r_e is the radial distance to an external boundary where the hydraulic head is Δh higher than at the well; the other symbols are as before. Since r_e in Eq 11, to be comparable with the case of Eq 10, would be infinite, it follows that Q in Eq 11 would be zero. Actually the flow to the well would not be zero. The flow in excess of zero would be that originating in the layer of thickness Δh, flow not accounted for in the proposed method. (To handle the case where the well penetrates to an impermeable layer, a known procedure, but one which is not as simple as that for the proposed single-well method, can be used (see reference 17, p. 134 or reference 28, p. 78). The procedure utilizes, besides the original pumped well, two additional observational wells, rather than a single well as for Eq 10.)

If one follows through the mathematical derivation of Eq 10, he will observe that a distance corresponding to r_e in Eq 11 drops out of the analysis, when r_e goes to infinity. This observation is, in short, the basis of the proposed single-well method.

Dry Auger Hole Method:

In order to determine the amount of seepage to be expected when a canal is constructed in an arid area, the Bureau of Reclamation has developed a dry auger hole method for determining the hydraulic conductivity of soil in place. They designate the procedure as a "well permeameter test" (35). The method consists of (*a*) boring an auger hole in dry soil, (*b*) casing the upper portion with a screen, (*c*) filling the lower portion with sand, and (*d*) observing the rate at which the hole will accept water. The diameter of the hole may be 4 to 8 in. and the depth 5 ft or more. Detailed procedures and nomographs have been supplied for the field worker.

Methods Depending on Discharge Measurement of Drain Tubes or Ditches:

Seepage formulas have been developed for flow into tile drains (36 to 40) and ditches (36, 41), etc., in terms of the hydraulic conductivity k. If all quantities in these formulas are known except k, then k can be computed. Such a procedure would probably not have very much use. Installations of the type in question are generally made, or should be, after a knowledge of the hydraulic conductivity of the soil has been obtained by other methods.

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MEASUREMENT OF PERMEABILITIES IN GROUND-WATER INVESTIGATIONS

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Synopsis

Certain physical conditions must be considered in the measurement of permeabilities of soils and sediments. Among these are sediment structure (particularly whether the sample is disturbed or undisturbed), the occlusion of air, and, finally, the effects of colloidal material. To meet some of these conditions, a special sampling procedure has been developed for obtaining undisturbed samples suitable for permeability measurement. The particular sampling device is of the piston type and contains an inner barrel, in which an undisturbed soil sample is taken. This inner barrel, with its undisturbed sample, is removable and serves as the permeameter tube in subsequent permeability measurements.

The techniques for permeability measurement are described.

It is emphasized that the test liquid must be one that is in equilibrium with the soil sample, so far as colloidal chemical properties are concerned. For example, in the determination of ground-water permeabilities, actual ground water, which is assumed to be in equilibrium with the sediments of the aquifer, must be used. If fluids such as distilled water or ordinary laboratory tap water are used, then severe washing of colloidal material may occur, owing to a change in the exchangeable ions, with a consequent erroneous measurement of the permeability.

Finally, the alterations in permeability due to trapped air and other gaseous material are discussed. They result in a decreased value of permeability.

Simple Transmission of Water Through Sediments

The principal items that characterize the response of a sediment to the transmission of a given fluid through it are the nature of the sediment, whether gravel, sand, or clay, the material initially occluded in its pores, such as air, and finally the kind of permeating fluid.

The basic ideas involved in understanding permeability phenomena in sediments are simple. They have been given by Hazen, Darcy, Kozeny, Slichter, and others. The vast majority of sediments contain channels that consist of networks of multiple conical capillaries. In nature, the flow through such tubes is generally laminar. In such flow there is no turbulence, and the energy associated with the driving head is used almost entirely to overcome viscous drag between

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flowing elements of liquid. For example, in the case of a simple cylindrical tube, the flow is in a series of concentric annuli, each one of which has a different velocity; the drag is between adjacent annuli, and all the energy of the driving head is used to overcome it. The flow is steady once the motion is established.

According to Darcy's law, the velocity for purely viscous flow, v_x, through an element, for a pressure differential, dp, between faces of unit area and a distance, dx, apart, is given by:

$$v_x = -K\,(dp/dx) \qquad (1)$$

The quantity K is called the coefficient of permeability and is determined by the geometry of the pore system and the nature of the fluid. The minus sign indicates that the flow is opposite to the direction of pressure increase. The pressure gradient is dp/dx. If uniform, the flow, Q, through a section of area A is Av_x, or $Q = Av_x$, and thus:

$$Q = -KA\,(dp/dx) \qquad (2)$$

Equation 2 may be written $K = -(Q/A)/(dp/dx)$ so that the permeability coefficient K is seen to be simply the quantity of fluid driven through unit area by a gradient of unity. This equation is not applicable to extremely coarse sediments, such as gravel, in which water is moving at moderately high velocity. The larger openings give rise to turbulence under moderate gradients, and the analogy is with pipe rather than capillary flow.

Corrections for Fine Capillaries:

Equations 1 and 2 are subject to corrections when extremely fine capillaries are used. One cause for correction arises from the so-called "slip flow" (**1, 2, 3**),[2] a condition that results when molecular diffusion contributes appreciably to the total flow. A second correction, necessary in very fine capillaries, is for the decrease in channel size caused by adsorption of a film (**4, 5**) of the wetting liquid. The film thickness constitutes an appreciable fraction of the size of the flow channel. There are also abnormal viscosity effects.

EFFECTS OF ION EXCHANGE IN CLAYS

The influence of the permeant fluid must be considered, especially as it affects the condition of the sample. The composition of the water has little or no effect on the structure of sands or silts, provided there is no clay present. Ordinary gases likewise have little or no effect on structure.

However, when clays are a part of the sediment, many complications arise in the permeability measurement. Ordinary gases, for example, may be adsorbed by the clay complex. If water is used as the permeant fluid, the physical properties of the clay are altered, according to the chemical composition of the water. If water is distilled to a high degree of purity and then passed through a calcium-saturated clay, the calcium will be leached out, and a hydrogen-saturated colloid formed by an exchange of ions. This colloid is generally freely mobile and begins to migrate. The hydrogen clay will generally pack mechanically into a layer somewhere else in the sediment, and the permeability will be characteristic of the sheet of closely packed hydrogen clay, rather than of the original sediment. This example illustrates the importance of ion-exchange phenomena. These effects depend on the type of clay also. Montmorillonite, for example, has several times the ion-exchange capacity of kaolinite, and therefore is capable of greater changes in physical properties because of ion exchanges.

Related effects were observed in 1940 in the laboratory of the U. S. Department of Agriculture, Division of Soil Chemis-

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[2] **The boldface numbers in parentheses refer to the list of references appended to this paper, see p. 114.**

try and Physics, by the senior author of this paper, in collaboration with C. Nikiforoff and M. Drosdorf, although the results of their tests were never published. It was planned at the time to do more extensive work, but war conditions prevented. At that time three columns of homogeneous medium quartz sand, about 12 in. long, were set up. Three permeants were prepared and passed upward through these columns: (1) a water suspension of a natural colloid from the B horizon of the Dayton silt loam, a clay pan; (2) a water suspension of calcium-saturated colloid from the same Dayton soil; and (3) a sodium-saturated colloidal suspension, also from the same Dayton soil. All colloids initially moved freely through the respective sands, but finally sealed them. The natural and calcium-saturated colloids sealed off their respective sands much more quickly, but much less completely, than the sodium colloid which actually took several times as long as the other suspensions to accomplish this. The tightness of the seal and the time required to achieve it is explained by the difference in the size of the ultimate particles of each: the particles were much smaller in the case of the sodium-saturated colloid.

When the columns were later examined, it was found that the sodium-saturated clay packed densely in a sheet about $\frac{1}{8}$ in. thick, to form the seal near the outflow (upper) end of the columns. Below this, small amounts of colloid were found for a distance of about 8 in.; these amounts were not sufficient to close the pores appreciably. Similar conditions prevailed in the case of both the calcium-saturated and natural colloid.

If water that is in equilibrium with the sediment is used as the permeant fluid, changes in the mechanical arrangement of the solids are not to be expected. For example, if a water containing sufficient calcium to be in equilibrium with a sediment containing calcium clay is passed through that clay, disturbance should not occur. If, however, the sediment is leached with water containing sodium ions, an ion exchange ought to occur, with the liberation of a freely mobile and more highly dispersed sodium clay, which doubtless will migrate and elsewhere seal off the sediment.

The above considerations seem to indicate that in permeability measurements it is necessary to use water that is similar in composition to that which occurs in, or will ultimately be passed through, the sediment. For example, permeability of the sediments lining an irrigation ditch should be measured with water typical of that used for the irrigation. Permeabilities of sediments below the water table should be measured with ground water taken from those sediments, or at least water that is closely similar in chemical composition, and hence is in or close to equilibrium with the sediment. These examples serve to illustrate the fact that the permeabilities, to be useful, must be measured with the permeant that is to be used in the actual application. They further illustrate the importance of the theory of ion exchange, in clays, where water is involved. Further information is available in an extensive amount of literature on the subject (6, 7).

INFLUENCE OF PHYSICAL CONDITION OF SAMPLE

Another item of importance in permeability measurements is the condition of the sample—that is, its structure. In order to understand clearly the role played by structure, consideration of the common soils is helpful.

Several different types of structure generally are recognized by pedologists. In the authors' opinion, the most useful classification is that suggested by Nikiforoff (8). In the soil there are primary and secondary structures, each of equal importance. The arrangement of the single soil particles is called the primary structure; in turn, the single particles are generally combined into aggregates, which are the units of secondary structure. The aggregates fall naturally into four simple patterns: granular, platy, prismatic, and blocky. In each of these, the aggregates have the approximate geometrical shape indicated by the name: grains, plates, prisms, and cubes. There are characteristic pore spaces between the secondary units. These relations are illustrated schematically in Fig. 1. Figure 2 is a photograph of a prismatic arrangement.

It is well to bear in mind again that each single aggregate, such as a prism, has a characteristic porosity, the primary porosity formed by the spaces included between the primary particles. When the soil has no secondary structure, but consists merely of one continuous mass of primary particles, it is said to be massive. If the soil possessing a natural secondary structure is crushed, there results an assemblage of fragments. Each fragment retains the porosity of the original aggregate of which it was a part, while a new system of secondary channels or openings between the fragments is formed. Consequently there is a new secondary porosity. In this connection it is to be observed that fragments slake easily in

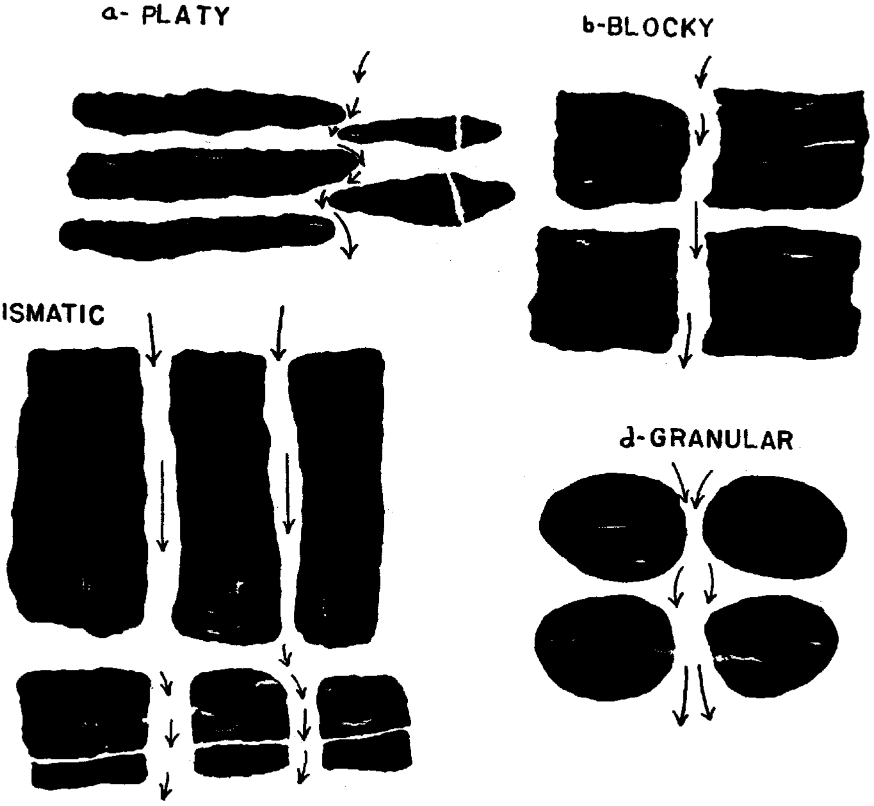


FIG. 1.—Schematic Diagram of Some Common Soil Structures.

water, whereas particles in their natural structural condition do not, because of protective coatings.

If water-transmission properties are required in soil that is to be used in a crushed state, then the permeability must be determined on the sample when it has reached a porosity that is characteristic of its new use. Slaking of fragments when water is first introduced results in a changing apparent permeability which eventually stabilizes, provided the soil is one that cannot be otherwise altered by the water.

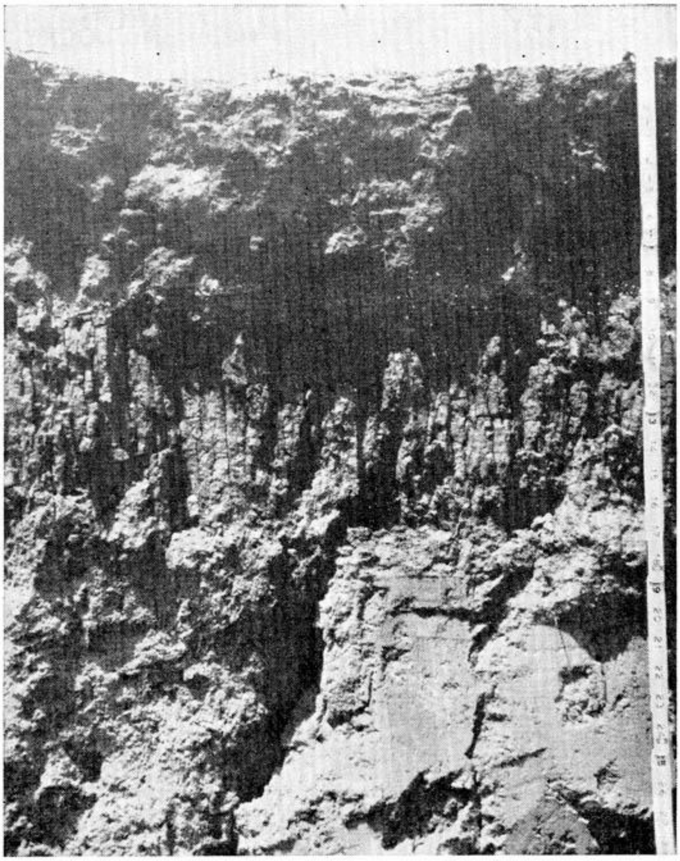
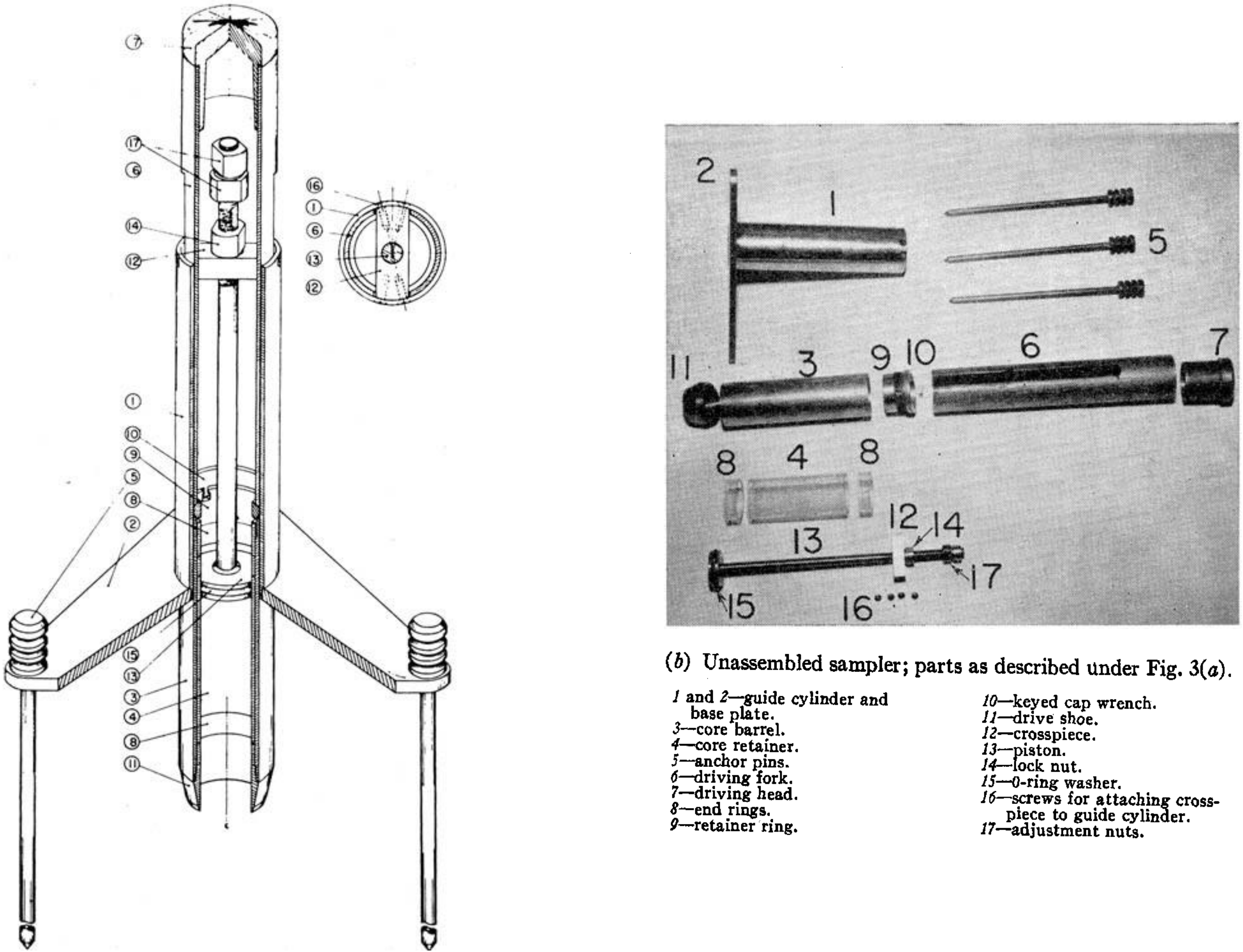


FIG. 2.—Semidesert Brown Soil, Found Near Akin, Colo., Showing Prismatic Structure.

If, on the other hand, an infiltration estimate is to be made on a natural soil, then the permeability of a natural and undisturbed sample must be made, because the greater part of the infiltration, especially in the initial stages, occurs in the secondary pore space (9). When swelling occurs, the secondary pore space gradually closes, and the ultimate ability to transmit water is determined by the primary pore—that is, the space between the primary particles that compose the aggregates.

It is thus seen that considerable care must be exercised in taking samples. To meet this condition, a drive sampler has



(*a*) Cutaway drawing.

(*b*) Unassembled sampler; parts as described under Fig. 3(*a*).

1 and *2*—guide cylinder and base plate.
3—core barrel.
4—core retainer.
5—anchor pins.
6—driving fork.
7—driving head.
8—end rings.
9—retainer ring.
10—keyed cap wrench.
11—drive shoe.
12—crosspiece.
13—piston.
14—lock nut.
15—0-ring washer.
16—screws for attaching crosspiece to guide cylinder.
17—adjustment nuts.

FIG. 3.—Piston Drive Sampler.

been designed and applied in a technique for making permeability measurements.

APPARATUS FOR TAKING UNDISTURBED SEDIMENT SAMPLES

Many of the sampling devices in current use have been reviewed in detail by Hvorslev (10).

A sampler suitable for ground-water investigations in which it is generally desired to determine the natural rate of transmission of water in the ground must take samples that are adequate for use in determining the hydraulic properties of sediments. Specifically, the sample must retain its natural structure during (*a*) the sampling operation, (*b*) transport to the laboratory, and (*c*) placement of the sample in a permeability apparatus.

The senior author of this paper has developed a sampler that meets these requirements (Fig. 3(*a*)). With this device, undue disturbance of the sample during the driving operation is prevented by means of a piston that can be adjusted to fit tightly against the sediment face. The sample is caught in a rigid core retainer of clear, smooth plastic. The sample is not disturbed as it passes into the tube. The clear plastic case containing the core can be made to serve as the barrel of a specially designed permeameter, without removal of the sample. Hence, the disturbance of the natural structure of the sediments is kept to a minimum from the beginning of the sampling operation to the end of the permeability test. The present equipment is designed for surface sampling in excavations. Additional equipment is contemplated to apply the present sampler design to sampling inside cased or uncased borings at considerable depths.

DESCRIPTION OF SAMPLER

The unassembled sampler is shown in Fig. 3(*b*). The triangular base, *2*, or anchorage plate with the attached vertical guide cylinder, *1*, is the basic support of the sampler during the sampling operation. It is held in position by three steel pins, *5*, that are driven into the soil through holes in the base plate. Part *13*, an adjustable piston fitted with a rubber piston ring is fastened to the vertical guide cylinder by means of a crosspiece, *12*, attached with four screws, *16*. The piston mount is different from that of the orthodox piston-drive samplers. In most devices the piston is permitted to ride freely on top of the sample as the sampler barrel is forced into the ground. This allows the sample to expand and thus to increase in length as it enters the barrel. In this sampler, axial expansion of the sample is prevented, and the sample preserves its original configuration. (Compaction of the sample is minimized by the technique used.)

The barrel, *3*, which is driven into the ground, has a steel drive shoe, *11*, attached to the bottom. The drive shoe has a 2-in. inside diameter, and the barrel a 2¼-in. inside diameter. Three pieces of clear plastic tubing, *4* and *8*, with a wall thickness of ⅛ in., are inserted into the barrel from the bottom before the drive shoe is attached. The center tube (5¼ in. long) is the core retainer, *4*, and ultimately becomes the barrel of the permeameter. The rings, *8*, ⅝ in. long, shown at each end of the center tube, are parts of the core retainer, and their use forms an important part of the sampling technique, as explained fully in the section on Obtaining the Sample.

Use of the clear plastic core retainer permits inspection of the sample in the field. If the sample is not a good one, as may be determined by inspection, another can be taken immediately, thereby eliminating the cost of visiting and preparing the sampling site more than once.

The plastic parts are held at the top by a retainer ring, *9*, screwed into the top of the barrel. A cap wrench, *10*, having keys to fit the slots of part *9*, fits freely on top of the retainer ring. The cap wrench transmits the driving force from the bottom of the driving fork, *6*, to the top of the sampler barrel. The driving fork is slotted to clear the crosspiece supporting the adjustable piston. The detachable driving head, *7*, made of brass, fits into the top of the driving fork, which is made of stainless steel in order to reduce the total weight. (This is an important factor in horizontal sampling.) A 24-oz rubber mallet is used for driving.

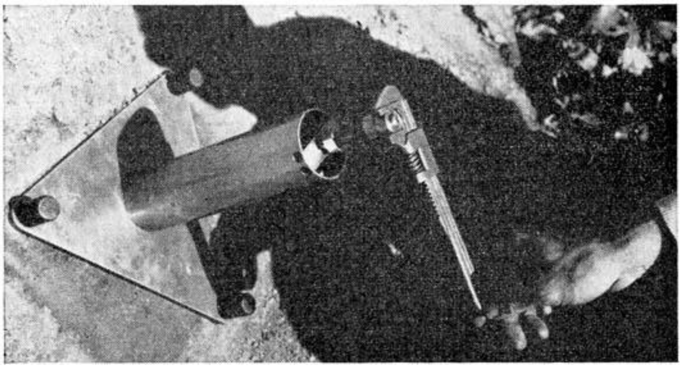


FIG. 4.—Adjustment of Piston to Make Good Contact with Sediment Face.

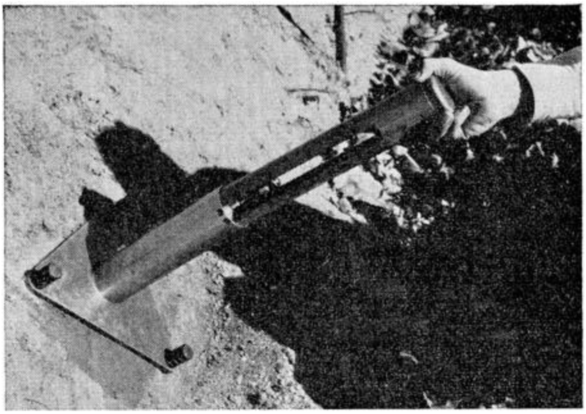


FIG. 5.—Placing of Driving Fork in Position.

All parts of the sampler except the driving head, base plate, and core retainer and end rings are made of stainless steel. The base plate and driving head are of brass; the core retainer and end rings are polished, clear plastic tubing, 2 in. in inside diameter, cut to length. The commercial plastic tubing used has an outside diameter slightly less than 2¼ in., just sufficient to allow a free fit in the core barrel.

OBTAINING THE SAMPLE

In obtaining undisturbed samples, proper technique is as important as the use of proper apparatus. The procedure now followed by the authors for taking undisturbed samples is outlined below.

Sites are selected in the area under investigation, from which to obtain representative samples of the materials. These sites should be staked or otherwise clearly described for the sampling crew. As excavation is required to obtain samples of materials below the surface, all construction excavations or borrow pits being dug at the time of sampling, or made not long before, should be examined to determine whether suitable samples can be taken from the walls. The cost of sampling can be reduced materially by thus taking advantage of existing excavations. After a network of the best sites is described, the sampling crew collects the samples.

Samples of all unconsolidated materials above the water table can be taken if the particle size of the material is small in comparison to the diameter of the barrel. Some have been taken a short distance below the water table, but if the saturated material is clean and granular, much care is required to keep the core undisturbed during its removal from the site and its preparation for storage. The size of the excavation needed for sampling varies with the depth from which samples are required and the structural properties of the earth materials forming the walls of the pit. In general, the pit should have a minimum cross-section of about 4 by 6 ft, and have straight, vertical walls. The rectangular pit is dug with a hand spade. Care must be taken throughout the spading to reduce to a minimum the disturbance of the materials at the wall face. A complete description of the materials encountered in the digging is recorded, to be a part of the notes describing the samples.

FIG. 6.—Further Removal of Sediments Around Sampler Barrel.

All surfaces to which the sampler is fastened are dressed flat with a spade or spatula. One end of the pit is used for taking vertical samples. This requires a ledge about 18 in. wide extending across the pit, with its top near the land surface. After a satisfactory sample is obtained at the initial elevation of the ledge, the ledge is cut down to the elevation at which the next sample is desired. Horizontal samples are taken from the wall of the pit. In taking horizontal samples, the top edge of the base plate of the sampler (*2* in Fig. 3) should be horizontal to obtain maximum support from the steel pins.

The sampler is "loaded" with a core retainer and end rings as follows: (*a*) the drive shoe, *11*, is removed; (*b*) the inside of the sampler barrel, *3*, is thoroughly cleaned with a brush; (*c*) a plastic end ring, *8*, a core retainer, *4*, and another end ring are inserted in that order; (*d*) the drive shoe, *11*, is fastened into position; and (*e*) the sampler barrel, *3*, is inserted in the guide cylinder, *1*.

The barrel of the sampler is retracted so that the shoe is above the base plate, and the sampler is held against the prepared sediment face as the steel pins are driven through the holes in the base plate into the sediment. The piston is adjusted by freeing the lower lock nut that rests against the crosspiece and applying a wrench to the upper of the two top lock nuts on the piston shaft (see Fig. 4). The shaft is turned slowly until the piston rests firmly on the sediment face; then the bottom lock nut, *14*, is tightened and the unit is ready for the driving operation.

FIG. 7.—View of Barrel Through "Relief" Hole.

The sampler must be rigid against the sediment face to obtain a perfect sample; if mounted loosely, it may wobble during the driving operation, causing the sample to be fractured.

The driving head is fitted into the top of the driving fork, which is inserted into the sampler, straddling the crosspiece (Fig. 5). Successive blows of the rubber hammer on the driving head force the barrel into the sediment. Generally, the

FIG. 8.—View of Barrel Just Before Removal from Sediments.

force of the hammer blow should be sufficient to move the barrel about $\frac{1}{32}$ in. per blow. It is desirable to strike the driving head with the rubber mallet as near center as possible. Glancing blows cause the sampler axis to deviate from the principal direction of driving, which fractures the core.

FIG. 9.—Removal of Core from Sampler Barrel. In this particular sampling, shoe remains attached to core during removal from barrel.

After the barrel has been driven about 2 in., a long screwdriver or similar "relieving" tool is used to remove material from around the barrel as far into the sediments as the junction of the barrel with the shoe (Figs. 6 and 7). This "relieving" is done to reduce the amount of compaction caused by entry of the sampler barrel. Care must be taken to leave sufficient bulk around the steel pins for support of the sampler during the remainder of the sampling operation. The driving and relieving process is continued until the plastic core retainer and end rings are filled. The steel pins are removed, and the outer part of the sampler is removed carefully from the sediment face. The sampler barrel will remain lodged in the sediments (Fig. 8). It should not be pulled from its location, because the forces involved may fracture the sample and possibly destroy it. The sampler barrel should be cut free by excavating around it, and the end of the

FIG. 10.—Severing of Top End Ring from Core by Insertion of Spatula Between Parts and Cutting Across End of Core.

core at the edge of the drive shoe should be severed with a spatula.

The drive shoe is carefully removed from the barrel, but remains intact on the core, keeping to a minimum the disturbance of the part of the core in the lower end ring. The end rings and core retainer are then easily removed as a unit (Fig. 9). A broad spatula is inserted between both end rings and the core retainer, and the core is severed by slowly forcing the spatula through it (Fig. 10). A small amount of the material taken from the end rings is heaped on the ends of the core and rounded off before the vinylite caps are placed over them. The addition of this loose material insures a tight fit of the core in the core retainer during transport; the loose material is carefully scraped away in the laboratory. A band of waterproof tape seals the cracks between the lucite core and vinylite caps and holds the caps in place. If long storage is contemplated, samples should be sealed in wax in addition to being taped. This procedure is repeated for the drive shoe end of the core (Fig. 11). The sample, ready for transport to the laboratory, is shown in Fig. 12.

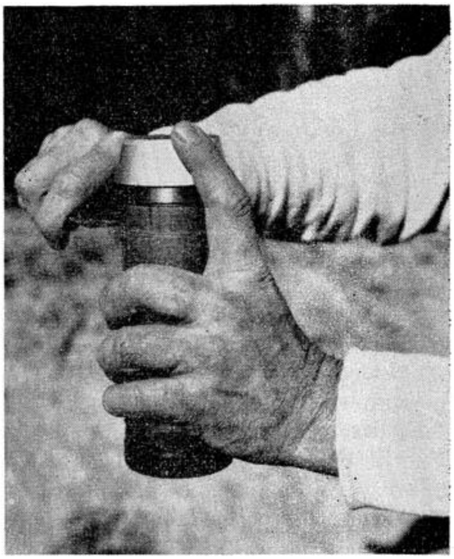


FIG. 11.—Inversion and Preparation of Other End of Core, Just Before Sealing with Vinylite Cap.

The lucite sample tube is readily made

to serve as the barrel of the permeameter. The top and the bottom parts required to accomplish this are shown in Fig. 13. The jig shown in Fig. 14 is used in drilling the barrel for insertion of head gages.

As was previously stated, the sampler described was designed primarily for sampling above the water table. It is to be expected that air or other gases partially fill the interstices of the sample. Even if the sample is taken below the water table, it is difficult to maintain 100 per cent saturation up to the time the sample is placed in the permeameter. The gases locked in the sample pores in effect decrease porosity and hence reduce the cross-section available for fluid transmission. As the permeability test is begun, liquids initially circulating through the sample by-pass the larger pores, sealing off quantities of gas. Capillary forces, as shown schematically in Fig. 15, hold the gas in place. If the permeant is saturated with gas, there is the possibility that the gas bubbles will grow larger and further reduce the cross-section of flow. If the permeant is unsaturated, the gas will be reduced or removed by solution. This factor has been observed to produce erratic results in the measurement of permeability. In order to be consistent for comparison purposes, it is preferable to measure permeabilities at a common fluid-saturation level. This is best achieved in practice by maintaining 100 per cent fluid saturation throughout the test.

FIG. 12.—Completely Sealed Core Ready for Transport.

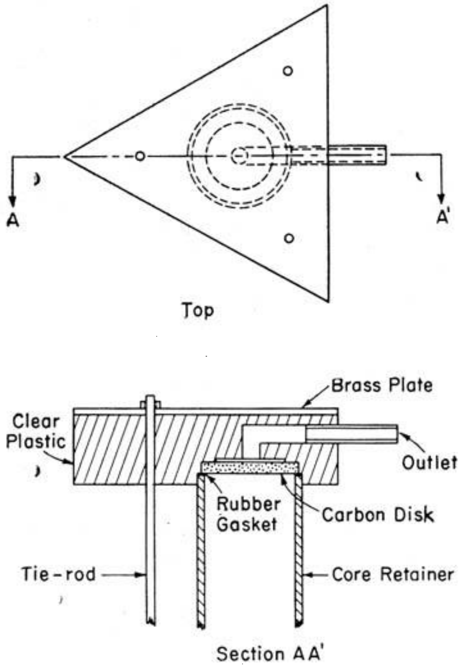


FIG. 13.—Drawing of Permeameter Parts Required for Permeability Measurements on Sediment Core Shown in Fig. 12. An empty core retainer is shown.

AIR REMOVAL

Air locked in a sediment sample can generally be removed by one of several procedures. An application of vacuum followed by an application of atmospheric pressure will remove trapped air if repeated sufficiently (11). The procedure has the disadvantage that it tends to disturb the sample, particularly by altering the porosity to that of a tighter packing. It is much more useful as a means to degas sediments when grain density is being determined.

A better procedure, which is less disturbing to the sample, is to circulate gas-free water through the sample until all trapped air is removed by solution. This may require several weeks in extreme cases. But approximate checks on trapped gases have shown that practically no gas remains in most sand samples after the circulation of deaired water for a period of 8 hr. Studies to determine residual gas by more refined means are contemplated for the future. The gas-free water must be in equilibrium, chemically, with the clay fraction of the sample.

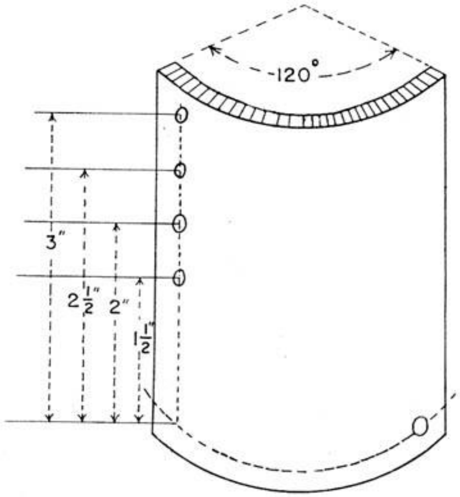


FIG. 14.—Brass Jig Used to Tap Lucite Core Barrel for Head Gages Required for Permeability Measurements.

The apparatus that the authors have used for some time to degas water is shown in Fig. 16. In it water is degassed by the application of a vacuum of approximately 28 in. of mercury produced by a high-capacity pump. The vacuum chamber used in the hydrologic laboratory of the Ground Water Branch, U. S. Geological Survey, is a large rectangular copper tank 1 ft by 4 ft, and 9 in. deep. A float controls the intake of water through valve *B* (see Fig. 16). An electrical timer, *C*, controls the vacuum application valve, *D*. The same timer controls the valve, *E*, that permits atmospheric pressure to prevail within the tank. The timer, *F*, controls the outflow of water into the sand filter and storage reservoir. The operation (referring to Fig. 16) is as follows:

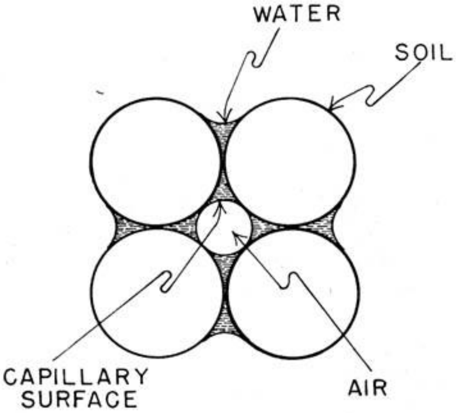


FIG. 15.—Locking of Air in Sediment Pore by Capillary Forces.

As the water level in the tank is lowered, a mercury switch, *A*, operated by a float in the tank closes the circuit through the solenoid valve, *B*, allowing water to pass into the tank. Water flow into the tank continues until switch *A* is opened and solenoid valve *B* shuts off the water.

The process timer, *C*, operates with a 7-min cycle. When it closes the circuit, *D* (a solenoid valve) opens, allowing vacuum to be applied to the water in the tank, while *E*, the air-intake valve, iso-

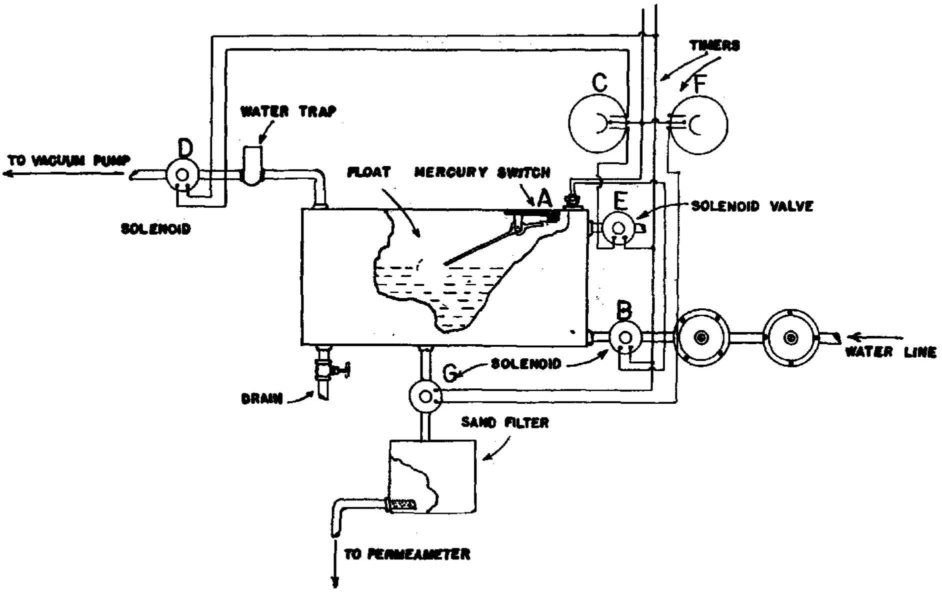


FIG. 16.—Schematic Diagram of Device for Obtaining Air-Free Water for Use in Permeability Measurements.

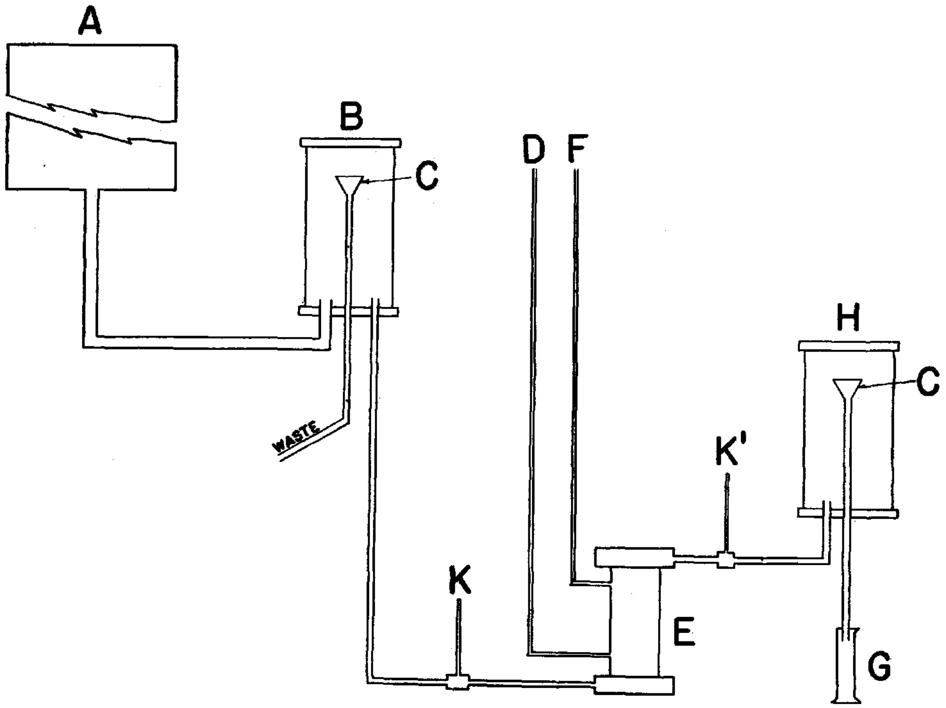


FIG. 17.—Schematic Diagram of Equipment Used for Measuring Permeabilities in Ground-Water Investigations.

lates the system from the atmosphere. After vacuum has been applied for about 2 min, timer *C* closes solenoid valve *D* and opens valve *E*, and the tank returns to atmospheric pressure. Then timer *F* closes the circuit to solenoid valve *G*, which opens to permit gravity flow from the evacuation chamber to the sand filter and reservoir below. After about 4½ min of gravity flow, solenoid valve *G* is closed by *F*. Timer *C* then takes over and starts the cycle again.

This device, constructed as described, permits the rapid degassing of water and provides a continuous supply of about 100 ml per min of practically air-free water, sufficient to operate several permeameters simultaneously.

MEASUREMENT OF PERMEABILITY

The actual procedures involved in permeability measurement are simple. A degassed water, preferably one in chemical equilibrium with the sediment, as suggested earlier, is used to saturate the sample and to maintain saturation during the test. The tube in which the sample is taken serves as the barrel of the permeameter without removal of the sample.

The schematic diagram of the constant-head apparatus for measurement of permeability is shown in Fig. 17. *A* is the deairing equipment, *E* is the permeameter, *D* and *F* are manometers. *B* and *H* are constant-head control tanks, intake and discharge, respectively; *C* is a constant-level funnel for the intake or discharge; *G* is a collecting vessel for the discharge from *A*. The form of *H* is subject to modification, depending on the sample under test and particularly its discharge rate. Measurements of head differential, discharge rates, and dimensions of the permeameter are sufficient to calculate the permeability. Thermometer wells, for measuring the temperature of entering and outgoing water, are placed at *K* and *K'*.

SUMMARY

The problem of determining the permeability of sediments in the laboratory has been discussed as it relates to groundwater investigations. Some of the significant factors in permeability measurements are discussed briefly. These are (*a*) influence of physical condition of the sample, (*b*) the effect of ion exchange on permeability of samples containing clays, and (*c*) the effect of occluded air on the observed permeability.

It is emphasized throughout that reliable permeability investigation can be accomplished only if all aspects of such studies are considered. These begin with field sampling and end with laboratory measurement. Only by such careful procedure can the erratic results so often seen in permeability measurements be eliminated.

It is suggested that the anticipated application of the data to be obtained be given detailed consideration when designing a permeability test.

Acknowledgments:

The authors acknowledge the help of A. H. Frazier of the Water Resources Division Equipment Laboratory, U. S. Geological Survey, Columbus, Ohio. Mr. Frazier built the prototype of the sampler described in this paper and made many helpful suggestions during construction. George H. Taylor, of the Survey's Ground Water Branch and Regional Engineer in charge of the Missouri River Basin program, made many suggestions. Gordon Andreason and S. J. Spiegel, both of the Geological Survey, gave much assistance to the authors.

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DISCUSSION

MR. ARNOLD I. JOHNSON.[1]—Smith and Stallman describe very well some of the important factors to consider when determining the permeability of soils and sediments. The Hydrologic Laboratory of the Geological Survey, formerly located at Lincoln, Nebr., and recently moved to Denver, Colo., has studied many of these factors and for a number of years has used sampling equipment and permeability apparatus similar to that described by Smith and Stallman, but modified by the writer. It seems pertinent to emphasize some of the limitations of equipment of this type.

The need to use undisturbed samples cannot be overemphasized. The permeability of the secondary structure can be many times more than the permeability of the primary structure.

Considerable field experience with the piston sampler described has shown that it is effective under many conditions. It generally must be used at exposed locations such as the land surface, test-pit wall, exposed faces of road cuts, walls at stream channels, etc. It has been used in the zone of saturation in shallow pits. However, modifications would be needed to make it suitable for use at depth in a borehole. This—like other existing samplers—does not take completely undisturbed samples of all materials under all conditions. In our experience, very moist materials were commonly compacted during sampling, and very dry silt or clay samples usually were badly fractured.

The test fluids used in permeability measurements should be considered carefully. Water of almost any composition probably will react differently, in flow through a sample, from oil, gas, or other test fluids. Likewise, water of one chemical composition may react differently from that of another composition. Therefore, it is seen that the test fluid should be similar in all ways to the fluid that is, or will be, intimately associated with the soils or sediments being tested. For ground-water investigations it would be ideal if the test fluid were of the same chemical composition as the water found in the formation from which the sample was obtained. This idea is recommended for small research projects, but when samples are shipped in, in quantity, from all over the United States, such as they are at the Denver Laboratory, it is not economically feasible to have enough of the actual water also shipped in, or prepared in quantities sufficient for routine use, unless desired for a special study. However, it is feasible to use a de-aired water system for all permeability tests.

The Denver Laboratory uses a permeability system similar in principle to that described in this paper, but because a large number of permeameters must be kept operating and a wide range of materials must be tested, the system differs considerably in physical features. One 30-gal tank for the vacuum chamber and another 30-gal tank for storage of the de-aired water are used. The timing cycle is set at 3½ min on vacuum and 3½ min on atmospheric

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[1] Chief, Hydrologic Laboratory, Ground Water Branch, Water Resources Div., U. S. Geological Survey, Denver, Colo.

pressure to provide a longer period for drawing the air out of the water. The tap water is sprayed in the tank near the top of the vacuum chamber. The permeameters themselves have had to be redesigned to adapt them for testing either disturbed or undisturbed samples of consolidated or unconsolidated materials. For samples of very low permeability, the permeameters, as described, have not been found satisfactory in our laboratory and have had to be adapted for use as variable-head permeameters.

The use of mechanically de-aired water as test fluid offers some difficulties, as well as advantages. The dissolved minerals, which are held in solution in the water by the dissolved carbon dioxide gas, may precipitate out upon removal of this gas in the de-airing process. The precipitation most likely may occur upon entrance of the water into the sample or within the entrance filter disk. This may cause progressive plugging of the sample by the colloidal precipitates, and an apparent progressive decline in permeability of the sample. However, from the investigations being conducted in the laboratory, it is known that the use of de-aired water may give results for permeability 2 to 5 times greater than those obtained by use of ordinary tap water on like samples.

In summary, it should be emphasized that the determination of permeability for soils and sediments is affected by many factors. Some of these still await definitive investigation, and the Denver Hydrologic Laboratory hopes to contribute to this end in the ensuing years.

MESSRS. W. O. SMITH AND R. W. STALLMAN (*authors*).—The drive sampler described in the paper was designed for use in exposed locations such as land surface, test-pit wall, exposed faces of road cuts, walls of stream channels, etc. It was not, of course, considered as generally applicable to all sampling problems. Rather, it was designed to incorporate many of the features contained in other individual sampler designs, such as the adjustable piston, transparent core retainer, core barrel guide, triangular base to provide access to barrel for relieving compaction, etc. The combination of these design features tends to make this unit more versatile than many other sampler designs which contain only one or two of the most desirable components.

Most soil technicians hesitate to take drive samples when soil moisture is very high or very low. Such an attitude has been common since the times of Dokuchaev and Glinka some 70 years ago. It is well recognized that, under drive sampling, very wet samples are usually compacted, and very dry ones generally fractured. It is possible, however, to obtain good undisturbed samples of very dry sediments if extreme care is used, but even then, only one out of three attempts may yield a good sample.

Samples of very moist or very dry soils can be obtained if a piston sampler is slowly jacked into the sediment. This procedure requires a modification of the techniques described in the paper. Jacking equipment has been designed for this purpose, and the results will be released after field trials are completed.

The need for a piston-type core barrel for use in boreholes is great. We have some preliminary designs for a piston sampler for use on a rotary drill stem, as well as jacking equipment for shallow boreholes. No construction has been attempted as yet.

Permeant fluids similar in composition to the water found in a formation are indicated only when clay-bearing sediments are under consideration (see Miss Foster's discussion). Water of a composition similar to that which is passing or is to pass through the sediment in the field is not required as a permeant in clay-free or nearly clay-free sands, for example. Some exceptions exist—for example, a sand containing a small amount of Na-montmorillonite. In many cases ordinary de-aired water is satisfactory as a permeant. If, however, its use is contraindicated, any results obtained will likely be of questionable value.

The use of de-aired water generally renders all pore space in a sample available for the transmission of the permeant. It eliminates the locking of air in pores by capillary forces, and the consequent reduction of pore space. It was recognized early in our investigations that some of the minerals initially held in solution in water by dissolved CO_2 may precipitate out upon removal of this gas in the de-airing process. Accordingly, a sand filter was inserted (unit—shown in Fig. 16) at the outlet of the storage tank to remove these precipitates. The sand filter did not prove to be effective for removing colloidal silica. However, MacQueen[2] found that the carbon disks at the sample ends did act as filters for the silica. If tap water contains appreciable amounts of silica, it can be removed by the inclusion of a carbon disk filter at the outflow end of the degassing chamber. These filters have proved to be very satisfactory.

Permeability investigations have been a subject of investigation by many hydrologists, engineers, and physicists since the times of Darcy and Hagen, a century ago. There are still problems to be solved, particularly those relating to the effect of clay minerals.

MISS MARGARET D. FOSTER[3] *(by letter)*.—Some factors of significance in determining the permeability of sediments have been discussed by Messrs. Smith and Stallman of the U. S. Geological Survey. Among these was the effect of ion exchange in samples containing clay. Another factor that may affect permeability measurements of clay-containing samples is the tendency of some clays to swell in water. Differences in the swelling characteristics of different clays may be related to their crystal structure, chemical composition, to the kind and degree of isomorphous replacements in their structure, and to the amount and nature of their associated exchangeable cations.

CRYSTAL STRUCTURE AND CATION-EXCHANGE OF CLAY MINERALS

The three great groups of clay minerals are the kaolins, the montmorillonites, and the hydrous micas, or so-called illites. In the kaolin group the unit structure consists of a silica sheet and an alumina sheet bound together. The sum of the positive charges of silicon and aluminum in these sheets exactly equals the sum of the negative charges of the O and OH ions; the structure has no unsatisfied electrical forces by which other ions can be held to it. The small cation-exchange capacity of the kaolin group of minerals—usually less than 15 milliequivalents per 100 g—is attributed to broken bonds on the edges of the particles. Clays of the kaolin group exhibit little or no swelling in water.

The montmorillonite group of clay minerals has the fundamental structural pattern of the micas—two silica sheets bound together by an alumina sheet. Isomorphous replacement of trivalent aluminum for some quadrivalent silicon in the silica sheets, or of magnesium or some other bivalent or univalent cation for trivalent aluminum in the alumina sheet, gives rise to net residual charges on the structure, which are neutralized by cations held electrostatically. These constitute the exchangeable cations. Thus, whereas kaolinite has a balanced structure and a low cation-exchange capacity, montmorillonite has an unbalanced structure, and, because of the

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[2] I. S. MacQueen, Engineer, U. S. Geological Survey (oral communication, 1950).

[3] U. S. Geological Survey, Washington, D. C.

cations held electrostatically to effect a balance, a high cation-exchange capacity —from 70 to 110 milliequivalents per 100 g.

Whether or not a montmorillonite swells greatly in water depends to a large extent on the character of the exchangeable cations. If the exchangeable cations are mostly calcium and magnesium ions, the montmorillonite swells only to about twice its dry volume; but if the exchangeable cations are largely sodium ions, the montmorillonite swells in water to 20 or more times its dry volume.

The hydrous mica group of clay minerals—by some called illites—has, like the montmorillonites, the fundamental structural pattern of micas. The hydrous micas have an even higher degree of isomorphous replacement in the structural sheets than the montmorillonites and consequently higher charges. However, these higher charges in the hydrous micas are to a large extent neutralized by potassium, which is fixed and not exchangeable. The cation-exchange capacity of the hydrous micas is usually greater than that of the kaolinites but less than that of the montmorillonites. They do not tend to swell greatly in water.

THE CAUSE OF SWELLING

A number of theories have been proposed to account for the swelling characteristics of clays. According to the theory generally accepted for many years, swelling is a function of the hydration of the exchangeable cations, and the great swelling of sodium montmorillonite as compared with calcium montmorillonite is due to the greater hydration of the sodium ion.[4] However, there is considerable divergence in the values given by different investigators for the size of the hydrated cations and for the number of molecules of water associated with each ion. Bernal and Fowler[5] presented data that created doubt as to hydration of some ions. Hendricks, Nelson, and Alexander[6] concluded that (*a*) the sodium ion, assumed to be most highly hydrated, is not hydrated at all, and (*b*) magnesium and the alkaline-earth cations have 6 molecules of water of hydration.

These and other differences of opinion as to the degree of cation hydration, and, in particular, as to whether the sodium ion is hydrated at all led Grim[7] and others[8, 9] to conclude that the concept of cation hydration as responsible for the thickness and other characteristics of the water film was not justified and that some other cause for the development of the water film must be sought.

As data from investigations of the relation of exchangeable bases to plastic properties indicate that the character of the adsorbed cation does affect the thickness of the water film, Grim[10] concluded that the adsorbed cations primarily control the thickness of the water film by means of the force with which they

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[4] G. Wiegner, "Some Physico-Chemical Properties of Clays. I. Base Exchange or Ionic Interchange II. Hydrogen Clays," *Journal*, Soc. Chemical Industry, Vol. 50, pp. 65, 103 (1931).

[5] J. D. Bernal and R. H. Fowler, "A Theory of Water in Ionic Solution with Particular Reference to Hydrogen and Hydroxyl Ions," *Journal of Chemical Physics*, Vol. 1, p. 515 (1933).

[6] S. B. Hendricks, R. A. Nelson, and L. T. Alexander, "Hydration Mechanism of the Clay Mineral Montmorillonite Saturated with Various Cations," *Journal*, Am. Ceramic Soc., Vol. 62, p. 1457 (1940).

[7] R. E. Grim, "Properties of Clay, Recent Marine Sediments," Thos. Murby and Co., London, p. 466 (1935).

[8] R. Houwink, "On the Structure of the Hydration Hull of Inorganic Soil Colloids," *Kolloid-Zeitschrift*, Vol. 93, p. 110 (1937).

[9] S. Spiel, "Effect of Adsorbed Electrolytes on the Properties of Monodisperse Clay-Water Systems," *Journal*, Am. Ceramic Soc., Vol. 23, p. 33 (1940).

[10] R. E. Grim, "Modern Concepts of Clay Minerals," *Journal of Geology*, Vol. 50, p. 225 (1942).

hold the sheets together. "According to this concept the great swelling of sodium-montmorillonite is due, not to the hydration of Na^+ serving as a wedge to force the layers apart, but to the absence of a strong bond holding the layers together because of the univalent character of sodium and because its small ionic size enables it to fit well into the hole of the silica sheet. Calcium-montmorillonite does not expand greatly because of the slight hydration of Ca^{++} but because the bivalent character of the ion tends to hold the sheets together so that a thick series of water sheets cannot form between them."

The high swelling of (sodium) bentonites in comparison with other clays strongly suggested to Baver and Winterkorn[11] that the former attract large amounts of water as a result of forces associated with the inner layer of the colloidal surface. In colloid chemical terms the nature of the inner layer and its effect on adsorption of ions and water molecules determine to a great extent the character and amount of hydration in aqueous clay systems, with the kind of ion adsorbed on the surface playing an important rôle.

In addition to these, a number of other theories have been proposed to explain the development of water films and the great swelling of sodium montmorillonite, but none seem as satisfactory as the concept of cation dissociation.

CLAYS AS COLLOIDAL ELECTROLYTES

From considerations of structure, Marshall and Krinbill[12] concluded that the clays of the montmorillonite group may be classed as colloidal electrolytes; that is, they are colloids the units of which are made up of a definite ionizing compound that is in true equilibrium and whose charge is due to ionization. When such a clay is dispersed in water, each unit can come to equilibrium with the outer solution; the exchangeable or electrostatically held cations tend to dissociate and the clay unit can be pictured as a complex anion. The suspended clay unit may be likened to a dissociated electrolyte with one of its ions in the colloidal range of dimensions.

Dissociation of the adsorbed cations leaves some of the structural units negatively charged. Thus charged, the units repel each other, and the clay appears to swell. The more complete the dissociation and the greater the number of units carrying a charge, the greater is the swelling; the less complete the dissociation, the fewer are the units carrying a charge and the less the swelling.

Marshall[13] has presented data showing the fraction active, that is, ionized, in the course of titration of an acid montmorillonite with NaOH and $Ca(OH)_2$. For sodium the values for active fraction, calculated from different parts of the titration curve, range from 0.007 to 1.20; for calcium the range is from less than 0.001 to 0.066. Near the point of neutrality there is almost a 20-fold difference in active fraction, 1.20 for sodium as against 0.066 for calcium. This great difference in the degree of dissociation of sodium and calcium montmorillonites can be correlated with the great difference in their swelling characteristics.

EFFECT OF IONIC SUBSTITUTION ON THE SWELLING OF SODIUM MONTMORILLONITES

Most studies on the swelling of montmorillonite have been on the effect of

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[11] L. D. Baver and H. Winterkorn, "Sorption of Liquids by Soil Colloids: II. Surface Behavior in the Hydration of Clays," *Soil Science*, Vol. 40, p. 403 (1935).

[12] C. E. Marshall and C. A. Krinbill, "The Clays as Colloidal Electrolytes," *Journal of Physical Chemistry*, Vol. 46, p. 1077 (1942).

[13] C. E. Marshall, "Ionization of Calcium from Soil Colloids and Its Bearing on Soil Relationships," *Soil Science*, Vol. 65, p. 57 (1948).

the exchangeable cation, although Kelley[14] observed that the swelling of sodium-saturated montmorillonites from different sources is not always proportional to the cation-exchange capacity and, furthermore, those having approximately equal cation-exchange capacity may swell quite differently. Grim[7] pointed out that the physical properties of clays may be ascribed to two factors —the character of the exchangeable cation and the composition of the clay mineral. By composition Grim referred to the makeup of the clay mineral, particularly with regard to substitutions within the structure.

In a study by the writer[15] on the swelling of montmorillonite, a range from 21 to 66 ml per g was found in the swelling volumes of twelve sodium-saturated montmorillonites. This range in the swelling volumes of samples that had the same exchangeable cation—sodium—strongly suggested that the swelling volume of montmorillonite is greatly influenced by some other factor or factors than the character of the exchangeable cation. The lack of correlation between swelling volume and total exchange capacity indicated that differences in the amount of sodium present due to differences in exchange capacity have little influence on swelling volume. On the other hand, there was good correlation between swelling volume and the amount of substitution—by iron and magnesium for aluminum—in the alumina layer, in the direction of decrease in swelling volume with increase in substitution. Samples with the lesser substitution had the higher swelling volumes; those with the greater substitution had the lower swelling volumes. The depressing effect of substituted ferrous iron was greater than that of substituted ferric iron, and the depressing effect of substituted ferric iron was about the same as that of substituted magnesium. Thus the composition of montmorillonite, at least with respect to the amount of iron and magnesium present, is an important factor in swelling.

If swelling is assumed to be related to dissociation of adsorbed or exchangeable cations, the decrease in swelling found to accompany increase in substitution of other cations for aluminum in the alumina layer suggests that these substitutions affect the degree of dissociation of the exchangeable cations; in other words, the substitution in some way affects the strength of the clay unit as an anion.

Because of differences in polarizing power, any ion substituting for another must cause a change in the polarization, particularly of neighboring ions. However, it is reasonable to assume that not only is this effect felt by the nearest neighbors but it is reflected throughout the whole structure, thus altering the energy relationships of the whole. The greater the substitution, the more the internal energy relations of the structure will be affected. Such an alteration in the energy relationships of the structure may well affect the bonding energy and the degree of dissociation of the exchangeable cations on dispersion of the montmorillonite in water. The decrease in swelling volume found to accompany increase in substitution in the alumina layer seems to indicate that the effect of the substitution is in the direction of decrease in the degree of dissociation of the montmorillonite. Although a colloidal electrolyte like montmorillonite cannot be strictly compared with a

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[14] W. P. Kelley, Review and Comments on Mattson's Papers on "The Laws of Soil Colloidal Behavior," *Soil Science*, Vol. 56, p. 443 (1943).

[15] Margaret D. Foster, "Geochemical Studies of Clay Minerals (II): Relation Between Ionic Substitution and Swelling in Montmorillonites," *American Mineralogist*, Vol. 38, p. 994 (1953).

soluble electrolyte, the change in the strength of acetic acid brought about by the substitution of one or more of the H atoms by positive or negative atoms or groups—such as Cl, OH, or CH_3—and the increase in the strength of ammonium hydroxide caused by the substitution of one or more of the H atoms by CH_3 groups are examples of the way in which the degree of dissociation of a compound can be changed by substitutions.

Although the hydrous micas have the same structural pattern as montmorillonites and have an even greater amount of isomorphous substitutions in their constituent layers and a greater charge on the structural units, they cannot be regarded as colloidal electrolytes. Most of the charge on the structural unit is usually balanced by potassium, which is bound in a different way from sodium and calcium. Potassium does not dissociate when the hydrous mica is dispersed in water; it cannot be exchanged as can sodium and calcium. Potassium holds the units making up the particles firmly together; exchangeable cations in the interior of the particle are inaccessible to water and cannot dissociate—the number of cations available for dissociation depend on particle size and not on the total number present as in montmorillonite. As relatively few of the electrostatically held cations present are available for dissociation, the degree of dissociation is correspondingly low regardless of the character of the exchangeable cation, and the swelling is of a very low order.

Marshall[13] found that the exchangeable cations of kaolinite are dissociated to a greater extent than those of montmorillonite, except in montmorillonite near the point of neutralization. However, the number of exchangeable cations present is so small that this greater dissociation does not lead to any great amount of swelling.

APPLICATION OF THE SWELLING CHARACTERISTICS OF CLAYS TO PERMEABILITY MEASUREMENTS

From the foregoing discussion it may be seen that the significance of swelling as a factor affecting permeability measurements of clay-containing sediments depends on the kind of clay present. If the clay is a kaolin or a hydrous mica, the effect of swelling would be of little importance, as these clays swell little, if any, in water. The presence of calcium montmorillonite may affect the measurement to some extent, as calcium montmorillonites swell to about twice their dry volume in water. But sodium montmorillonite, with its characteristically great swelling in water, would be expected to have considerable effect on the permeability, the extent of the effect depending not only on the amount of sodium montmorillonite present but also on its composition. Thus swelling as a factor affecting permeability measurements is of great importance only when the clay present is a sodium montmorillonite.[16]

MESSRS. SMITH AND STALLMAN.—Miss Foster has given a very clear outline of ion-exchange phenomena which play an important part in the transmission of water in clay-bearing sediments. Its role in the swelling of clays, and the consequent effect on permeability, are of great importance.

If dry air or organic liquids were used as permeants the observed permeabilities would be relatively high. The action of water is quite different because of ion-exchange phenomena; and thus it is apparent that, if the permeability to water is desired, water is generally the only permeant that can be used.

In this connection it should be recog-

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[16] Publication authorized by the Director, U. S. Geological Survey.

nized that all clays usually swell almost instantaneously once water is in contact with them. The amount of swelling depends upon the type of clay. For example, Na-montmorillonite swells to about 15 or 20 times its initial volume, whereas Ca-montmorillonite swells to about twice its original volume. Kaolinite swelling is a matter of several per cent and may be significant in some tests.

A knowledge of the clay present in a sediment is often useful when selecting permeameter equipment suitable for a given sediment. If, for example, the sediment consists of a sand matrix in which the clay embedded in the pores contains large amounts of Na-montmorillonite and the permeant is pure water, a variable-head permeameter properly protected for evaporation is better suited to a measurement of permeability than is a constant head device. This is because the excessive swelling of the Na-montmorillonite will almost completely seal off the pores, resulting in very low permeability; and the consequent low flows cannot be measured accurately with the constant head device described.

Air removal is best accomplished by subjecting the sample to a high vacuum prior to the introduction of water. A mercury head can be used to drive water through a clay sample in a variable-head permeameter. Its use eliminates evaporation of the inflow water. With a sediment containing limited amounts of Ca-montmorillonite, a constant-head permeameter probably would be more suitable because of the existence of a more permeable pore space.

In connection with the ion-exchange mechanism outlined by Miss Foster, two additional items should be mentioned. First, the type of exchangeable ions affects the migration of clays in sediments and the subsequent packing. Second, there is the role of organic ions in exchange phenomena. Many of the organic ions show a behavior similar to that of the inorganic ions. Hendricks[17] has studied the exchange phenomena in montmorillonite when organic ions are present.

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[17] Sterling B. Hendricks, "Base Exchange of the Clay Mineral Montmorillonite for Organic Cations and its Dependence Upon Adsorption Due to Van der Waals Forces," *Journal of Physical Chemistry*, Vol. 45, pp. 65–81 (1941).

DETERMINATION OF PERMEABILITY OF GRANULAR SOIL BY AIR SUBJECTED TO A DECREASING PRESSURE DIFFERENTIAL

By Arthur S. Weaver[1]

Synopsis

The object of this paper is to describe the design and operation of an apparatus for determining the coefficient of permeability of granular soils, utilizing air as the percolating fluid, under so-called falling-head conditions. Also, the results of a few typical tests are presented, and comparisons made with the results obtained using water as the test fluid.

The majority of investigators agree that, in determining the coefficient of permeability by the commonly used water-test procedures, it is necessary to dry the sample, evacuate it, and then saturate it with distilled, deaerated water to prevent clogging of the void spaces with air or solid contaminants. If it is desired to determine the permeability at several different values of void ratio, it is therefore necessary either to have a large amount of the material on hand or to dry out the sample between tests. When attempting to determine the coefficient of permeability by the use of water, it is found that if the material is in a relatively loose state, the seepage forces produced by the flow of the water may produce settlement of the sample. This action decreases the void ratio and, consequently, the coefficient of permeability. Since the computation of the void ratio is based on constant predetermined sample dimensions, the correction to be applied is difficult to determine accurately.

If air is used as the test fluid it is necessary only to dry the sample and to dry the air to be passed through the sample of soil. Furthermore, the tendency to settle is less, since the pressure gradient increases toward the low-pressure end of the sample. The seepage forces tending to produce settlement are therefore greatest at the low-pressure end, where a given particle may move the shortest distance, and least at the high-pressure end where the possible displacement is greatest.

If a constant pressure differential is applied to the sample, the coefficient of permeability may easily be calculated by the use of Darcy's law, upon taking suitable measurements of the absolute pressures at the ends of the sample, of the temperature of the air flowing, and of the volume passed during a given time. However, the problems involved in maintaining constant pressures, and especially in measuring the volume of air with sufficient accuracy, are such as to nullify any advantages this procedure has over the usual water test.

Testing Apparatus

In the author's apparatus (Fig. 1), the

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[1] Assistant Professor of Mechanical Engineering, University of Maine, Orono, Me.

air used is supplied by a compressor and passes through a dryer to a receiver tank. When the pressure in this tank has reached a predetermined value, the valve in the compressor line is closed, and the air is allowed to flow through a suitably prepared sample to the surrounding atmosphere.

The size and capacity of the compressor may be varied within wide limits, and, in fact, in certain installations it might prove desirable to use a cylinder of commercial compressed air as the source of the air for testing.

Silica gel is used as the drying agent, as it may be re-used upon heating it periodically to drive off the adsorbed moisture. A 41-cu in. refrigeration type unit is used in the present apparatus and is installed by means of flare type fittings, which simplifies its removal for heating.

No means of cooling the air after the compression and drying processes are necessary, as the heat thus liberated is dissipated through the walls of the piping before reaching the pressure tank.

The tank used in all testing mentioned in this paper is a cylindrical steel drum, 16 in. in diameter and 36 in. long. The volume of the tank was determined by filling it with deaerated water and measuring the weight and temperature thereof.

Two manometers are provided to measure the pressure in the reservoir. The tubes are of $\frac{1}{4}$-in. outside diameter by $\frac{3}{16}$-in. inside diameter transparent plastic, approximately 8 ft tall, and are connected to the metal piping system with flare type fittings. One manometer contains mercury and will measure pressures up to approximately 50 psig; the other contains water, and is used to obtain more accurate determinations of pressures in the range from 0 to 4 psig. A Bourdon-tube pressure gage is also installed to indicate the reservoir pressure before opening the valve to either manometer, as a safeguard against blowing the liquid out of the tube.

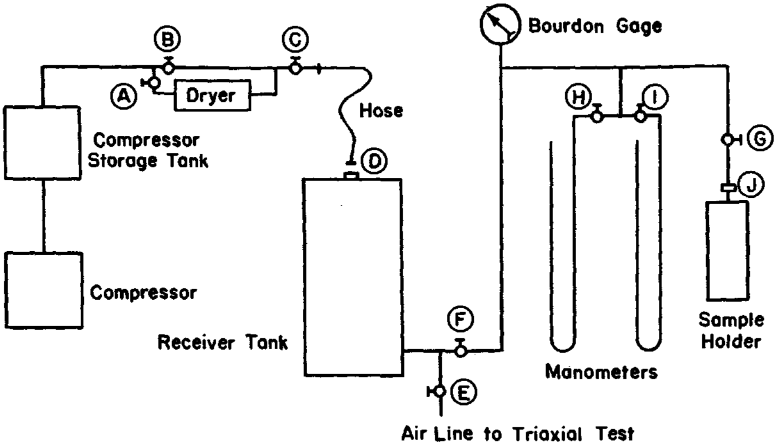

FIG. 1.—Schematic Diagram of the Air Permeameter.

The details of the sample holder are shown in Fig. 2. Originally, all parts were to have been made of brass; however, due to the then current unavailability of seamless brass tubing of the proper size, seamless steel tubing was substituted for the holder barrel, with very little difficulty experienced due to rusting. The sample is held in place by circular screens of 200-mesh brass cloth, backed by circular brass plates perforated with small holes. One screen and plate combination is made to fit snugly inside the barrel against the locating pins while the other fits into the annular space machined in the bottom cap. Leakage of air past the screw threads joining the upper cap to the barrel is prevented by a soft rubber ring, placed as shown. The unit is connected to the main piping by means of a quick coupler.

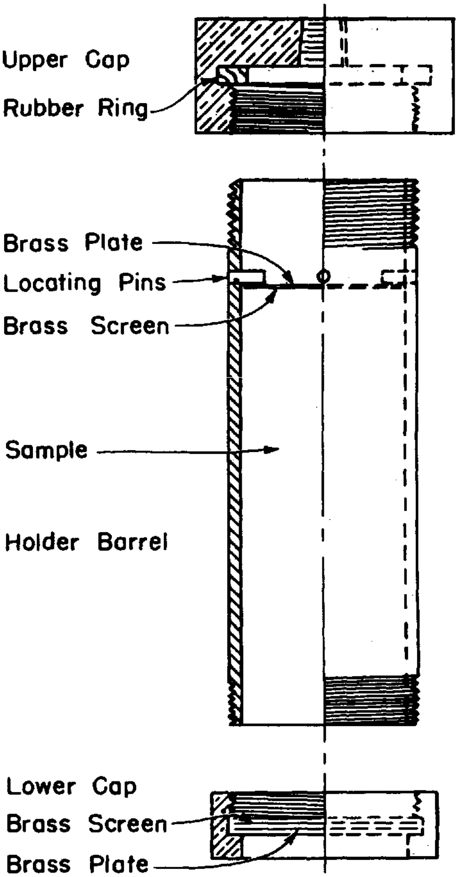

FIG. 2.—Detail of Sample Holder.

Originally the temperature of the air flowing through the sample was measured by thermistors placed immediately before and after the sample. However, this proved to be an unnecessary refinement, since a change of 1 C in the temperature of the air produces a corresponding change of only 0.3 per cent in the coefficient of viscosity. The temperature may be satisfactorily measured with a mercury-in-glass thermometer clamped directly below the sample holder outlet.

The atmospheric pressure is measured by means of a mercury column barometer.

The pressure in the tank, which is determined at the beginning and end of a suitable time interval, the atmospheric pressure, the temperature of the air flowing, and the various dimensional constants of the equipment provide sufficient information for calculating the coefficient of permeability.

THEORY

The following derivation shows the development of the equation used in such calculations. The soil will be considered to be isotropic and homogeneous, and the velocities and flow rates involved will be assumed to be so small that isothermal flow conditions will exist. Then, if the fluid flowing is a nearly perfect gas, Muskat[2] indicates that the flow relationship is

$$\nabla^2\gamma^2 = \frac{2n\mu\gamma_0}{k_p}\frac{\partial\gamma}{\partial t},$$

in which γ represents the density of the fluid, n the porosity, μ the viscosity, and γ_0 the density at atmospheric pressure. In this expression, k_p is the so-called physical permeability, which is related to the coefficient of permeability for any specific fluid by the relationship $k_p = k\frac{\mu}{\gamma}$. The k_p remains constant for a soil of

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[2] M. Muskat, "The Flow of Homogeneous Fluids Through Porous Media," McGraw-Hill Book Co., Inc., New York, N. Y. (1937).

given porosity and temperature and is independent of the properties of the fluid flowing. In the proposed testing apparatus, the flow is one-dimensional, so that

$$\frac{\partial^2 \gamma^2}{\partial x^2} = \frac{2n\mu\gamma_0}{k_p} \frac{\partial \gamma}{\partial t}.$$

Although this equation accurately represents the flow process, it is believed insolvable, at least in any usable form, and further simplification is necessary.

If the pressure is to be measured only at the extremities of the sample, it becomes unnecessary to consider the manner in which the density or pressure varies along the length of the sample. In the following discussion, the actual pressure-time relationship is replaced by a series of constant-head "steps." Analyzing the air flow during one so-called "step" shows that the tank pressure actually drops from P_x to $P_{x'}$, and a certain quantity of air is discharged. It is then assumed that during this "step," the tank pressure remains constant and equal to the average pressure, $P = \frac{P_x + P_{x'}}{2}$ while discharging the same quantity of air. The pressure then drops instantaneously to the average pressure of the next "step" and so on. At the beginning of the "step," $P_x V = w_x RT$, and at the end, $P_{x'} V = w_{x'} RT$, express the conditions in the tank. The weight of air lost, therefore, $\Delta w = w_x - w_{x'}$, is equal to $\frac{V}{RT}(P_x - P_{x'})$. Expressing Δw in terms of volume measured at the mean pressure in the sample, $\frac{P + P_a}{2}$,

$$\Delta V = \frac{2V(P_x - P_{x'})}{P + P_a}$$

in which P_a represents the pressure of the atmosphere at the sample exit. Writing Darcy's expression applied to the constant head "step," the volume rate of flow measured at the mean sample pressure may be written

$$Q = \frac{k_p A g (P - P_a)}{\mu L}.$$

The time to discharge a volume of air ΔV at a constant rate Q equals $\Delta V/Q$. Or,

$$\Delta t = \frac{2V(P_x - P_{x'})}{P + P_a} \cdot \frac{\mu L}{k_p A g (P - P_a)}$$

$$= \frac{2V\mu L}{k_p A g} \frac{P_x - P_{x'}}{(P + P_a)(P - P_a)}$$

Now, if the duration of each step is assumed to be infinitely small, $P_x - P_{x'} \sim \mathrm{d}P$, and

$$\mathrm{d}t = \frac{2V\mu L}{k_p A g} \frac{\mathrm{d}P}{(P + P_a)(P - P_a)}$$

By integration, the total time to reduce the tank pressure from an initial value P_1 to some lower value P_2 may be found:

$$T = \frac{V\mu L}{A g k_p P_a} \ln \left| \frac{(P_1 + P_a)(P_2 - P_a)}{(P_1 - P_a)(P_2 + P_a)} \right|$$

Since, during the actual testing procedure, the pressures are obtained by means of manometers, this equation may be written, after rearranging terms,

$$k_p = \frac{V\mu L}{T A g P_a} \ln \left| \frac{(P_1' + 2P_a)(P_2')}{(P_2' + 2P_a)(P_1')} \right|$$

where P_1' and P_2' are gage pressures. Finally, since the results of this investigation will be of use primarily in soils engineering applications, the foregoing expression should be changed to read in terms of the coefficient of permeability with respect to water flow, or,

$$k_w = \frac{V L \rho_w \mu_a}{T A P_a \mu_w} \ln \left| \frac{(P_1' + 2P_a)(P_2')}{(P_2' + 2P_a)(P_1')} \right|$$

where ρ_w and μ_w are based on standard conditions of temperature and pressure, and μ_a is based on the temperature in the sample. V is the volume of the pressure tank and piping leading to the sample; L and A are the length and cross-sectional area, respectively, of the sample.

Certain other factors must be considered in conjunction with the use of this equation in permeability determinations. One may not merely apply an arbitrary initial pressure and measure the time for it to decrease a certain amount, because it is necessary for laminar flow to exist in the flow passages for the foregoing analysis to be valid. It is suggested that the presence or absence of laminar flow be determined experimentally, rather than by an attempt at mathematical analysis. If the sample is compacted to a certain density and several consecutive tests are made, the second having a lower initial pressure than the final pressure of the first, and so on, the coefficients of permeability as computed from these tests should be approximately equal. If turbulence exists at the higher pressures, a markedly lower value of the coefficient will be noted.

At the beginning of a test, when the valve between the tank and the sample is opened, a transient flow condition of somewhat uncertain duration is created, during which no valid permeability determinations may be made. It was found experimentally that, in all cases, the transient condition had essentially disappeared when the tank pressure had decreased to approximately 70 per cent of its original value. Tests made during this period of transition indicate an apparent coefficient of permeability that is much greater than the true value. The equations given above are based on the presence of falling-head flow throughout the sample, whereas at the instant of opening the valve, flow exists only at the face of the sample nearest the source of air. As the time interval increases, flow is established further along the sample, until falling-head conditions prevail throughout. During this transition period, then, the effective length of the sample increases from zero to the actual length and is reflected by a coefficient of permeability which decreases from infinity to the true permeability of the material.

TEST PROCEDURE

The procedure used in testing with the air permeameter follows.

Sample:

The sample was oven-dried to constant weight and mixed until uniform.

Procedure:

1. *Preparation of the Sample in the Sample Holder.*

The arrangement of the components of the sample holder is shown in Fig. 2. The material tested was placed in the barrel dry and in layers of approximately equal thickness. The layers were compacted with a tamping rod, the amount of compaction depending upon the void ratio desired. After each layer was compacted, the surface was scarified to insure uniform blending of the material at the boundary. Great care was taken to prevent stratification or pocketing of particles of uniform size.

2. *Testing.*

(*a*) After compaction of the sample, the weight of the sample was determined and the brass caps screwed on tightly.

(*b*) The assembled sample holder was attached to the test panel by means of the quick coupler, J (Fig. 1).

(*c*) Valves $A, C, F,$ and G were opened; valves $B, E, H,$ and I were closed.

(*d*) Either valve H or I was opened, corresponding to the manometer to be used.

(*e*) Compressed air was admitted to the pressure tank by connecting the rubber hose to the spring-loaded valve, D.

(*f*) When the manometer indicated the desired pressure, the hose was removed, closing *D*.

(*g*) The reservoir pressure was allowed to drop to approximately 75 per cent of its original value.

(*h*) After thus eliminating the transient effect, the time for the liquid in the manometer to fall a measured distance was determined by means of a stop clock.

(*i*) The air temperature and the atmospheric pressure were measured as described previously.

The procedure followed in determining the coefficient of permeability of the same materials using water is found in the ASTM publication, "Procedures for Testing Soils,"[3] with one modification to the equipment: plastic spacer rings were substituted for the Ottawa sand, so that the length of the sample could be kept more nearly constant, and to facilitate assembly of the unit. The temperature of the water was determined by thermistors located at both ends of the soil sample, as well as by placing a thermometer in the water supply bottle. Since the coefficient of viscosity of water varies by approximately 2.5 per cent per deg Cent change in temperature, it is necessary to effect a more accurate determination of the water temperature than of the air, in order that the results of the two types of test may be compared. It was found that the water supply temperature did not accurately represent the temperature of the water actually percolating through the sample.

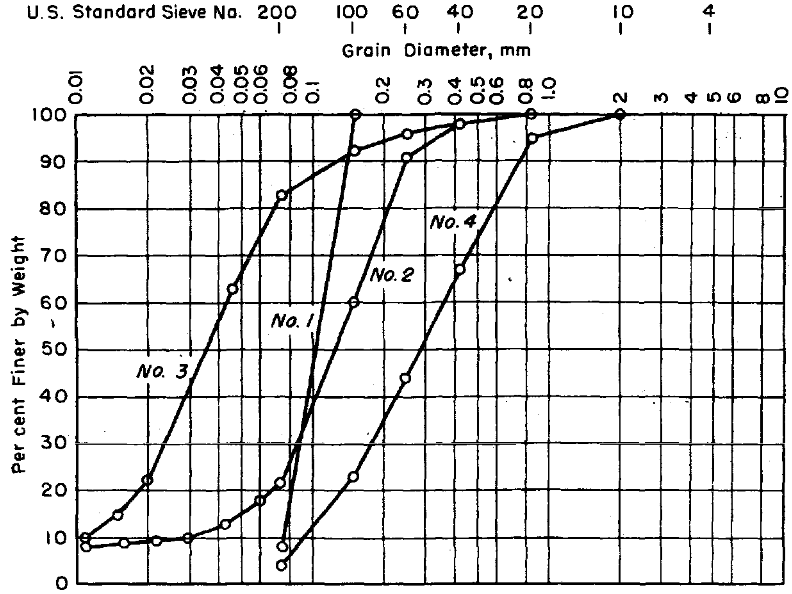

FIG. 3.—Grain Size Distribution Curves for Samples Tested in the Air Permeameter.

TEST RESULTS

Several samples of granular soil of varying description were tested in accordance with the foregoing procedure with most satisfactory results. The grain-size distribution relationships of four

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[3] Procedures for Testing Soils, Am. Soc. Testing Mats., July, 1950, p. 179.

typical samples are illustrated in Fig. 3. Sample 1 is an artificially prepared material consisting of particles passing a No. 100 sieve and retained on a No. 200 sieve of the U.S. Standard sieve series. Samples 2, 3, and 4 are natural soils obtained from the Soils Laboratory of the Maine State Highway Commission. Void ratio *versus* coefficient of permeability relationships obtained by testing these four soils in the apparatus described in this paper are illustrated in Fig. 4. It is noted that in each case the experimental relationship conforms very closely to the expression, $k \approx \frac{e^3}{1+e}$, suggested by Taylor.[4] Also shown are values of the permeability coefficient obtained by the water test specified previously. It is seen that there is very close correlation between the results of the two types of tests on both samples 1 and 4. The discrepancy evidenced in the water and air values for sample 2 is due to malfunctioning of one of the thermistors used to measure the water temperatures. The water test could be performed on sample 3 only at the lowest void ratios obtainable, due to the marked instability at lesser densities. The high vacuum used to deaerate the samples, combined with the large capillary rise naturally occurring in a silt, drew the saturating water into the pores of this sample faster than it was supplied. Because of the extremely high velocity of the water, "boiling" and complete structural rearrangement resulted, with ensuing segregation of coarse and fine particles and subsidence. Two consolidation tests were performed in an attempt to determine the permeability by other means, but due to the extreme sensitivity and rapid consolidation rate of the material, no valid results were obtained.

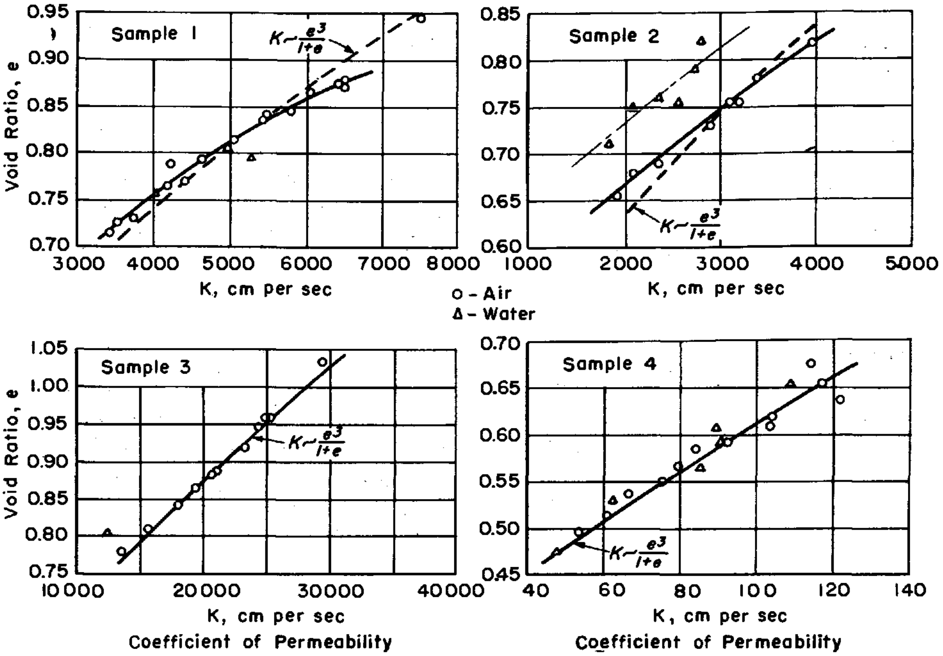

FIG. 4.—Void Ratio *versus* Coefficient of Permeability.

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[4] D. W. Taylor, "Fundamentals of Soil Mechanics," John Wiley and Sons, Inc., New York, N. Y. (1948).

CONCLUSIONS

The relationship between the void ratio and the coefficient of permeability is found to correspond very closely to the empirical relationships proposed by other investigators as representing the average behavior of granular materials. This fact is taken as experimental verification of the theory that the flow of air or other gases through a porous material is subject to the same laws that govern the flow of liquids. The use of a gaseous fluid rather than a liquid fluid for testing purposes requires a slightly more involved application of the basic theory, which is compensated for by simplification of certain aspects of the testing procedure and, in certain instances, increased accuracy.

If the procedure described above is adhered to, the values of the coefficient of permeability obtained by means of the air test will exhibit less deviation from the mean curve than will those resulting from the water test. A greater variety of materials may be tested in the air permeameter, and a greater range of void ratio may be utilized in the testing of each material. It must be emphasized that the results of air permeameter tests represent actual subsurface conditions no more nor less accurately than do water-test results, and should not be used without careful evaluation and comparison of laboratory and field conditions. The air permeameter is simpler, less expensive, and less time-consuming to operate and eliminates the nuisance attendant upon the use of water. It would, furthermore, be more suited to installation in a field or mobile testing laboratory.

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