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Measurement of the Hydraulic Conductivity of Fine-Grained Soils

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ABSTRACT: The purpose of this paper is to review the state of the art in the measurement of hydraulic conductivity of fine-grained soils. Both field and laboratory tests for saturated and partially saturated soils are considered.

For saturated soils, field tests are to be preferred because they permeate a larger volume of soil, thus taking into account the effects of macrostructure better than laboratory tests. Field tests are generally best performed by using a cylindrical piezometer tip, installed by methods that minimize disturbance, and measuring flow under a constant head. Laboratory tests offer the advantage of economy. Laboratory specimens should be as large as practical and should be oriented to produce flow in the direction of maximum hydraulic conductivity. The permeant should be a fluid similar to that encountered in the field. Without proper experimental technique, the conductivity measured in the laboratory may differ from the field value by several orders of magnitude.

Field tests for unsaturated soils are not well developed and can only be recommended for cases where water will be ponded on the surface of a site. The most versatile laboratory techniques are the instantaneous profile method using tensiometric or psychrometric probes, and the pressure plate outflow method. The best method to use depends on the soil suction expected in the field.

KEY WORDS: permeability, hydraulic conductivity, soils, fine-grained soils, clays, sample preparation, laboratory tests, field tests, suction, unsaturated soils, groundwater

Analyses of water flow in saturated soils are usually based on Darcy's [*1*][2] law which, in turn, is based on the experimental observation of a linear relationship between the rate of flow and the driving forces. The law has been written in many forms depending mainly on the discipline of the user and the date of usage. The form most commonly encountered is

$$q = -kiA \tag{1}$$

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[2]The italic numbers in brackets refer to the list of references appended to this paper.

where q is the flow rate [length^3/time (L^3/T)]; i is the hydraulic gradient (dimensionless); A is the total cross-sectional area of flow (L^2); and k is the constant of proportionality (L/T), which is termed the hydraulic conductivity in most disciplines but is often termed the permeability by civil engineers. In Eq 1, k is dependent on the properties of both the fluid and the porous medium. An alternative form of Darcy's law is

$$q = -K\frac{\gamma_w}{\mu}iA \tag{2}$$

where γ_w [units of mass/length^2 time^2 (M/L^2T^2)] and μ (ML/T) are the unit weight and viscosity of the fluid, respectively, and K is the constant of proportionality (L^2). In most disciplines K is termed the coefficient of permeability but is sometimes termed intrinsic permeability [*2*]. A third form of Darcy's law is [*3*]

$$q = -\frac{k}{\mu}\frac{dp}{dx}A \tag{3}$$

where q is the flow rate (cubic centimetres per second), μ is viscosity (centipoises), p is pressure (atmospheres), x is flow distance (centimetres), A is total cross-sectional area (square centimetres), and k is the permeability in units of "darcies." For a permeant of pure water at 20°C the conversions are [*4*]

$$1 \text{ cm/s} = 1.02 \times 10^{-5} \text{ cm}^2 = 1.04 \times 10^3 \text{ darcy}$$

Because of its simplicity and the fact that we are concerned with the flow of a single fluid, water, we prefer to use Eq 1. To conform with general usage we will also term k the hydraulic conductivity, or, where the meaning is clear, simply conductivity. However, when considering general matters not concerned with a specific equation or parameter we will use the term permeability, as, for example, "permeability tests."

Most engineers and geologists are familiar with techniques for measuring k in coarse-grained soils—for example, constant or falling head tests in the laboratory and pump tests from wells in the field—but are less familiar with techniques for measuring k in fine-grained soils. The reason for this, in the case of geologists, is probably that primary interest has traditionally been in development of groundwater supplies, which cannot be extracted economically from fine-grained soils due to their low hydraulic conductivities. In engineering, past practice has frequently been to assume that fine-grained soils are effectively "impervious" and to forgo attempts to measure k. The need for measurements of hydraulic conductivity in fine-grained soils seems to be increasing as a result of recent developments. One such de[illegible]ment is in-

creased concern over the long-term environmental effects associated with burying toxic wastes in the ground. When consideration is given to water and pollutant movements over periods of up to hundreds of thousands of years, the fine-grained soils can no longer be treated as impervious and their conductivities must be measured. The attractiveness of sites in arid regions for disposal of radioactive and other toxic wastes has also created interest in measuring the conductivity of partially saturated soils.

Another area of increased interest involves consolidation problems; evidence now indicates that the accuracy of field predictions may be improved by using either laboratory [5] or *in situ* [6] measurements of conductivity, as opposed to evaluating the coefficient of consolidation directly by fitting theoretical curves to laboratory time-settlement data [7]. Measurements of hydraulic conductivity in fine-grained soils have always been, and will continue to be, of concern in design of earth dams, slurry trenches used as groundwater cutoffs, and compacted clay used to line water-storage reservoirs or waste-disposal pits, as well as in hydrological investigations of groundwater recharge.

In the pages that follow, we will examine the state of the art for measuring the hydraulic conductivity of both saturated and partially saturated fine-grained soils, including both field and laboratory methods. It is convenient to discuss the relative merits of laboratory versus field tests first. Then laboratory testing for both saturated and unsaturated soils will be considered, followed by a similar discussion of field testing. Finally, laboratory and field values will be compared. Testing techniques and testing errors will be discussed. No attempt will be made to cover in detail all methods that have been used, but references will be made to numerous papers to aid in a more expanded study.

Laboratory Versus Field Measurements

Soils tend to be nonhomogeneous. Fine-grained soils may be stratified on a large scale but may also be nonhomogeneous on a small scale as a result of sand partings, fissures and joints, and root holes. It is desirable, therefore, to test a volume of soil that is large enough to contain a statistically significant distribution of these features. Such volumes are almost always too large to be included in laboratory tests. Further, the results of laboratory tests may be influenced by effects of sampling disturbance, by various laboratory errors to be discussed subsequently, and by a tendency to select the most uniform, intact samples for testing. The soil is also likely to be anisotropic, and laboratory samples are not likely to be oriented so that flow occurs in the direction of highest conductivity. Clearly, field tests will be required in many cases.

There are several cases in which laboratory tests seem appropriate. In the case of compacted soils, permeability tests on soil prepared at various densities and water contents are most readily performed in the laboratory. Comparative tests to determine the effects of different permeants on conductivity are also typically performed in the laboratory.

Two practical reasons may also be advanced for use of laboratory tests. The first is economics. Large numbers of laboratory tests can be performed in a well-equipped laboratory with minimal expense, but field tests can be prohibitively expensive. A second reason for using laboratory tests is ignorance of field testing methods. Essentially all soil mechanics texts mention acceptable laboratory testing methods, but none currently provides a similar treatment for field measurements of the hydraulic conductivity of fine-grained soils.

Laboratory Tests for Saturated Soils

Permeability Cells

The standard laboratory tests for saturated fine-grained soils generally have the soil in the form of a disk with radial boundaries of metal, or occasionally plastic, and with the flow vertically upwards. Consolidation cells work well for undisturbed samples, whereas compaction molds are used directly for compacted soils.

A simple design for a consolidation-cell permeameter is shown in Fig. 1. The ring has a sharpened upper edge to facilitate trimming. With such apparatus and hand trimming, values of k less than 1×10^{-12} cm/s have been measured. Mechanical devices [8,9,10] may be used to prevent tilting of the ring during trimming and thus minimize the possibility of creating voids between the soil and the ring.

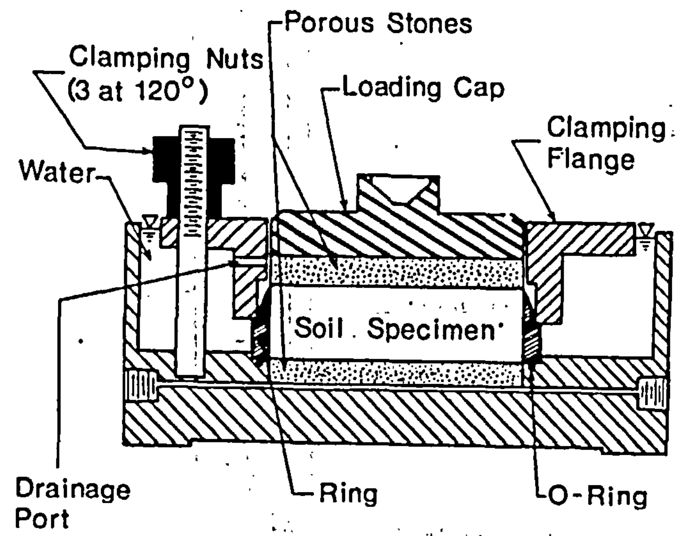

FIG. 1—*Consolidation cell permeameter.*

Cells for samples from about 4 to 10 cm (1.5 to 4 in.) in diameter, and heights up to about 10 cm (4 in.), can usually be mounted in standard consolidation loading frames if the effect of void ratio or effective stress on conductivity is to be determined. The base should be provided with two drainage lines to facilitate flushing with deaired water.

For compacted soils, a standard 10-cm (4-in.) diameter mold can be used with a special base containing a porous stone. If desired, a top plate may be sealed against the top of the mold and a vacuum may be applied to the top during the saturation stage. For soils containing coarse particles a larger mold may be used [*11,12*]. With such apparatus it is common practice to prevent soil swelling by clamping a plate against the top of the sample, but calibrated springs may also be used [*12*].

Permeability tests are also conventionally performed in standard triaxial cells [*13*]. This apparatus has the advantages that back pressure can be used to promote saturation and the applied total stresses can be controlled. Similar advantages accrue from use of back-pressure consolidation cells [*14,15,16*].

Standard Test Methods

Constant Head—In the constant head test, the hydraulic gradient i is maintained constant at a value h/L, where h is the head loss associated with flow through the soil sample and L is the length of the sample, and the total volume of flow, Q, is measured during a time period, t. From Eq 1

$$k = QL/hAt \tag{4}$$

For fine-grained soils the constant head is typically applied using a Mariotte bottle, one example of which is shown in Fig. 2 [*12*]. Such equipment is designed to apply only small heads (a few feet of water) so it is most useful with rather pervious soils or in cases where prolonged testing times can be tolerated. The main advantages of constant head tests are the simplicity of interpretation of data and the fact that use of a constant head minimizes confusion due to changing volume of air bubbles when the soil is not saturated.

Falling Head—A more common test for fine-grained soils is the falling head test in which the time, t, for the head loss, h, to decrease from h_1 to h_2 in a volumetric tube, typically a pipet or a buret with cross-sectional area, a, due to flow through a sample of area, A, and length, L, is measured. Equation 1 is used with $q = adh/dt$ to obtain

$$k = \frac{aL}{At} \ln \frac{h_1}{h_2} \tag{5}$$

The advantage of using this procedure is that small flows are easily measured using the pipet or buret. The observation time may still be long, in which case correction for water losses due to evaporation or leakage may be needed.

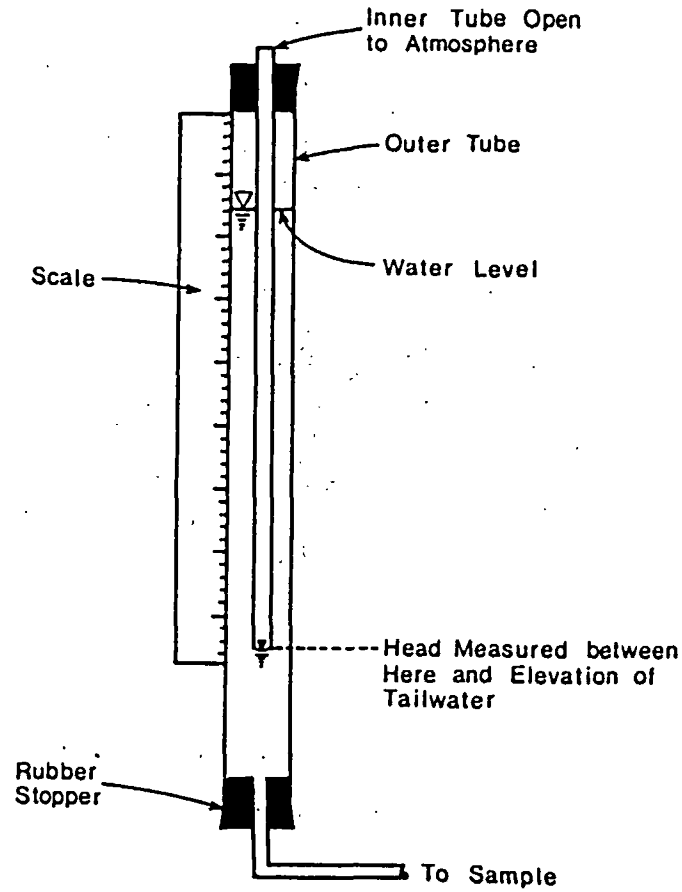

FIG. 2—*Mariotte bottle for maintaining constant head and measuring flow rates [12].*

The testing time may be reduced by increasing the flow rate, for example, by superimposing an air pressure, u_a, on top of the water in the pipet, thus increasing the heads by an amount u_a/γ_w. An apparatus such as that shown in Fig. 3 works well for this purpose; it allows the pipet to be refilled without removing the air pressure. However, two problems still exist. First, there is a tendency to superimpose excessive heads. Second, the water in the pipet tends to become saturated with gas at elevated pressure, in accord with Henry's law [*17,18*]. As the water flows through the soil sample, and the water pressure drops, there may be a tendency for air bubbles to evolve in the sample. This problem is minimized by replacing the water in the storage reservoir with deaired water periodically and flushing that water through the base to remove water containing excessive gas. Alternatively, the volume change device

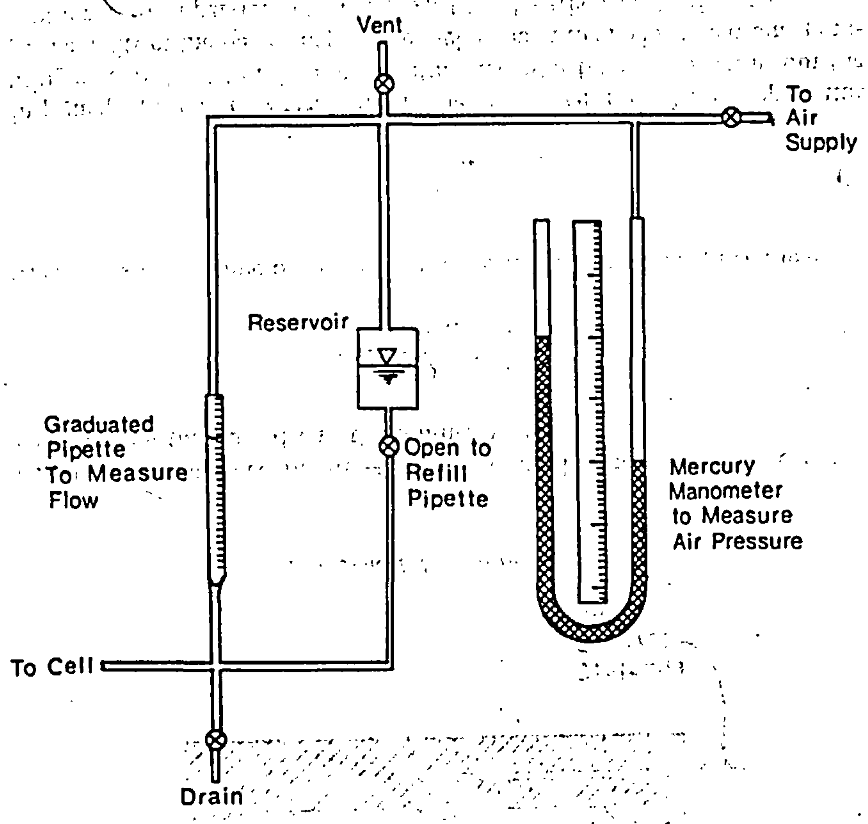

FIG. 3—*Apparatus for superimposing air pressure on water.*

reported by Mitchell, Hooper, and Campanella [*19*] may be used or pressure may be obtained from the hydrostatic pressure of a column of mercury and a "single buret volume change" device [*20,21*] may be used. In cases where the applied pressure substantially exceeds that of the water head, the test becomes essentially a constant head test and Eq 4 is used.

A falling head test naturally lends itself to automatic data recording using a differential pressure transducer mounted in the base of the cell with the tailwater reservoir connected to the reference port.

Special Test Methods

One-Dimensional Consolidation Testing—The hydraulic conductivity can be calculated from the coefficients of consolidation, *c*, and compressibility, *a*, and void ratio, *e*

$$k = ca\gamma_w/(1 + e) \quad (6)$$

(see Ref *7*, page 228). Terzaghi [*22*] found that the measured k (k_m) from conventional permeability tests and the value calculated using Eq 6 (k_{KT}) were essentially equal for one soil. Casagrande and Fadum (Ref *23*, page 480) also found substantial equality, provided there was a distinct break between primary and secondary consolidation, but presented no supporting evidence. Taylor [*24*] found significant differences between k_m and k_{KT}.

We have compared k_m and k_{KT} for numerous samples of undisturbed, remolded, and resedimented clays. Typical data are shown in Fig. 4. For highly overconsolidated clays k_m/k_{KT} ranges from perhaps 2 to 1000, whereas for normally consolidated clays the ratio varies from about 0.9 to 5. The discrepancy between k_m and k_{KT} presumably results, at least in part, from the fact that the classical theory of consolidation makes no adjustment for the structural viscosity of the soil.

Radial Flow Tests—The horizontal conductivity can be measured in the laboratory using a cell (Fig. 5) with a central sand drain (radius = r_w) and a

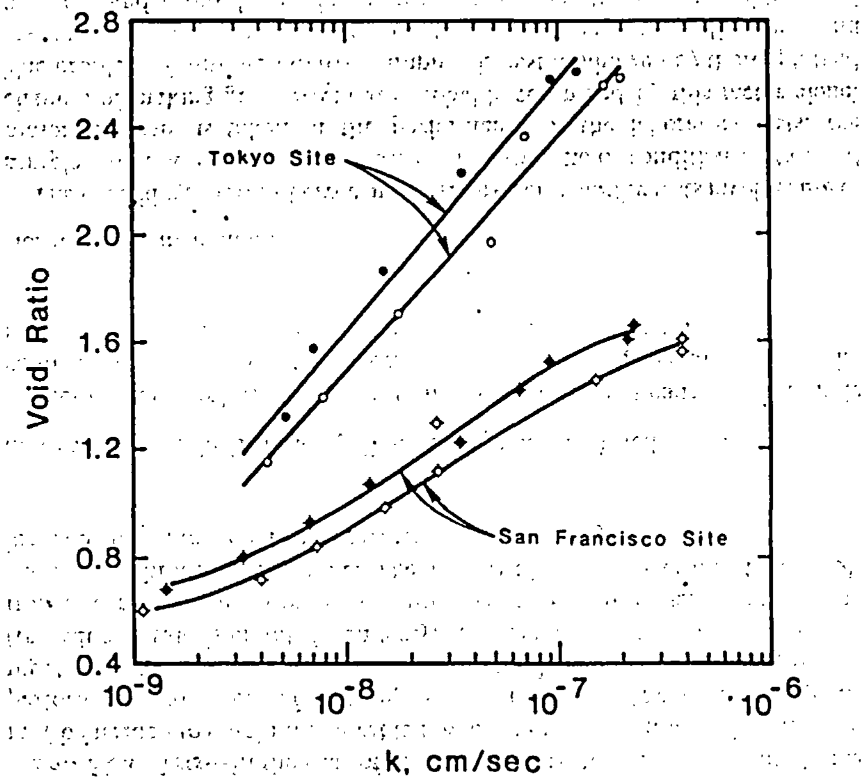

FIG. 4—*Typical comparisons of measured conductivity (open symbols) and conductivity computed from Terzaghi's theory (solid symbols).*

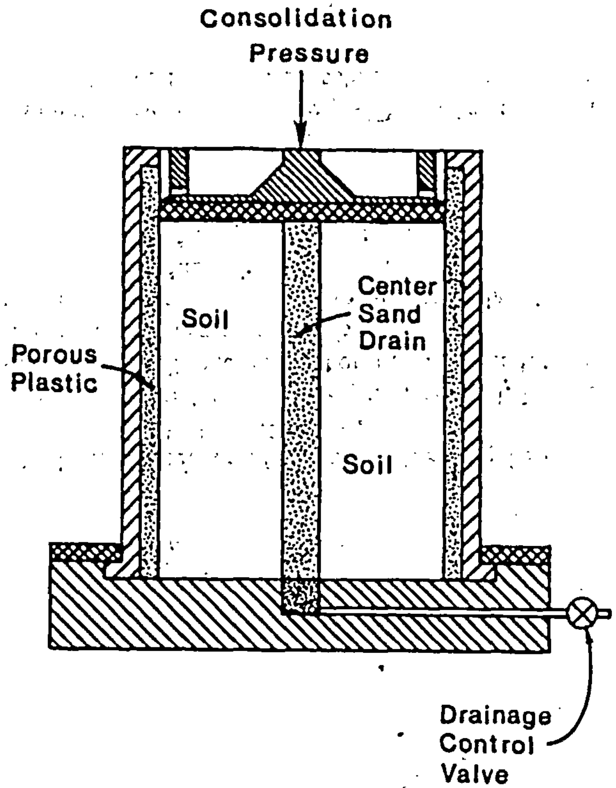

FIG. 5—*Radial flow permeameter.*

porous outer boundary (radius $= r_0$). For a sample height of L and a constant head of h on the sand drain, the conductivity is

$$k = \frac{Q}{2\pi Lht} \ln \frac{r_0}{r_w} \tag{7}$$

where Q is the volume of flow in a time period t. For a falling head test

$$k = \frac{a}{2\pi Lt} \ln \frac{h_1}{h_2} \ln \frac{r_0}{r_w} \tag{8}$$

In principle, radial flow tests can also be performed in the triaxial apparatus using a central sand drain and a continuous outer filter paper drain, but the permeability of the paper may not be high enough to provide free drainage except for fairly impervious clays [25,26]. Use of a triaxial cell offers the advan-

tages that no special apparatus is needed and that large enough samples may be used to include the effects of macrostructure, such as fissures.

Low Flow Tests—In the case of relatively impervious clays, substantial time may be needed to obtain measurable flows. A possible solution [*27,28,29*] is to perform the equivalent of a falling head permeability test by replacing the pipet with a compliant pressure transducer. The pressure on the upstream (transducer) side is elevated suddenly, thus deflecting the diaphragm of the transducer. The pressure is measured as water slowly leaks out of the transducer through the sample. The volume of flow is determined from calibrations of volume change versus transducer readings.

Sources of Error in Laboratory Tests Using Saturated Soils

"Sources of error" is here taken to mean all errors that cause the hydraulic conductivity measured in the laboratory to differ from the conductivity in the field. The term "saturated" is taken to mean nominally saturated.

Nonrepresentative Samples

The overriding source of error in laboratory permeability tests involves use of samples that are not representative of actual field conditions. Provided reasonable care is taken in the performance of the laboratory tests, the chances of making gross errors are probably controlled by this factor alone. The problem of unrepresentative sampling is best minimized by thorough field investigation, by attention to details (sampling along faults, fissures, sand partings, and so on), by prudent selection of samples for testing, and by use of large samples.

Laboratory Testing Errors

Voids Formed During Sample Preparation—For undisturbed samples, voids may be formed around the edges due to inadequate control of trimming, and fissures may open as a result of stress relief, thus leading to unrealistically high measured hydraulic conductivities. The first problem can be minimized by proper technique during trimming and the second by subjecting the samples to stresses approximating those in the field.

Smear Zones—If the sample contains such features as thin sand partings or root holes, the trimming operation may smear clay across the surface and tend to block entrance to these zones. Van Zelst [*30*] considered the effect of disturbance during trimming of the faces of specimens of clay for one-dimensional consolidation testing and concluded that each of the flat faces of his clay was remolded to a depth of about 0.2 cm (0.1 in.). Chan and Kenney [*31*] trimmed samples of varved clay using a vibro-tool moved parallel to the stratification

and found that the thickness of disturbed soil on each face was 0.06 cm (0.025 in.).

To minimize the effects of smear, (1) use a sharp knife for final trimming and cut the soil rather than trowel it; (2) include open root holes and other visible zones of higher conductivity in the specimen to be tested; and (3) use as large a specimen as possible.

Alterations in Clay Chemistry—A belief apparently held by some is that the permeability test should be performed using distilled water because such water is inert. Actually, leaching a sample with distilled water may cause expansion of the diffuse cloud of absorbed cations around clay particles and reduce hydraulic conductivity. Further, in some soils the leaching may increase particle mobility, either because of expansion of diffuse double layers or because of removal of cements, and lead to particle migration. An example of these effects is shown in Fig. 6 [*32*]. A solution is to use a permeant of the same chemistry as the original pore water but the time and expense involved in extracting, analyzing, and duplicating the pore water makes this solution impractical. Alternatively, samples of groundwater may be obtained in the field and used as a permeant. Another possible solution is to perform laboratory tests using very small amounts of flow and using sealed permeameters in which the flow can be reversed, thus cycling the same fluid to and fro [*19,33*].

Large changes in conductivity are likely to occur if a permeant is used with a chemistry that is widely different from that of the pore fluid. For example, Fireman [*34*] leached samples of Hesperia sandy loam with various aqueous solutions and found the following conductivities: 4×10^{-3} cm/s (originally), 6×10^{-3} cm/s (800 ppm calcium chloride), 3×10^{-3} cm/s (tap water), 1×10^{-4} cm/s (4500 ppm sodium chloride), and 2×10^{-5} cm/s (distilled water). Numerous studies have shown changes in conductivity for samples originally prepared with different chemistries [*35-38*].

When it is not feasible to determine the chemistry of the natural pore fluid and duplicate it as a permeant, agronomists have often used 0.01 *N* calcium sulfate as the permeant [*39,40*]. Some prefer to use tap water, which, though not ideal, generally seems a much better choice than distilled water.

Air in the Sample—In testing compacted samples, engineers often assume that soaking from the bottom, with the top open to the atmosphere, will lead to saturated samples. Smith and Browning [*41*] used this procedure to "saturate" 200 samples. They found that the average degree of saturation of their samples was 91 percent with the lowest value 78 percent. Because water cannot flow through an air bubble, the bubbles effectively reduce the void space that can be occupied by water and thus reduce hydraulic conductivity. Bjerrum and Huder [*42*] noted that air bubbles may tend to accumulate near the end of the sample where the water emerges, causing a clogging and erroneous measurements. Christianson [*43*] soaked a number of air-dried samples from the bottom up and then ran permeability tests. As flow continued

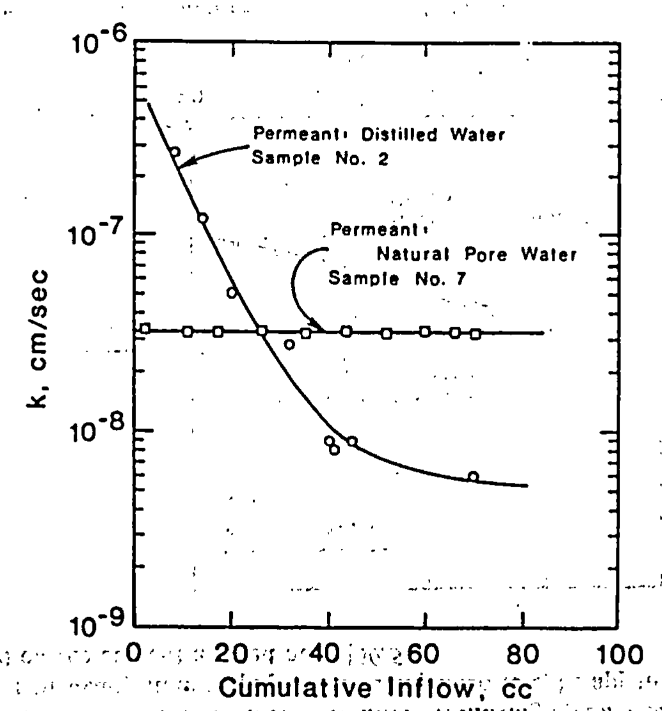

FIG. 6—*Influence of using distilled water (from Wilkinson [32]).*

the entrapped gas bubbles were slowly removed, and the measured hydraulic conductivities went up by factors ranging from 2 to 40 times. A typical range for many soils may be closer to 2 to 5 times [*44*]. When Christianson soaked his samples under vacuum he found no time-dependent increase in conductivity.

If water is forced through the soil using compressed air, the water entering the sample may contain a higher gas concentration than that corresponding to gas saturation at a lower pressure, and thus gas bubbles may form as the pressure in the flowing water decreases.

Growth of Microorganisms—Prolonged performance of permeability tests may result in a substantial reduction in hydraulic conductivity due to clogging of the flow channels by organic matter that grows in the soil during the test. Allison [*45*] reported on tests in which a variety of disinfectants were added to the permeating water to stop such organic growth. He found that phenol (1000 ppm) and formaldehyde (2000 ppm) were the most effective agents in delaying growth of organic matter, but eventually even these soils "sealed up." Finally, he sterilized samples of three soils, including one that was rich in organics and two that contained little organic matter, and used elaborate procedures to keep the permeating fluid sterile. In the sterile samples there was an insignifi-

cant decrease in k with time. However, the long-term conductivities of sterile soils were 8 to 50 times the values for soil that was unsterile or was originally sterile but had been allowed to regain organisms (Fig. 7).

The implications of these tests for field problems depends on the application of interest. For some problems, such as ponding of water on the surface of a site, microorganisms are just as likely to plug up the soil in the field as they are a laboratory specimen [45]. Hence, it might be best not to try to prevent growth of such organisms in laboratory tests. In other applications, growth of microorganisms in the field may be unlikely, in which case a disinfectant should be added to the permeant and testing times should be minimized.

Meniscus Problems in Capillary Tubes—In an effort to minimize other errors some investigators have used low hydraulic gradients and attempted to measure the outflow by observing the movement of the air-water interface in a capillary tube. Olsen [33,46] has demonstrated that significant errors can occur in the calculated pressure drop across the specimen because of essentially unavoidable contamination of the capillary tubes, which leads to an indeterminate, but nonzero, contact angle between the water and the glass. To eliminate this problem Olsen used a constant flow rate and measured the pressure drop.

Olsen [33] also pointed out that water flow rates should not be measured by observing the rate of movement of an occluded air bubble in a capillary tube because water can bypass the bubble.

Use of Excessive Hydraulic Gradients—In an effort to reduce testing time, large hydraulic gradients may be imposed on samples. If Darcy's law is valid, such gradients will not alter the measured conductivity. Schwartzendruber [47] surveyed the then-existing literature and found many experiments in which k, defined as in Eq 1, increased as the gradient increased, with ratios of the maximum to minimum measured k typically between 1 and 5 but with one value of 84. Other studies, such as those by Mitchell and Younger [48] and Gairon and Schwartzendruber [49], found decreasing values of k as the gradient was increased, apparently as a result of particle migration, causing clogging. It seems desirable to use gradients as close to those encountered in the field as is economically feasible.

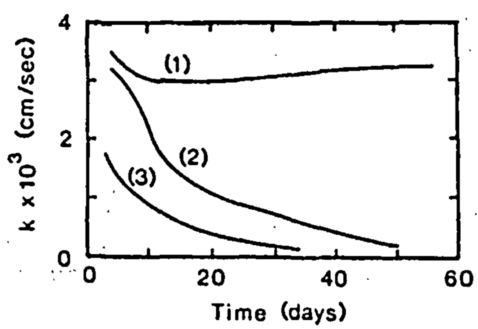

FIG. 7—*Time dependence of conductivity of (1) sterile soil and water, (2) initially sterile soil permeated with unsterile water, and (3) unsterile soil and water (from Allison [45]).*

Temperature—On occasion, engineers correct the measured conductivity to a standard temperature by adjusting for the effect of temperature on the viscosity and density of pure water (Ref 7, page 113, and Ref 12, page 592). We have measured the effect of temperature on conductivity of three fine-grained soils (Fig. 8) and found that simple viscosity and density adjustments are generally adequate for taking into account the effects of temperature. Note that conductivity is not particularly sensitive to small or moderate changes in temperature when water is used as the permeant; the viscosity of water decreases approximately 3 percent per degree Celsius rise in temperature from 21°C. However, the conductivity of fine-grained soils is probably influenced by complex interaction between the water, adsorbed and free ions, and the mineral surfaces. Consequently, it is a good idea to perform permeability tests at approximately the relevant temperature when the results are to be applied to the solution of a problem in the field.

Volume Change Due to Stress Change—If a change in pore pressure is imposed on a sample under a constant total stress, the resulting change in effective stress must result in a change in volume of the sample. Thus, in a constant head test some of the initial measured inflow is making up for volume change rather than steady-state seepage. In a falling head test the apparent k would depend on the current applied head [50,51].

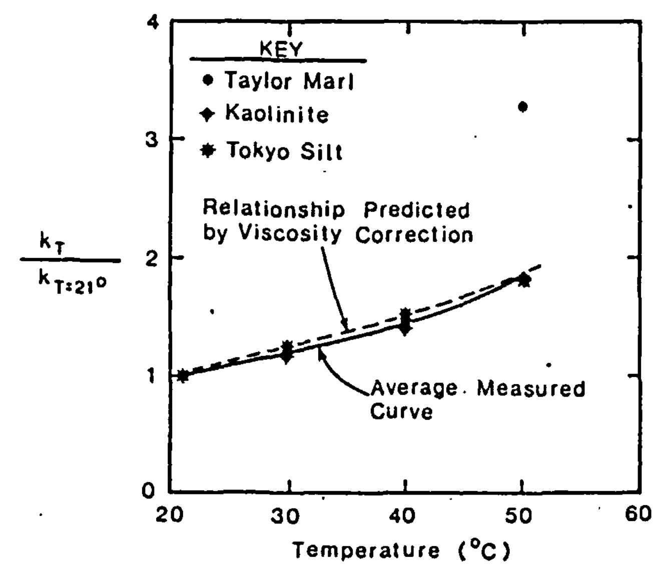

FIG. 8—*Effect of temperature on conductivity. Conductivities at temperature t (k_T) are normalized with respect to the measured conductivities at 25°C.*

Al-Dhahir and Tan [52] have presented a solution for the consolidation (or swelling) of a sample subject to a constant total stress and an instantaneous change in pore pressure at one boundary. They suggest that a plot be prepared of the flow rate at the boundary where the pore pressure is changed, q, versus $t^{-1/2}$. The relationship should have a sloping portion, representing the time period in which the soil is undergoing volume change, followed by a leveling off at small values of $t^{-1/2}$, where volume change ceases and steady-state seepage occurs (Fig. 9).

Flow Direction—It is nearly always easier to perform laboratory permeability tests with the soil in the same orientation as in the field and with the flow vertical. However, sometimes the horizontal conductivity, k_h, is larger than the vertical value, k_v, which usually leads to predominantly horizontal flow in the field. Data published on the ratio k_h/k_v are summarized in Table 1 [31,53-60]. For varved or stratified clays, the ratio may exceed 10, whereas for less stratified soils, the ratio is likely to be closer to 1. For soils containing root holes, k_v may exceed k_h. Clearly, laboratory specimens should be oriented to produce flow in the direction that will dominate in the field.

Laboratory Tests for Partially Saturated Soils

Methods available for laboratory measurement of the conductivity of partially saturated soils are similar to those used with saturated soils, and the problems are similar but more severe. Two of the problems require immediate discussion because they influence testing procedures strongly; these are (1) measurement of pore water pressure and (2) the effect of the degree of saturation. We will examine these problems first, and then consider testing methods and errors as for saturated soils.

Measurement of Pore Water Pressures

Pore water pressures in partially saturated soils are negative compared with the pore air. To avoid use of negative numbers we will use the term "suction" as the negative of the pore water pressure. Techniques for measurement of suction have been reviewed in the engineering literature by Croney and Coleman [61] and in this volume by Daniel, Hamilton, and Olson [62]. References to agricultural literature will be included in the following discussion.

Only a few of the methods discussed in the literature can be used when conductivities are to be measured. Typical problems with other methods include slow response, inadequate sensitivity, and instability. The useful apparatuses include tensiometers, pressure plates, and psychrometers.

Tensiometers—A tensiometer consists of a porous sensing element connected to a pressure measuring device by a tube. The sensing element is typically a ceramic probe. The pores in the probe must be fine enough to prevent air from blowing through the stone and draining the measuring system.

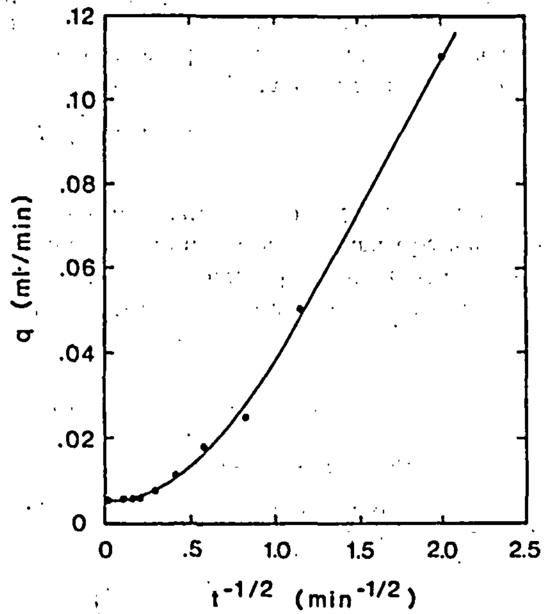

FIG. 9—*Time dependency of rate of water flow for remolded clay tested in consolidation cell permeameter.*

TABLE 1—*Data on the ratio of horizontal to vertical conductivity of fine-grained soils.*

Reference	k_h/k_v	Notes
Subbaraju et al [53]	1.05	soft marine clay, $w = L_w$, L_w = 65 to 90%, I_w = 24 to 55%
Lumb and Holt [54]	1.2	highly plastic marine clay
Bazett and Brodie [55]	1.5	soft clay, L_w = 60 to 80%, I_w = 35 to 50%
Tsien [56]	1.2 to 1.7	organic silt with peat, w = 191 to 570%
Chan and Kenney [31]	1.5 to 3.7	varved clay, laboratory tests
Kenney and Chan [57]	1.5	varved clay, field tests
Haley and Aldrich [58]	0.7 to 3.3	Boston blue clay, w = 40 to 45%
Wu et al [59]	3 to 15	varved clay, w = 20 to 30%, L_w = 25 to 35%, P_w = 8 to 20%
Casagrande and Poulos [60]	4 to 40	varved clay, w = 45 to 75%, L_w = 50 to 80%

However, tensiometers are generally limited to suctions less than about 0.9 atm [63] because larger suctions lead to nucleation of air bubbles in the measuring system ("cavitation"), and the immediate expansion of these bubbles reduces the suction in the measuring system essentially to zero. A typical tensiometer is shown in Fig. 10.

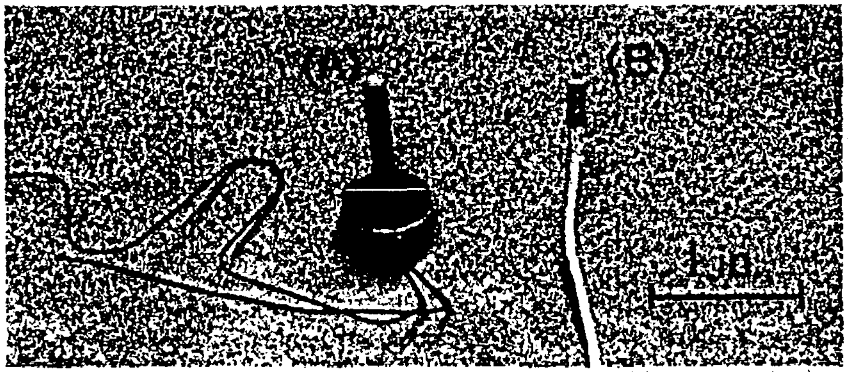

FIG. 10—*Photograph of (a) tensiometer in brass housing and (b) thermocouple psychrometer.*

Pressure Plates—When used to measure suctions, the pressure plate device [*61,64*] may be the same as a tensiometer except that cavitation of the measuring system is prevented by superimposing an air pressure within the soil sample until the pressure in the measuring system is near zero. The suction is defined as the pore air pressure minus the pore water pressure, and is unaffected by increasing the pore air pressure. The method may also be called the "axis translation" method [*65*]. The probe may be made of ceramic for suctions up to about 15 atm. For higher suctions a Visking membrane may be placed over a ceramic probe [*64,66,67*]. Although air may not blow through the fine probe, it may pass through in solution and reform as air bubbles in the measuring system.

Psychrometers—Suctions up to about 80 atm can be measured by determining the relative humidity, H, of the pore air using a psychrometer [*62,68,69*] and calculating the suction, p, using

$$p = \frac{RT}{M} \ln H \tag{9}$$

where R is the gas constant, T is temperature, M is the molecular weight of water, and H is expressed as a ratio. Psychrometers are inaccurate for suctions lower than 1 or 2 atm because of the difficulty of measuring relative humidities near 100 percent. A photograph of a typical psychrometer is included in Fig. 10.

Effect of Degree of Saturation

Measurements show that (1) the degree of saturation decreases as the suction increases (Fig. 11*a*), and (2) the conductivity decreases rapidly as the degree of saturation decreases (Fig. 11*b*). The imposition of a hydraulic gra-

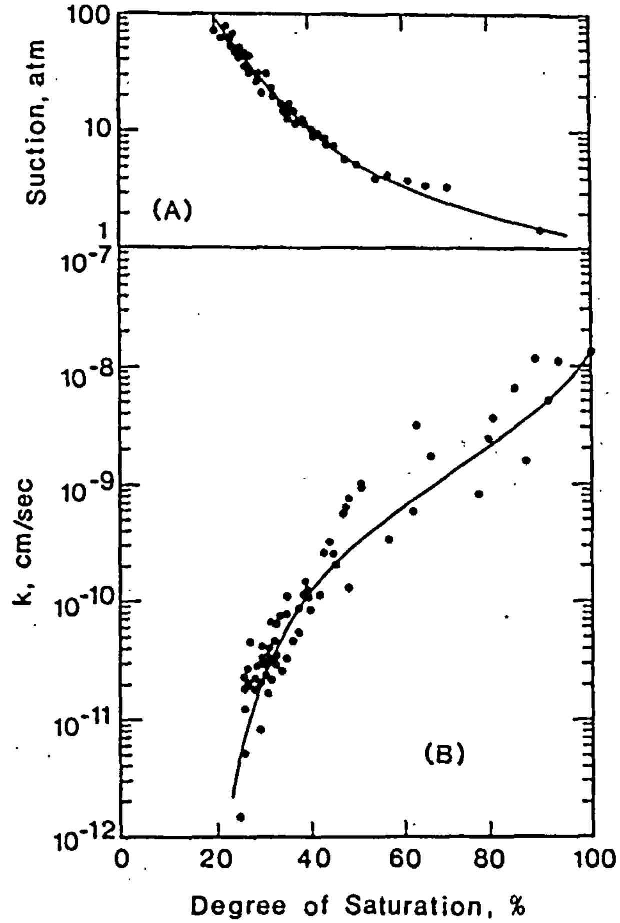

FIG. 11—*Suction and hydraulic conductivity versus degree of saturation for compacted fire clay.*

dient on a sample leads to spatial variations in suction, degree of saturation, and thus conductivity. Therefore, either tests must be performed using small gradients, or water flow-gradients-suctions must be measured simultaneously at a point, or some other method used to resolve this problem.

Measurement of Conductivity

Steady-State Methods—Steady-state methods are similar to those used for saturated soil except the head is negative and is controlled at both ends of the sample [*70*]. For fine-grained soils, the sample is typically cylindrical, with the diameter of the order of 25 to 100 mm and the length 50 to 500 mm, with a

horizontal axis and flow direction. The change in elevation head is typically negligible and Darcy's law is written

$$q = \frac{k}{\gamma_w} \frac{dp}{dx} A \tag{10}$$

where p is suction, x is flow distance, and other variables have been defined previously.

The pore air is typically vented to the atmosphere, and suctions at the two ends are maintained at values between about 0 and 0.9 atm using porous stones and manometers (Fig. 12 [*71*]). To avoid end effects the suction is typically measured at two or more points along the length of the sample. For suctions greater than about 0.9 atm, the pore air pressure may be raised to develop suctions up to about 15 atm. The test may be repeated at different suctions to yield a relationship between conductivity and suction.

If a relationship between conductivity and water content is desired, then (1) tests may be performed on replicate samples with the water content measured destructively after each test; (2) the tests may be performed by using a single sample, with the water contents measured nondestructively using neutron backscattering techniques, or by weighing the entire sample and apparatus and obtaining the dry weight at the conclusion of the test; or (3) the relationship between water content and suction may be measured on a separate sample. The relationship between water content and suction may have hysteresis, so measurements are taken by either wetting or drying a sample through the range of suctions of interest.

Several variations of these testing procedures have been used. In one [*72*] a pore water pressure gradient is applied in one direction and an air pressure gradient of the same amount in the opposite direction, leading to flow with a constant suction, thus constant water content. In another [*73*], water is introduced at a constant suction at one end and is removed at a constant rate of evaporation at the exit end.

Instantaneous Profile Method—In this method [*74-76*] a long cylindrical sample of soil, typically with a horizontal axis, is provided with a number of suction probes arranged along the length of the tube (Fig. 13). The soil is initially in hydraulic equilibrium, and then the hydraulic conditions at one end ("near end") are altered. The alteration may be the imposition of a constant or time-dependent suction, either above (outflow) or below (inflow) the suction in the soil, or a constant or time-dependent imposed inflow. Non-steady-state seepage develops within the sample.

The volume of water, V_w, between any probe and the "far end" (away from the end where hydraulic conditions were altered) is

$$V_{wi} = \int_{x_i}^{L} \theta A \, dx \tag{11}$$

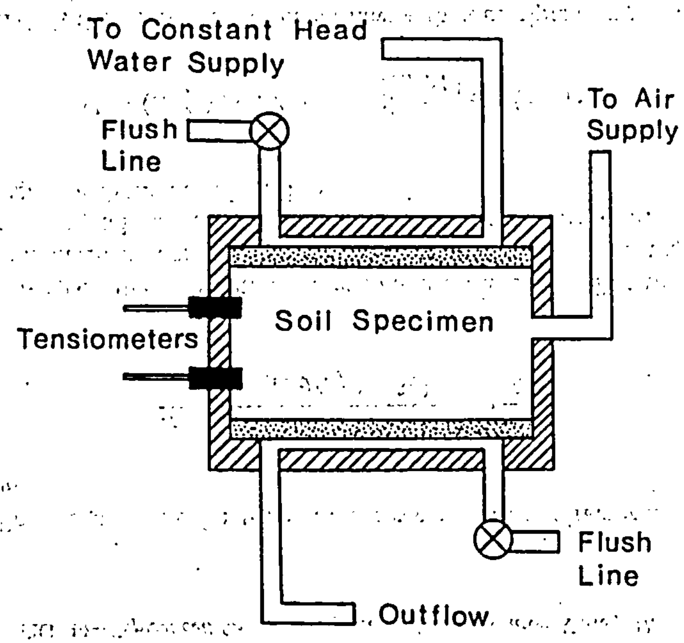

FIG. 12—*Cell for steady-state method of measurement of hydraulic conductivity of unsaturated soil (after Klute [71]).*

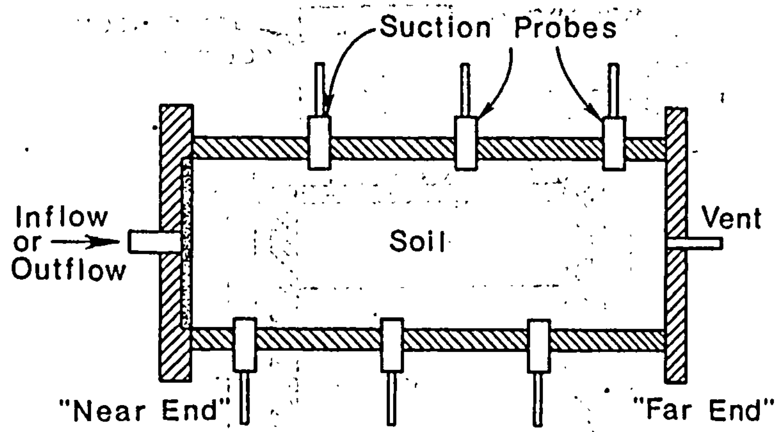

FIG. 13—*Apparatus for laboratory measurements of hydraulic conductivity of unsaturated soils with the instantaneous profile method.*

where x_i is the x coordinate of any probe, "i"; L is the length of the sample; θ is the volumetric water content (ratio of the volume of pore water to the total volume); and A is the total cross-sectional area. The change in V_{wi} in relation to time, dV_{wi}/dt, is the flow rate, q_i (Eq 1). The volumetric water

contents (Eq 11) are obtained from the measured suctions, p_i, and curves of θ versus p, either by using a numerical integration scheme or by fitting an analytic function to the θ-x curve and integrating the function. The average hydraulic gradient at the probe, i, can be approximated as

$$(dp/dx)_i = (p_{i+1} - p_{i-1})/2\Delta x \tag{12}$$

where p_{i+1} and p_{i-1} are the measured suctions at adjacent probes, and Δx is the spacing of the probes. Again, higher order finite difference equations can be used or a fitted analytic function can be differentiated. The conductivity at the probe, k_i, is then calculated from

$$k_i = \left(\frac{dV_{wi}}{dt}\right)\left(\frac{\gamma_w}{A}\right)\bigg/\left(\frac{dp}{dx}\right)_i \tag{13}$$

The conductivity may be calculated for each time step, Δt, for each node past which there has been a measurable flow, thus leading to a large number of observations which can be plotted against water content, suction, or degree of saturation.

In using this method it is essential to have an accurate θ-p relationship. The most satisfactory procedure is to introduce, or remove, moisture at a slow pace so the wetting or drying front is spread out. Just before the leading edge of the front reaches the far end of the sample, the test is stopped and water contents and suctions are measured at each probe, thus yielding a θ-p relationship for the sample tested. Measurement of the θ-p relationship on a replicate sample set up solely for this purpose may provide useful supplementary data.

A steady-rate-of-inflow test, starting with a nearly dry sample and continuing until the soil near the entrance end is nearly saturated and the wetting front has reached the far end, requires about 2 weeks.

Pressure-Plate Outflow Test—In this test, a sample of soil is placed on a saturated, fine, porous plate in a pressure vessel (Fig. 14). An appropriate air pressure is applied, the water pressure in the plate is maintained at atmospheric pressure, and the sample is given time to come to equilibrium. Then the air pressure in the vessel is suddenly increased (or decreased), thus generating a uniform excess pore water pressure in the sample. The excess water drains out through the porous plate. The rate of outflow from an element of area, dA, and height, dz, is

$$\frac{\partial(dV_w)}{\partial t} = \left(\frac{\partial q}{\partial z}\right) dz \tag{14}$$

Substitute

$$\frac{\partial(dV_w)}{\partial t} = \frac{\partial(dV_w)}{\partial p}\,\frac{dp}{dt} \tag{15}$$

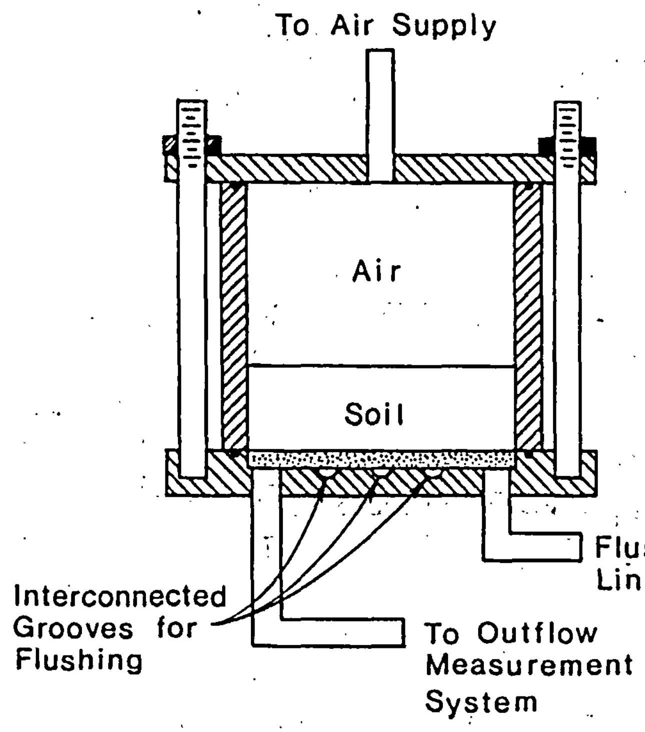

FIG. 14—*Typical cell for pressure plate outflow tests (after Klute [71]).*

Insert Eqs 9 and 15 into Eq 14, assume k is constant, and factor the resulting equation to

$$\frac{\partial p}{\partial t} = \frac{k}{\gamma_w(d\theta/dp)}\,\frac{\partial^2 p}{\partial z^2} = D\frac{\partial^2 p}{\partial z^2} \tag{16}$$

where θ is again the volumetric water content, and D is called the diffusivity. Equation 16 is identical to the differential equation governing one-dimensional consolidation (Ref 7, page 228) and has the same solution. The average degree of drainage, U, is thus

$$U = (\Delta V_w)_t/(\Delta V_w)_u = \sum_{m=0,1}^{\infty} \frac{1}{M^2}\exp(-M^2T) \tag{17}$$

where ΔV_w is the volume of water outflow with subscripts t and u indicating values at times t and ultimately, and

$$M = \frac{\pi}{2}(2m + 1) \tag{18}$$

and

$$T = Dt/L^2 \tag{19}$$

where T is the time factor, t is elapsed time, and L is the height of the sample. The diffusivity can be calculated using a log t or $\sqrt{t}$-fitting methods [77], or by methods recommended in the agricultural literature [77].

Sources of Error in Laboratory Tests Using Partially Saturated Soils

The sources of error involved in laboratory testing of partially saturated soils are similar to those previously discussed for saturated soils, but considerably less information is available. Errors involved with use of nonrepresentative samples, smear zones, and incorrect flow directions need no further comment. Growth of microorganisms again results in difficulties, and use of a 0.1 percent phenol solution [78] or solutions of mercuric chloride, thymol [40], or formaldehyde is recommended to minimize biological activity. Several sources of error deserve special mention, either because of availability of data or because of their unique nature in testing partially saturated soils.

Chemical Effects

If the suction in a sample is constant, but there exists a variation in electrolyte concentration, the pore water will flow in the direction of increasing electrolyte concentration, and the electrolyte will diffuse in the opposite direction until equilibrium is finally established. Thus, permeation of a sample with a fluid of different electrolyte concentration will lead to diffusive flow of water in addition to flow induced by variation in suction. When the conductivity is high the flow induced by electrolyte gradients is probably too small to be of much interest, but as k drops, the importance of diffusive flow may increase. Letey et al [78] wrote Darcy's law in a form similar to

$$q = \frac{k}{\gamma_w}\frac{dp}{dx} + \frac{k_\pi}{\gamma_w}\frac{d\pi}{dx} \tag{20}$$

where π is the osmotic pressure, and k_π is the osmotic conductivity. Measurements of k_π were obtained by maintaining a constant difference between the electrolyte concentration across a sample of partially saturated soil. The ratio of k_π/k increased from essentially zero for nearly saturated soil to about 0.16 at a suction of about 0.66 atm. No data exist for higher suctions.

Temperature Effects

What little evidence that exists of temperature effects [79] indicates that an increase in temperature may reduce the thickness of water films at constant suction, thus decreasing the conductivity, but also reduces the viscosity of the water, thus increasing conductivity. The net result is that temperature changes of the order of ten degrees Celsius cause changes in the p-k relationship that are smaller than experimental scatter.

Filter Impedance

In steady-state and instantaneous profile measurements, tensiometers or psychrometers are placed at various locations along the length of a sample to eliminate end effects. In the pressure plate outflow test, however, a nonfreely draining porous stone may retard flow and lead to significant errors. Attempts to account for filter impedance [80] have produced a tedious method which, however, does not require knowledge of the filter impedance.

An alternative approach is to derive a Fourier series solution for the one-dimensional non-steady-state flow problem with an impervious upper boundary and a non-freely draining lower boundary. The solution is

$$U = 1 - \sum_{n=1}^{\infty} C_n \sin^2(r_n)\exp(-r_n^2 T) \tag{21}$$

where

$$U = Q_t/Q_u \tag{22}$$

and Q_t and Q_u are the volumes of outflow of water at times t and ultimately, respectively. Further, r_n represents successive roots of the equations

$$Rr_n \tan r_n - 1 = 0 \tag{23}$$

in which R is termed the impedance ratio and is defined as

$$R = (k/k_d)(L_d/L) \tag{24}$$

where k_d and k are the conductivities of the porous stone and soil, respectively, and L_d and L are the thickness of the porous stone and soil, respectively. Also

$$C_n = 2(R^2r_n^2 + 1)/(r_n^2)(R^2r_n^2 + R + 1) \tag{25}$$

and

$$T = Dt/L^2 \tag{26}$$

where T is dimensionless time (conveniently termed the "time factor"), and D is the diffusivity, given by

$$D = k/m\gamma_w \tag{27}$$

where

$$m = d\theta/dp \tag{28}$$

and θ and p are the volumetric water content and suction, respectively. Equation 21 is solved for the U-T relationship for suitable range in values of R (Fig. 15). For a particular test, Q_t is plotted versus log t and the time, t_{50}, corresponding to half of the ultimate flow (U = 50 percent) is read from the curve. From Eqs 26 to 28

$$k = \frac{T_{50}\gamma_w m L^2}{t_{50}} \tag{29}$$

The solution proceeds as follows:

1. Determine the hydraulic conductivity, k_d, and length, L_d, of the filter by direct measurement before the test begins.
2. Determine the length of the soil sample, L, and the slope of θ-p curve, m, from measurements for this particular increment of pressure.
3. Assume a trial value of k. Use the value from the last pressure increment if you are on the second or greater pressure step.
4. Calculate the impedance ratio, R (Eq 24).

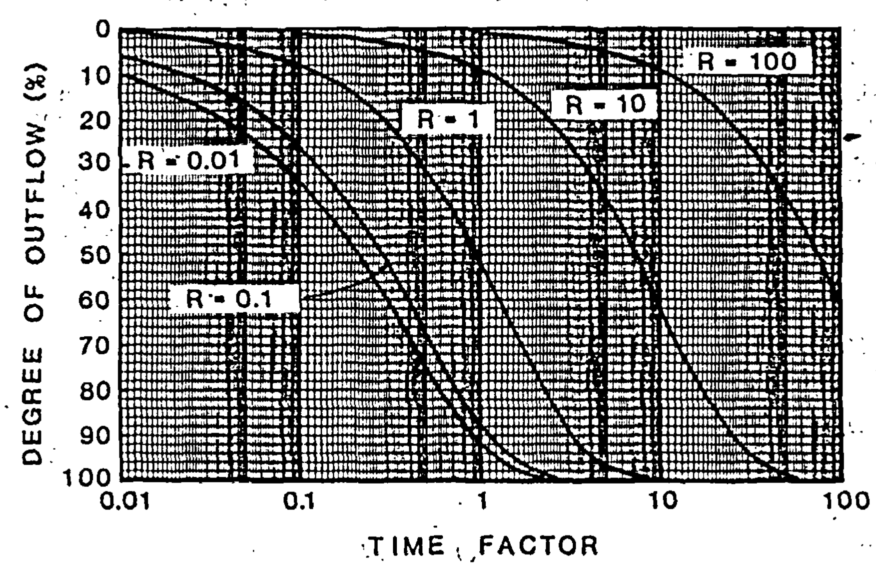

FIG. 15—*Theoretical curves of degree of outflow versus time factor for a pressure plate outflow test [illegible]npeded drainage.*

5. Read T_{50} from Fig. 16.
6. Calculate k using Eq 29.
7. If the new k is the same as the previously assumed value, quit. If not, use the new value and return to Step 4.

The accuracy of this approach hinges on knowing the characteristics of the filter, ensuring essentially perfect contact between the soil and the filter, and measuring Q accurately.

One-Step Pressure Plate Outflow Tests

Based on an analysis by Gardner [*81*], Doering [*82*] suggested that the pressure plate outflow test could be modified by using only a single large step of air pressure. However, the basis for calculation of k includes so many erroneous assumptions (such as constant properties and no filter impedance) that this method seems of little value and should not be used.

Variable Properties in Incremental Outflow Method

To avoid some of the problems associated with the one-step outflow method, outflow tests are usually performed with small steps in the applied pressure.

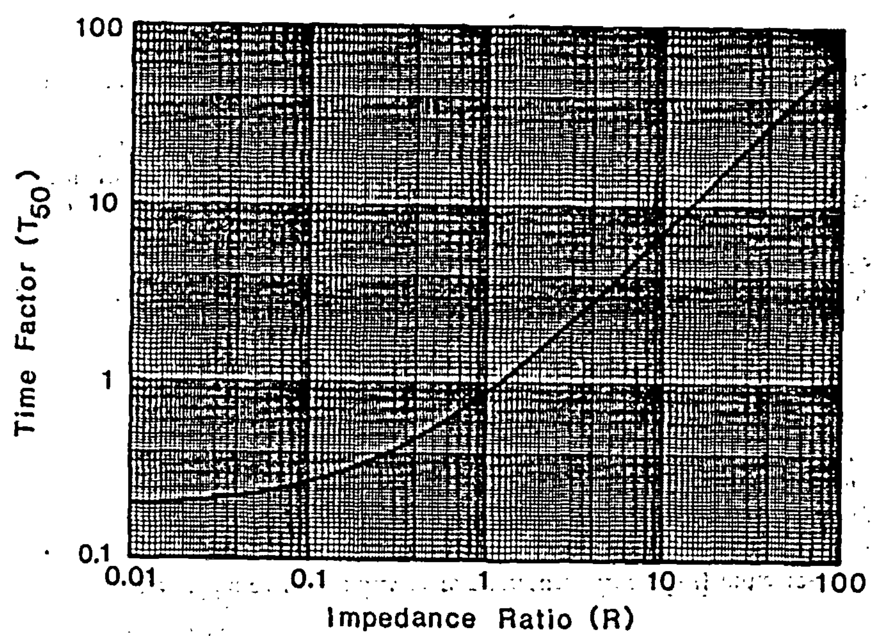

FIG. 16—*Influence of boundary impedance on the time factor at which 50 percent of the outflow has occurred.*

However, the increments of pressure must be large enough to produce measurable outflow. As a result, substantial changes in water content, and attendant changes in k and D, usually occur in each increment. Hence, interpretation of the data is often difficult and reproducibility of results is less than ideal [83,84].

Evolution of Gas in Pressure Plate Outflow Test

Pressure plate experiments performed at suctions greater than 1 atm require superposition of an air pressure on the soil to prevent cavitation of water in the measuring system. A major problem exists with evolution of air bubbles in outflow tests and the errors that these bubbles tend to cause in the measurement of outflow quantities [71]. To minimize this problem, a trap [71] or pump and trap [20] may be used to remove the bubbles.

Validity of Darcy's Law

Data on the effect of the hydraulic gradient on flow rates are much more meager for partially saturated soils than for saturated soils. Schwartzendruber [85] used data collected by Rawlins and Gardner [86] and concluded that, for the one soil tested, Darcy's law was valid for $35 \leq \theta \leq 55$ percent but flow rates increased more than proportionally to gradient for $15 \leq \theta \leq 35$ percent. Similar non-Darcy behavior was reported by Schwartzendruber [47] but Weeks and Richards [75] found Darcian behavior (without presenting diagnostic diagrams), and Olson and Schwartzendruber [87] presented definitive data showing the validity of Darcy's law for narrow ranges in degree of saturation (80 to 89 percent, 73 to 87 percent, 66 to 89 percent, 66 to 83 percent) for four soils of rather low plasticity. Hamilton, Daniel, and Olson [76] report measurements of hydraulic conductivity on a clay compacted over a range in saturation of 25 to 95 percent; the data do not suggest any tendency for k to vary with hydraulic gradient.

The existing evidence thus suggests that Darcy's law is a useful approximation for the q–i relationship in partially saturated soils but is probably not valid in all cases, thus leading to the conclusion that gradients used in measuring hydraulic conductivity should be as close to those encountered in the field as feasible.

Field Measurements of the Hydraulic Conductivity of Saturated Soils

The hydraulic conductivity is generally measured in the field by drilling a hole in the ground, measuring the rate of flow of water into or out of the hole, and using an appropriate formula to calculate the conductivity. Tests may be performed at constant head, generally by establishing a high head of water in the borehole and pumping at a rate sufficient to maintain this head, or with a variable head, that is, with the head set at a nonequilibrium value initially and then measured as a function of time with no further pumping. Additional variables to be included in the equations for k must account for the presence or absence of casing, the location of the bottom of the casing in relation to the bottom of the boring, the shape of a piezometer tip if one is used, anisotropy in the soil, soil compressibility, presence of impervious surfaces near the tip, amount of air in the soil, secondary effects, and, doubtless other effects as well. Analysis shows that rigorous solutions can be obtained in only a few cases with unrealistic soil properties or test geometries. Solutions for more realistic conditions have generally been one of the following three types: (1) replacing the actual geometry with a simpler one and obtaining an approximate analytical solution; (2) using a three-dimensional electrical model; and (3) using a numerical method, such as finite differences and a digital computer. The inevitable result of having numerous special cases, complex geometry, and approximate solutions, is a rather complicated, and sometimes contradictory, literature. We will review a few of the methods that seem to be in most general use.

Testing Methods

Auger Method—In principle the simplest field test is performed by drilling a hole, without the use of casing, and then performing either a constant head or variable head test, using either inflow or outflow. The method is termed the *auger method* by agronomists [88]. In its usual form, the method involves boring a hole to beneath the water table, pumping the water level down several times to flush out the voids in the soil, and then pumping the hole down again and measuring the water level in the hole as a function of time. The equation for k was derived by Kirkham and van Bavel [89] and applications have been discussed by van Bavel and Kirkham [90] and Kirkham [91]. The relevant equation is

$$k = \frac{\pi^2}{16} \frac{r}{Sd} \frac{\Delta h}{\Delta t} \tag{30}$$

where k is the conductivity (L/T), r is the radius of the well (L), S is a shape factor (dimensionless), d is the depth of the bottom of the hole below the water table (L), h is the height of water in the hole (L), and t is the time elapsed since the cessation of pumping (T). Values for the shape factor are shown in Fig. 17 (Ref 92, page 141). The solution applies only for an incompressible soil, a hole drilled down to an impervious base, and no drawdown of the water table (keep h/d less than 0.2). To simplify analysis, most users of the auger method appear to assume the presence of an impervious base. Boersma (Ref 99, pages 223–229) has presented shape factors for an impervious base below the bottom of the hole but only for a range in values of d/r from 6 to 14.

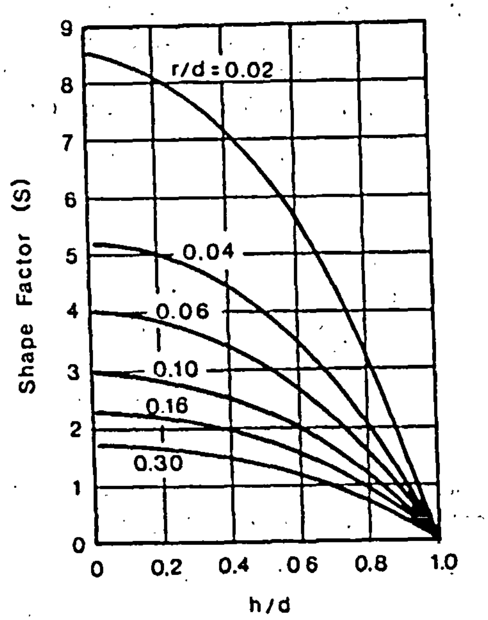

FIG. 17—*Shape factors for use with the auger method [92].*

The auger method is generally used only near the water table because of a tendency of the soil to fail by piping or sloughing. Further, the test can be used only in moderately pervious soils because of the slow rate at which the water level rises for less pervious soils. For example, if a 10-cm (4-in.) diameter hole is drilled to a depth of 1.8 m (6 ft) below the water table, the time needed for the water to rise from a depth of 1.2 m (4 ft) to 0.9 m (3 ft) for a soil with $k = 1 \times 10^{-6}$ cm/s is about 30 h.

Testing Using Cased Holes—For applications in geotechnical engineering, it is common practice to case the soil either to prevent sloughing or to isolate the flow to a single layer. Equations have been derived in a number of forms. For a constant head test a common form is

$$k = q/FDh \tag{31}$$

where q is the flow rate (L^3/T), F is a shape factor (dimensionless), D is hole diameter (L), and h is the head loss (L). For cases in which the bottom of the borehole is beneath the water table, h is the difference between the elevations of the water in the borehole (the equivalent elevation is used if the water has been pressurized) and the water table. If the base of the borehole is above the water table, then h is often taken as the depth of water in the borehole. For falling head or rising head tests a common form for the equation is

$$k = \frac{A}{FDt} \ln \frac{h_1}{h_2} \tag{32}$$

where A is the area of the standpipe, and t is the time for the head to change from h_1 to h_2.

For incompressible, homogeneous, isotropic soils, Hvorslev [*50*] tabulated the shape factors shown in Table 2 [*7,50,93-96*]. The factors for Cases 4 through 6 are approximate. Case 6 is the one of major interest to geotechnical engineers.

Testing Using Porous Probes—Field permeability tests are probably most easily performed by sealing a more-or-less cylindrical cavity at an appropriate depth with one or two tubes extending to the surface [*97*]. The cylinder may be formed by drilling a borehole and sealing a well point or porous stone in a sand-filled lower cylindrical portion [*97*] or by forcing into place a more-or-less cylindrical probe [*98-100*]. The borehole is often sealed just above the probe with bentonite, grout, or some other reasonably impervious material. Theoretical work by Vaughan [*101*] (Fig. 5) suggests that the sealing material can be up to 100 times more pervious than the soil into which the probe is inserted without overestimating field conductivity by a factor greater than 1.3 to 1.9. The self-boring pressuremeter also has application in this area [*102*].

TABLE 2—*Shape factors of piezometer tips infield permeability testing.*

Case	F[a]	Condition	Source
1	2π	spherical tip in an infinite soil	Samsioe [*93*] Dachler [*94*]
2	π	hemispherical tip extending below an impervious upper boundary	Samsioe [*93*] Dachler [*94*]
3	2	borehole with a flat bottom at an upper impervious boundary	Forchheimer [*95*] Dachler [*94*]
4	2.75	cased borehole with a flat bottom in the middle of a deep soil layer	Hvorslev [*50*] Harza [*96*] Taylor [*7*]
5	$\dfrac{2\pi(L/D)}{\ln\left(2L/D + \sqrt{1 + \left(\dfrac{2L}{D}\right)^2}\right)}$	borehole with a flat bottom extending a distance, L, below an impervious upper boundary, no casing	Dachler [*94*]
6	$\dfrac{2\pi(L/D)}{\ln\left(L/D + \sqrt{1 + \left(\dfrac{L}{D}\right)^2}\right)}$	cased hole in a semi-infinite soil with an uncased section of length, L, below the casing	Dachler [*94*]

[a] Shape factor.

As a result of the wide use of cylindrical tips there have been a number of studies to evaluate the shape factors. Numerical values of these shape factors are shown in Fig. 18 [*10,50,103-106*]. The factors presented by Smiles and Youngs [*106*] and Al-Dhahir and Morgenstern [*104*] seem to have the best theoretical base and to represent the most reliable values at present.

A number of special conditions may need to be evaluated. Several of these are discussed in the following section.

Special Conditions and Testing Errors

Anisotropy—For cross-anisotropic soils, where the vertical and horizontal conductivities are k_v and k_h, respectively, the actual soil may be replaced by an equivalent isotropic soil of conductivity, k_m, where

$$k_m = \sqrt{k_v k_h} \tag{33}$$

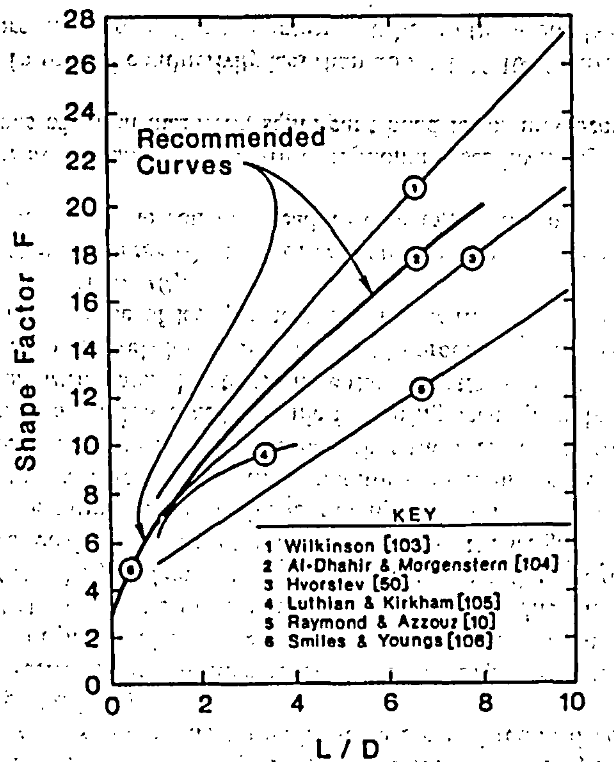

FIG. 18—*Shape factors used by various investigators.*

The transformation is performed by multiplying all horizontal dimensions by $\sqrt{k_v/k_h}$ [*24,93,94*]. For Cases 5 and 6 in Table 1, Hvorslev [*50*] shows that the equations for isotropic soils can be used for anisotropic soils, provided that the terms *L/D* are replaced by *mL/D* where

$$m = \sqrt{k_h/k_v} \tag{34}$$

Values for m can be estimated by performing field permeability tests using probes with differing *L/D* ratios [*6,10*].

Compressible Soil—If the soil is compressible, then changes in water pressure in the probe cause swelling or consolidation in the surrounding soil, and part of the water entering or leaving the probe results from volume change in the soil rather than exclusively from steady-state seepage. The resulting problems of interpretation lead to the conclusion that field tests should preferably be of the constant head type. Gibson [*107*] originally analyzed this problem for a spherical probe in a semi-infinite soil. He assumed that the change in pressure in the probe simply altered the pore pressure at the probe-soil interface (constant total stress). He also assumed validity of a consolidation equation of the heat flow type. The resulting analysis yielded the solution

$$q = 4\pi a \frac{k}{\gamma_w}\left(1 + \frac{1}{\sqrt{\pi T}}\right)\Delta u \tag{35}$$

where q is the flow rate (L^3/T), a is the radius of the spherical probe (L), k is conductivity (L/T), γ_w is the unit weight of water [force/volume (F/L^3)], Δu is the changed pore pressure in the probe [force/area (F/L^2)], and T is the time factor (dimensionless) given by

$$T = ct/a^2 \tag{36}$$

where c is the coefficient of consolidation, and t is time. A plot should be made of q versus $t^{-1/2}$, and the intercept at $t^{-1/2} = 0$ defines q_u. The permeability is then given by

$$k = q_u\gamma_w/4\pi a\Delta u \tag{37}$$

The variation of q with time, such as in Fig. 19 [*108*], indicates possible errors associated with ignoring the compressibility of the soil. Subsequently, Gibson [*109*] modified his solution to account for the fact that a change in water pressure in the probe may also change the total stresses in the soil. The shape and slope of the q versus $t^{-1/2}$ curve is altered, but the intercept remains unaffected.

Head Losses in Probe and Surrounding Zone of Incompressible Material—Gibson [*110*] has also analyzed the case of a spherical probe of finite conductivity and a further thickness of some other incompressible material of finite conductivity (sand, disturbed soil). In this case, the slope, shape, and in-

tercept of the q versus $t^{-1/2}$ curve are all affected. The solutions are probably mainly of use in designing a probe that does not retard flow, but a trial solution can be used to find k if necessary. It may be noted that forcing a probe into place is likely to result in the formation of a zone of reduced conductivity around the probe [*102*].

Use of Excessive Heads—With a probe, use of excess pressures that are near the initial minor principal effective stress in the soil is likely to cause hydraulic fracturing of the soil and a measurement of a value of conductivity that is too high [*111,112*]. The problem may be particularly acute in the case of measurement of conductivity in a slurry trench where part of the weight of the trench backfill may be supported through shear along the sides of the trench, which leads to arching and to lower vertical effective stresses in the trench backfill than expected. Bjerrum et al [*112*] report that the vertical effective stress in such cases may be so low that hydraulic fracturing occurs merely by filling a cased borehole with water. For tests where the water flows into the probe from the soil, use of excessively low heads may lead to problems with cavitation and to formation of a zone of less permeable soil near the probe due to the increased effective stress [*10*].

Use of "Dirty" Water—Use of dirty water in tests where water flows from the probe into the soil may lead to clogging of the pores of the soil [*91,105,113*].

Head Loss in Entrance Tubes—In exceptional cases involving use of long entering tubes of small diameter, significant head losses may occur in these tubes [*99*].

Sealing—In soils of conductivity less than about 1×10^{-8} cm/s there may be serious problems in sealing the entrance tubes to the probe [*99*].

Field Measurement of Hydraulic Conductivity in Partially Saturated Soils

All of the methods discussed previously for field measurements in saturated soils could also be used in partially saturated soils, provided consideration is restricted to inflow tests. However, data from simple inflow tests are difficult to interpret because the water content, and hence hydraulic conductivity, is continually changing during the test. Even if steady-state seepage were eventually achieved, water content and conductivity would vary spatially. Thus, more elaborate testing methods are required with partially saturated soils. Two methods are discussed in the following sections.

Instantaneous Profile Method

Various forms of the instantaneous profile method have been used in agronomy to measure the conductivity of shallow, partially saturated soils in the field. Typically, the procedure is as follows. A plot of land, several metres or more across, is ringed with a low dike. Probes for measuring suction (usually

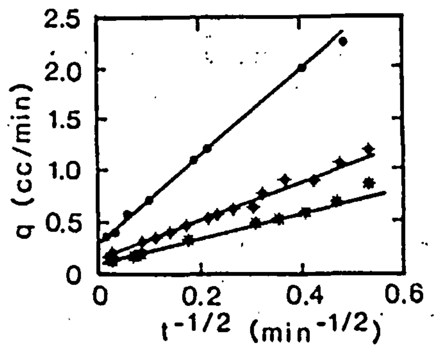

FIG. 19—*Field observations of flow rate versus the inverse of the square root of time (from Al-Dhahir, Kennard, and Morgenstern [108]).*

tensiometers) are inserted into the ground near the center of the plot at several depths. Probes for measuring water content may also be inserted. The plot of land is then flooded with several inches of water. After the water has seeped into the ground, the plot is covered with a sheet of plastic to prevent evaporation. As water percolates downward, suctions are measured as a function of depth and time. The water content is either measured directly or estimated from field measurements of suction and laboratory correlations between water content and suction [*114*]. The data are reduced using Eqs 10 to 12.

Several variations in this procedure have been tried (Table 3) [*115-123*]. Instead of flooding a plot of land with water, one can wait for a heavy rain. Evaporation may be allowed, but interpretation of the data is uncertain unless the rate of evaporation is known. Some have tried measuring the water content *in situ* with neutron probes and estimating the suction from water content-suction curves measured in the laboratory [*118,119*].

Advantages of the instantaneous profile method for field measurements include modest equipment requirements and relatively straightforward interpretation of data. Problems include the following: (1) tensiometers are often installed improperly; (2) water frequently flows into access tubes housing tensiometers; (3) if tensiometers are used, suctions are restricted to less than about 0.9 atm; (4) water flow at the probe locations may not be purely one-dimensional; (5) the method is restricted to shallow depth; (6) the plot of land must be level; and (7) testing times may be long in relatively impervious soils [*114*].

Infiltration Through Impeding Layer

In this method, described by Gardner [*124*], Hillel and Gardner [*125*], and Bouma et al [*126*], a column of soil in the field is isolated by pushing a thin-walled tube to a suitable depth. Baker [*127*] reports that the best tube

TABLE 3—*Experimental apparatus and techniques in field measurements of conductivity with the instantaneous profile method.*

Reference	Soil	Size of Plot, m	Evaporation Allowed?[a]	Suction Probes	Water Content Probe	Maximum Depth of Probes, m
Richards et al [115]	sandy loam	2.6 × 2.6	yes	tensiometers	direct sampling	0.8
Ogata and Richards [116]	sandy loam	2.6 × 2.6	no	tensiometers	direct sampling	0.5
Nielson et al [117]	clay loam	3.7 × 3.7	no	tensiometers	none	1.5
Rose et al [118]	loam	...	yes	none	neutron method	1.8
Rose and Krishnan [119]	clayey sand	...	yes	none	neutron method	1.6
van Bavel et al [120]	clay loam	5 × 10	no	tensiometers	neutron method	1.6
Davidson et al [121]	loam and silty clay	3 × 3 and 10 × 10	no	tensiometers	none	1.8
Hillel et al [122]	sandy loam	10 × 10	no	tensiometers	neutron method	1.5
Nielson et al [123]	clay loam	6.5 × 6.5	no	tensiometers	none	1.8

[a] No special plot; the test was performed in the field after a heavy rain.

diameter and length are about 24 and 30 cm, respectively. The column is capped with a relatively impervious porous stone, membrane, crust of soil, or other impeding material. Water is ponded on the impeding layer and a small, constant head is maintained long enough for steady-state seepage to develop. The rate of flow through the stone is measured with a Mariotte bottle or other suitable device. Developers of the method claim that this procedure produces a hydraulic gradient of unity in the soil directly beneath the porous stone; hence the conductivity of the soil is equal to the measured velocity of inflow. Suction in the soil directly beneath the impeding layer is a function of the conductivity of the impeding layer. Typically, tensiometers are inserted into the ground to confirm that the gradient is one and to measure the suction corresponding to the observed conductivity.

This method appears to offer little advantage over laboratory tests because a thin-walled tube must be pushed into the ground to ensure one-dimensional flow. Essentially identical results could probably be obtained by removing the tube filled with soil and testing the material in the laboratory. For fine-grained soils, the impeding porous stone would typically have to be so impervious that accurate measurements of flow rates might be impossible to obtain. Similarly, in many fine-grained soils, it may be impractical to wait for steady-state seepage to develop.

Comparison of Hydraulic Conductivities Measured in the Laboratory and Back-Calculated from Full-Scale Field Observations for Saturated Soils

When the permeability tests are performed to allow a designer to estimate flow rates in the field, the ultimate check on the validity of the laboratory measurements is clearly a comparison of predicted flows with values measured in the field for full-scale projects. For fine-grained soils we find no such comparison. The reasons seem to include the following:

1. The total amount of water that moves is too small to be of interest if the soil is fine grained. Interest is concentrated in cases where water flow leads to settlement, change in stability conditions, or transport of pollutants.
2. Field conditions are often so complex that there is no means available for collecting flows. Even in cases where the observation could consist of measuring the arrival time of a pollutant, the hydrogeologic conditions are often too complex to allow the field conductivities to be backed out of an analysis.
3. The costs involved in obtaining the laboratory and field measurements have precluded obtaining the data in many cases.
4. In some cases, such as flow around toxic waste disposal sites, the flows will occur over times ranging from decades to centuries, thus making it difficult to obtain useful observations in reasonable periods of time.

One of the best opportunities for estimating field conductivities is obtained by measuring time rates of settlement of wide embankments above soft clays, although in this case the coefficients of compressibility must be obtained from other measurements. Sophisticated analytical techniques are required to account for the dependency of soil properties on effective stress. Several cases have been analyzed to compare laboratory and field curves of void ratio and coefficients of consolidation versus vertical effective stress [*5,128,129*], but no values for the conductivity were reported.

Comparison of Hydraulic Conductivities Measured in the Laboratory and *In Situ*

An attempt was made to tabulate data from various sites where field and laboratory conductivities had both been measured. In many cases it was necessary to simplify data by reporting average values when there was significant scatter or by reporting data at only one effective stress. It was often unclear how certain measurements were made, and inferences were drawn from general discussions in some cases. The data are presented in Table 4 [*6,10,60,102,103,108,130–137*]. The range in the ratio of field k/laboratory k is from 0.3 to 46 000, but nearly 90 percent of the observations lie in the range from 0.38 to 64. It appears that the major causes of the higher values of the field k are (1) a tendency to run laboratory tests on more clayey samples [*6*]; (2) the presence of sand seams, fissures, and other macrostructures in the field which are not represented properly in laboratory tests; (3) the use of laboratory k values back-calculated from consolidation theory rather than directly measured values; (4) measurement of vertical flow k in the laboratory and horizontal flow k in the field; (5) the use of distilled water in the laboratory; and (6) air entrapment in laboratory samples.

Larger scale field tests may be performed by isolating an area of soil using a metal wall, flooding the isolated area, and then measuring the inflow and the pore pressure distribution with depth. One of the most complete studies of this kind was reported by Ritchie, Kissel, and Burnett [*135*]. They isolated two areas, one a 10-m square and the other a 2.5-m square. Field pore pressure measurements showed that the flow was straight down with a hydraulic gradient of one. The upper soil was Houston black clay; free drainage occurred at a depth of 175 cm. They also performed laboratory permeability tests. The value of k back-calculated from field observations was 3×10^{-5} cm/s at both sites, compared with laboratory measurements of k that varied with sample size but fell within the range of 8×10^{-7} to 3×10^{-6} cm/s. To investigate flow through fissures, they also permeated samples with fluorescein, a material that fluoresces in ultraviolet light, and found that the fluid was apparently flowing through the following percentages of the total area:

Depth, cm	Flow Area, %
5	100
20	60
35	10
50	2

In this fissured clay, the flow apparently concentrated in the fissures.

Summary and Conclusions

There are significant margins for error in both laboratory and field tests. For saturated soils, field tests are to be preferred, provided they are performed and interpreted properly, because they permeate a larger volume of soil than laboratory tests, thus taking into account the effects of macrostructure, such as roots and fissures. Field tests are generally best performed by using a cylindrical piezometer tip, installed by methods that minimize disturbance, and using the constant head technique. Curves should be prepared of q-$t^{-1/2}$ and flow continued until $dq/dt = 0$ and a reasonable estimate of the steady-state q is obtained. More research is needed to develop improved methods of field testing and correlation of predicted and actual flows.

Laboratory tests offer the advantage of economy, and for many current applications this consideration is a dominating factor. Laboratory tests on natural samples should use undisturbed samples of the largest practicable size, and samples should be oriented in the proper direction, typically so that the flow is in the direction of maximum hydraulic conductivity. The permeant should be a fluid similar to that encountered in the field. Care should be taken to avoid accumulation of air bubbles in the sample. Hydraulic gradients should be kept as low as possible while still allowing tests to be performed within a reasonable time. If these precautions are not heeded, the laboratory k may differ from the field values by as much as several orders of magnitude (Table 5).

Field testing for measurement of conductivity in unsaturated soils is at such a rudimentary stage of development that field measurements cannot currently be recommended except for agricultural purposes or cases where water will be ponded on the surface of a site. Laboratory testing methods for unsaturated soils are better developed than field methods, but many of the problems mentioned previously for saturated soils also apply to unsaturated soils. The best laboratory techniques presently appear to be as follows: (1) for suctions between 0 and 0.9 atm, the instantaneous profile method with tensiometric probes; (2) for suctions between 2 and 80 atm, the instantaneous profile method with psychrometric probes; and (3) for suctions between 1 and 15 atm, the pressure plate outflow method.

For problems of practical interest, it is clear that permeability tests (lab-

TABLE 4—*Comparison of laboratory and field hydraulic conductivities.*

Reference	Site	Soil	Laboratory Test[a]	Field Test[b]	*k* cm/s Laboratory	*k* cm/s Field	Field *k* / Laboratory *k*
Skempton and Henkel [*130*]	Bradwell	clay	M	piezometer	4.5×10^{-9}	3.7×10^{-9}	0.8
			KT	rising head	1.2×10^{-9}	3.7×10^{-9}	3.1
Golder and Gass [*131*]	Netherlands	sandy clay		suction bellows	3.2×10^{-7}	1.2×10^{-7}	0.4
Weber [*6*]	Pismo	silty clay	KT	P	6.9×10^{-9}	3.4×10^{-5}	4 900
		silty clay			2.0×10^{-8}	2.7×10^{-7}	14
		silty clay			1.1×10^{-8}	7.7×10^{-9}	0.7
	Lafayette	silty clay			5.5×10^{-8}	2.5×10^{-7}	4.6
					4.0×10^{-7}	1.7×10^{-5}	43
	Atascadero	sandy silty clay			3.9×10^{-8}	1.8×10^{-3}	46 000
		sandy silty clay			8.5×10^{-5}	8.5×10^{-5}	1
	La Trianon	silty clay			2.8×10^{-7}	4.2×10^{-6}	15
		silty clay			1.6×10^{-7}	2.0×10^{-4}	1 200
		silty clay			1.2×10^{-7}	6.1×10^{-6}	51
	Napa River	bay mud			3.3×10^{-7}	7.1×10^{-7}	2.2
					2.9×10^{-7}	9.6×10^{-7}	3.3
					2.1×10^{-7}	9.1×10^{-7}	4.3
					3.4×10^{-7}	4.2×10^{-7}	1.2
					1.4×10^{-7}	4.1×10^{-7}	2.9
					1.9×10^{-7}	6.2×10^{-7}	3.3
					1.9×10^{-7}	4.3×10^{-7}	2.3
					4.2×10^{-7}	3.9×10^{-7}	0.9
					1.2×10^{-7}	3.3×10^{-7}	2.8
Wilkinson [*103*]	Frodsham	organic silty clay			1.8×10^{-7}	5.0×10^{-7}	2.8
					8.0×10^{-8}	5.0×10^{-7}	6.2
					2.7×10^{-8}	5.0×10^{-7}	19
Al-Dhahir, Kennard and Morgenstern [*108*]	Fiddler's Ferry	silty clay	KT	P, C	1.1×10^{-8}	1.2×10^{-7}	11
Raymond and Azzouz [*10*]	Lyndhurst	peat	M	constant head, flow into cell	5.0×10^{-5}	3.0×10^{-4}	6.0
		marl			3.3×10^{-6}	2.1×10^{-4}	64
		algae			5.0×10^{-7}	1.9×10^{-4}	38
		AEC(?)			5.7×10^{-7}	1.4×10^{-6}	2.5
		clay			4.2×10^{-7}	5.2×10^{-7}	1.2
					3.7×10^{-7}	1.7×10^{-7}	0.5
					3.0×10^{-7}	8.7×10^{-8}	0.3
					1.0×10^{-7}	1.1×10^{-7}	1.1
Bishop and Al-Dhahir [*132*]	Balderhead	clay fill	M		1.0×10^{-8}	1.0×10^{-8}	1.0
	M6	clay fill	M		3.4×10^{-8}	5.0×10^{-8}	1.5
	Fiddler's Ferry	alluvium	KT		1.0×10^{-8}	8.0×10^{-8}	8.0
	Selset	core clay	M	P, C, I	7.7×10^{-8}	7.9×10^{-8}	1.0
	Selset	foundation clay	KT		3.8×10^{-9}	8.4×10^{-9}	2.3
	Diddington	core	KT		1.2×10^{-8}	3.3×10^{-9}	2.8
		foundation clay	KT		4.0×10^{-9}	1.8×10^{-9}	0.4
Casagrande and Poulos [*60*]	New Jersey Turnpike	varved clay	M, H		2×10^{-6}		
			M, V		2×10^{-7}		
				P, jetted		2×10^{-6}	1.0
				P, driven		2×10^{-7}	0.1
				WP, jetted		6×10^{-6}	3.0
				WP, driven		2×10^{-6}	1.0
				SD, jetted		4×10^{-5}	20
				SD, driven		4×10^{-6}	2
James [*133*]	Malaya	silty clay	KT	P-V	1.0×10^{-7}	1.0×10^{-5}	100
Murray [*134*]	Avonmouth	brown clay	KT	P	3.0×10^{-7}	2.0×10^{-6}	6.1
		blue clay	KT	P	1.7×10^{-6}	1.5×10^{-5}	8.6
		peat	KT	P	2.0×10^{-7}	2.0×10^{-6}	10
		silty clay	KT	P	3.4×10^{-7}	1.6×10^{-5}	47
Ritchie, Kissel and Burnett [*135*]	Houston	clay	M	10 × 10 m	5.7×10^{-6}	3.1×10^{-5}	5.4
					3.5×10^{-6}	3.1×10^{-5}	8.9
					8.1×10^{-7}	3.1×10^{-5}	38
					4.0×10^{-6}	3.1×10^{-5}	7.8
Jezquel and Mieussens [*102*]	Bordeaux	clay	KT	constant head, flow into soil	3.0×10^{-7}	2.6×10^{-7}	0.9
					8.0×10^{-8}	2.6×10^{-7}	3.2
					5.0×10^{-8}	2.5×10^{-7}	5.0
					8.0×10^{-8}	4.0×10^{-7}	5.0
					1.7×10^{-8}	2.0×10^{-7}	11.8
					5.0×10^{-8}	2.0×10^{-7}	4.0

TABLE 4—*Continued.*

Reference	Site	Soil	Laboratory Test[a]	Field Test[b]	k cm/s Laboratory	k cm/s Field	Field k / Laboratory k
Goodall and Quigley [136]	Sarnia, Ontario	silty clay	KT	P-V	1.5×10^{-8}	1.2×10^{-8}	0.8
			M, KT	P-V	1.6×10^{-8}	1.2×10^{-8}	0.8
			M	P-V	2.1×10^{-8}	1.6×10^{-8}	0.7
				P-V	2.6×10^{-8}	4.6×10^{-8}	1.8
Mieussens and Ducasse [137]	France	clay	KT	P	3.0×10^{-7}	2.6×10^{-7}	0.9
					8.0×10^{-8}	2.6×10^{-7}	3.3
					5.0×10^{-8}	2.5×10^{-7}	5.0
					8.0×10^{-8}	4.0×10^{-7}	5.0
					1.7×10^{-8}	2.0×10^{-7}	12.0
					5.0×10^{-8}	2.0×10^{-7}	4.0

[a] Key to symbols used:
KT—back-calculate *k* using Terzaghi's theory
M—measure *k* directly
H—horizontal
V—vertical

[b] Key to symbols used:
P—piezometer method
A—auger method
I—inflow (water into soil)
O—outflow (water out of soil)
C—constant head
V—variable head
WP—well points
SD—sand drains

TABLE 5—*Summary of published data on potential errors in laboratory permeability tests on saturated soils.*

Source of Error and References	Measured *k* Too Low or Too High?	Published Data on Typical (Measured *k*)/(Correct *k*)
1. Voids formed in sample preparation	high	>1
2. Smear zone formed during trimming	low	<1
3. Use of distilled water as a permeant [32,34]	low	5/1000 to 1/10
4. Air in sample [44]	low	1/10 to 1/2
5. Growth of microorganisms [45]	low	1/100 to 1/10
6. Use of excessive hydraulic gradient [47,48]	low or high	<1 to 5
7. Use of wrong temperature (Fig. 8)	varies	1/2 to 1 1/2
8. Ignoring volume change due to stress change (Fig. 9)	high	1 to 20
9. Flowing water in a direction other than the one of highest permeability (Table 1)	low	1 to 40
10. Performing laboratory rather than *in situ* tests (Table 4)	usually low	<1/10 000 to 3

oratory or field) must be performed with a great degree of care and attention to detail. However, just performing the tests properly does not ensure successful results. Thorough field investigation to identify zones of maximum and minimum conductivity, and careful selection of samples or layers for testing, are in some respects more important than experimental technique. Even with a comprehensive field investigation and suitable experimental technique, some degree of judgment must inevitably be exercised before the results are used for field predictions.

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Rock Permeability or Hydraulic Conductivity—An Overview

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ABSTRACT: The water transmission characteristics of rock formations are far more variable than those of most soils. The flow conduits include (1) primary porosity, the voids between mineral grains or fragments; (2) genetic porosity, voids which developed within the rock during its formation; and (3) secondary porosity, the joints, shear zones, and other cracks that formed subsequent to deposition. The porosity changes over time as the voids both fill up and become enlarged.

The flow patterns are complex, with tortuous paths between the different forms of porosity. Laminar and turbulent flow occur simultaneously, depending on void size and energy gradients. The flow can be characterized by a variable pseudopermeability coefficient, k_p, in the expression $q = k_p i^N A$, where N varies from 1 for laminar flow to 0.5 for turbulent flow. Laboratory tests are useless unless the sample size is an order of magnitude larger than the secondary porosity spacing. Field measurements in bore holes or an evaluation of subregional discharge is necessary for making a realistic evaluation of k_p of rock aquifers. There are likely to be large variations in k_p depending on location, time, and changes in the groundwater environment.

KEY WORDS: rock, groundwater, permeability, seepage well tests, pumping test

The abutment of an earth dam in the southeastern Piedmont region was leaking nearly 4.6 m^3/min (1200 gal/min). A reevaluation of its stability was undertaken to determine the mechanism of the leak and its possible risk to the project integrity. A network of piezometers was installed to establish the pore pressures and flow gradients. The data were to be used in stability analyses and to aid in planning for grouting to reduce the flow. Flow net analyses were made of the abutment to select the most effective piezometer locations and depths.

The piezometric readings were different from what had been expected from seepage analysis based on porous media. Some downstream piezome-

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