Measurement of the Hydraulic Conductivity Above a Water Table in Situ

Anthony S. Dylla

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MEASUREMENT OF THE HYDRAULIC CONDUCTIVITY

ABOVE A WATER TABLE IN SITU

BY

ANTHONY S. DYLLA

A thesis submitted
in partial fulfillment of the requirements for
the degree Master of Science, Department of
Agricultural Engineering, South Dakota
State College of Agriculture
and Mechanic Arts

March, 1960
This thesis is approved as a creditable, independent investigation by a candidate for the degree, Master of Science, and acceptable as meeting the thesis requirements for this degree; but without implying that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Thesis Adviser

Head of the Major Department
ACKNOWLEDGEMENTS

The author is deeply indebted to Professor Dennis L. Moe and Professor John L. Wiersma of the Agricultural Engineering Department, to Associate Professor Jack R. Runkles of the Agronomy Department, and to Martin Fogel, Irrigation Specialist, all of South Dakota State College, for their technical assistance and encouragement in conducting this study and in preparing this paper.

Sincere appreciation is also acknowledged to South Dakota State College Agronomy Department for their generous cooperation in providing a site for the field experiments.

Acknowledgement is also noted of the generous assistance that was willingly granted by all members of the Agricultural Engineering Staff whenever it was needed during the construction and performance of the field trials. The author is also indebted to members of the Agricultural Engineering Staff for their review and constructive criticisms during the preparation of this paper.

A.S.D.
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CHAPTER I

Introduction

The physical property of soil which enables it to pass or conduct water through its macro-pores is known as its permeability. Permeability should not be confused with infiltration, which is the term for expressing the absorption of water into the soil. The infiltration rate is the rate at which water will enter the soil surface. The rate at which water moves through soil is called the hydraulic conductivity; whereas a measurement of this same rate is used to calculate the permeability.

Hydraulic conductivity is most commonly expressed in the units of velocity, whereas permeability is expressed as the square of some unit of length, or \((L)^2\). The specific need for hydraulic conductivity and permeability measurements is to determine the rate at which water will move through soil. The design of most subsurface drainage is based on hydraulic conductivity and permeability measurements. This study will deal primarily with the measurement of soil permeability and conductivity as a function of the drainability of agricultural lands.

As the food requirements of a growing population increases, drainage will be one of the principle tools to help increase food production. The most fertile, and potentially productive agricultural land is located on bottom lands along rivers and streams. The hindrances preventing it from being put to maximum productive use are the presence of a high fluctuating water table and periodic surface flooding. Both of these can be controlled or minimized by the installation of properly designed drainage and water control measures.
Subsoil drainage of agricultural land is accomplished by two principal methods - interceptor drains and collector systems. The interceptor drain is an underground tile line placed perpendicular to the direction of the horizontal movement of water through the soil. It collects or intercepts the water, thereby lowering the water table on the down-slope side of the tile line. Its effectiveness is limited to the more arid regions where there is a minimum of water table recharge from the surface directly above the affected area. Collector drains are commonly designed in a geometric pattern. The lines are generally parallel and drain to a common outlet. They are most commonly used in areas where the ground water build-up or recharge is primarily from surface infiltration.

There is also a particular need for drainage in areas where irrigation is being practiced and no natural drainage exists. It is also needed for certain areas that are being contemplated for irrigation development. Past experiences indicate that adequate drainage is necessary for successful irrigation. Any development and expansion of irrigation in some of the new areas proposed for irrigation is going to depend considerably on the economics of providing the drainage system along with the water distribution system.

Some common mistakes made in the past have been on lands that were irrigated without having provided adequate soil drainage. Lack of good quality irrigation water, presence of high concentrations of salt and poor drainability all contributed to creating problems that became more acute as time passed. Several of the irrigation projects had to be abandoned until adequate drainage could be provided to remove the excess water and
salt accumulations. This has resulted in the installation of some very expensive drainage systems. Many of the costly mistakes could have been eliminated at the beginning if sufficient soil drainage information had been acquired before the projects were initiated.

South Dakota has several areas where irrigation could be performed successfully if adequate drainage were provided. Most prominent of these areas is the James River Basin, also referred to as the Lake Plains and the old Lake Dakota Basin. This proposed irrigation area, which will get its water supply from the Oahe Reservoir on the Missouri River, extends along both sides of the James River approximately from near Redfield, South Dakota to the North Dakota border. The rate of development of this area for irrigation is going to hinge considerably on the costs of providing adequate drainage.

Other proposed irrigation areas in South Dakota are located principally along rivers. Some of these areas already have adequate natural drainage, whereas other areas must have drainage systems constructed.

It requires a great deal of knowledge, experience, and investigation to determine drainage requirements and costs. This brings up some questions: What are the drainage design requirements? What criteria are used to determine drainage requirements? What lateral line spacing and what depth of placement of tile drains will most economically provide effective drainage? None of these can be answered without some knowledge of the soil permeability.

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Methods of determining tile drain line spacings and depths are numerous. Almost every physical property of soil has been used at some time or other as a gauge or "measuring stick" for determining tile spacings and depths. Soil texture, organic matter content, porosity, soluble salt content, upper and lower plastic limits, per cent sodium and soil permeabilities have all been used to determine tile line spacings and depths. Practically all of the above listed properties have been determined by performing laboratory tests on relatively small samples of soil.

Results of laboratory tests on small soil samples may have a good scientific basis for determining tile spacings except for two arguments. One argument is that an assumption has been made that the sample tested was representative of all the soil in the area from which it was taken. Generally speaking, one was assuming that the soil in that particular area was homogeneous. This assumption should not be accepted except in the case of performing certain mathematical calculations where it is impossible to predict the soil variability. The variability can be minimized by utilizing larger and more representative samples. The other argument for discrediting most of the laboratory tests is that the sampling procedure does disturb the physical structure of the soil as found in its natural position. Different technicians may vary their laboratory procedure. This may give a variety of results, none of which may truly represent the sample in situ. Many of the drainage scientists and investigators agree that hydraulic conductivity measurements made only on undisturbed core samples and samples in situ are acceptable.
CHAPTER II

The Problem

It has been recognized that the development of the James Basin or Lake Plain area for successful irrigation will require constructed drains to control the rise of the water table. Considerable investigative work has already been conducted by the U. S. Bureau of Reclamation on this proposed project.

Three methods of measuring the permeability or hydraulic conductivity were employed by the Bureau. Field laboratory permeability measurements were made on disturbed samples and undisturbed core samples. Relatively small samples were used for both of these methods. The third method obtained hydraulic conductivity measurements in the field using the ring infiltrometer. A brief description of each procedure is included in Appendix D.

It seems logical that the field site measurement of permeability would be the most accurate. However if any correlation could be obtained between a simple low cost sampling procedure and the field measurements, considerable time and expense could be saved.

Correlation studies were conducted by the U. S. Bureau of Reclamation to determine if any significant correlation existed between any of the three methods they employed. These studies reveal that considerable

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error could result if the field permeabilities were to be predicted from permeabilities obtained from disturbed and undisturbed samples.\(^3\)

Based on field observations and tests, it was decided that the Lake Plain soils should be adequately drained with an average tile spacing of 330 feet. It was stated that the danger of excessive salts moving to the surface would be small if the water table could be maintained below the 48 inch level.\(^4\)

There were several different ideas on what tile spacing would be adequate, as evidenced by the following quote:

Using the median of the required data obtained during the drainability investigations, the Glover drain spacing formula and Moody correction, which is a theoretical formula for computing drain spacing, developed by the Bureau of Reclamation, gives a spacing 180 feet which would theoretically provide for "perfect" drainage. Experienced drainage personnel, both in the Bureau of Reclamation and those hired as Consultants by the Bureau, differ in their opinions on what the practical drain spacing should be in lands similar to those found in the Lake Plain. Each of the 3 Consultants on the Consulting Board for Oahe Unit, after numerous field inspections, selected different practical drain spacings for the Lake Plain, ranging from a minimum of 225 feet to a maximum of 330 feet for tile at a 9-foot depth. Personnel from the Drainage and Ground Water Engineering Office in Denver selected a spacing of 360 feet for a 9-foot deep drain.\(^5\)

A decision was made at a Bureau of Reclamation conference in Denver in 1955 to use an average drain spacing of 330 feet at 9-foot depth to estimate the cost of drainage. In actual construction the spacings may vary from less than 100 feet to more than 600 feet.\(^6\)

\(^3\)Ibid., pp. 48\textsuperscript{f}-52\textsuperscript{f}, and p. 103\textsuperscript{f}.

\(^4\)Ibid., p. 3\textsuperscript{f}.

\(^5\)Ibid., p. 77\textsuperscript{f}.

\(^6\)Ibid., p. 78\textsuperscript{f}.
It is believed by this author, that the Bureau Engineers and Consultants probably have exercised good judgement in increasing the tile line spacings from the theoretically required 180 feet to the more practical 330 feet. However, this author is not aware of any proven evidence to support their conclusions. One reason advanced for the increased spacing was that the James Basin area would have relatively low surface evaporation as compared to the arid regions of the Southwest. It is the belief, therefore, that less drainage requirements would be needed to keep salt concentrations from moving toward the surface by capillarity.  

Casagrande in 1937 and Vreedenburgh in 1938, both advanced the theory of soil anisotropy. Basically, soil anisotropy is a concept that the horizontal permeability is different than the vertical permeability. None of the known methods employed by the Bureau of Reclamation would determine if soil anisotropy was present in the upper profile of the Lake Plain soils. However, there are reasons for believing that the lake bed silts of this area could be anisotropic. Geologically, the silts were water deposited and consequently they may have some horizontal structural development.

There is still some question as to whether the practical 330 foot tile line spacing at the 9-foot depth would provide adequate drainage for irrigation purposes. The author believes that some support for a wider spacing than the 180 feet may be obtained if the concept of soil

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7Ibid., p. 777.
anisotropy is valid for the Lake Plain soils. The author also believes that only field measurements or in situ permeability measurements are acceptable.

The problem involved is one of measuring the hydraulic conductivity above a water table. The objective of this study is to devise a method or procedure which may be used to obtain these measurements. Some of the desired features of the method are:

1. May be conducted with relatively little expense.
2. May be conducted without time delays.
3. May be performed over extensive areas without great inconveniences.
4. Must be conducted in situ.

To accomplish the above stated objective, a review will be conducted of the various methods proposed by other investigators. Field trials will be performed to determine if one or several of the methods may have some practical adaptation to the particular problem of this study.
CHAPTER III

Review of Pertinent Studies

An extensive amount of studies have been made and an abundance of material is available on the theory of drainage and its applications. It is not the intent of this writer to mention all of the review literature in this manuscript. The vast amount of material does emphasize the importance of land drainage and soil physics. That drainage is not a newly recognized need can be evidenced by the following quotation:

The practice of the art of drainage is as old as the art of agriculture. The first recorded examples occurred during the times of the Roman Empire and probably earlier. The Romans recognized the importance of soils information as a basis of drainage design and superiority of deep and covered drains under certain circumstances. The methods used by these people were little improved until the present day tile drainage had its origin in England. . . . (An earlier use of tile in France in 1620 in the Convent Garden at Musbeuge was followed by widespread adoption of the practice.)

Darcy’s Law of Flow

The theoretical development of drainage principles may be considered to have started about 100 years ago in France by Henry Darcy. Darcy’s Law has been written in many different forms. Probably most commonly expressed as

$$Q = \frac{G \cdot A \cdot H}{L}$$

where $H$ is the energy expended to produce a quantity of flow $Q$ through a

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flow path of cross-sectional area $a$ and of length $L$. The constant $C$ is characteristic of the porous media through which the flow is caused.

Investigators studying Darcy’s Law have concentrated on two aspects. One group attempts to verify the law as shown by equation (2), with some modification using the dimensional theory. The other group has been primarily concerned with the constant $C$ and its relationship to the characteristics of the porous medium through which flow occurs. Darcy’s law of flow has been verified as valid as long as the velocity of flow remains viscous or laminar. As soon as the flow becomes turbulent or even partially turbulent, the law loses its strict validity.\(^\text{10}\)

By applying the theory of dimensions to the law of flow, the relationship is illustrated as

$$\Delta P = \text{constant} \frac{\mu^2}{d^2} F \left( \frac{dv}{\mu} \right) \phi \left( \frac{As}{d} \right) \quad (2)$$

where $\Delta P$ is the pressure drop through a column of length $As$, carrying a fluid of density $\gamma$ and viscosity $\mu$ with an average velocity $v$. Function $F$ can be recognised as the Reynolds number, and $d$ of function $\phi$ is a length characteristic of the pore size of the medium. For low velocities or viscous flow, functions $F$ and $\phi$ can be accepted at equality.\(^\text{11}\)

Simplifying equation (2) would then give

$$\frac{\Delta P}{As} = \text{constant} \frac{\mu v}{d^2} \quad (3)$$


\(^{11}\)Ibid., p. 56.
It is difficult to determine the exact limits of validity for Darcy's Law. The difficulty lies in the ambiguity of the definition of the quantity $d$ entering into the Reynolds number.\(^{12}\) If Reynolds number is less than 1, the law is assumed valid for any drainage situation.\(^{13}\)

Equation (3) is often expressed in other forms. The term $(\Delta P/\Delta s)$ is the potential gradient and more commonly denoted as $i$. The term $(d^2/$constant$)$ is otherwise denoted as $k$. The equation is now shown in one of its more common forms:

$$v = k i$$ \quad (4)

The constant $k$ is called the hydraulic conductivity of a specific fluid through a specific body, and will carry the dimensions of velocity. To clarify some of the terminology, permeability is defined as the property of the porous media independent of the fluid, and henceforth shall be denoted by $\bar{k}$. It shall also be referred to as the intrinsic permeability, and will be expressed as the square of some dimension of length.\(^{14}\) Equation (4) could also be written in the following form:

$$v = \frac{\bar{k} g i}{\gamma}$$ \quad (5)

where $\gamma$ is the gravitational constant, $\gamma$ is the density of the conducted fluid, and $\nu$ is the absolute viscosity of the conducted fluid.

\(^{12}\text{Ibid., pp. 66-67.}\)


\(^{14}\text{Ibid., p. 48.}\)
Permeability and Hydraulic Conductivity

The definitions of permeability and hydraulic conductivity given in the preceding section will now be adhered to throughout the study. In comparing the definitions of these two measures, it would appear that permeability and hydraulic conductivity would be directly related if permeability were determined using water as the conducted fluid. Fundamentally, however, the permeability expression seeks to eliminate those factors of soil-water interaction and of flow characteristics of water. It is expressed as the square of the mean effective pore diameter of the porous medium and is independent of characteristics of the conducted fluid, such as viscosity, surface tension, and density. Permeability, then, seeks to express the rate of movement of any fluid as a function of pore size and pore distribution of the porous medium, whereas hydraulic conductivity tends to describe the rate of movement of water as a function of these properties at some standard condition of temperature.

In discussing the changes of viscosity of the conducted fluid, we shall be concerned primarily with those of ground water. The porous medium through which the ground water will pass will be soil. A change in viscosity may be caused by a marked change in temperature. However, some viscosity changes are brought about by the amount of colloids and salts present in the water. These latter may result in a partial clogging of the soil pores, as well as changing the viscosity of the fluid.

A small amount of sodium chloride in ground water will change the viscosity very little; however, it may cause a large change in the soil
structure. This fact is important to hydraulic conductivity measurements. Hydraulic conductivity is related to the pore space and sodium will tend to close the pores. Soils high in "expanding" clay particles will also swell or close when saturated with water. If this "expanding" effect were not true, it would be relatively simple to determine the relationship between total pore space and hydraulic conductivity. Conductivity relationships could be readily established from the porosity of sands and other non-expanding media. However, this study is no longer concerned with the hydraulic conductivity of sands, for it is with these media that no real drainage problem exists.

Of two soils with the same total porosity, that which has the greater percentage of macro-pores will have the higher hydraulic conductivity. In soils of fine texture, the hydraulic conductivity is dependent almost entirely on the amount of macro-pores, which is a manifestation of the development of good soil structure.

In speaking of the soil structure, reference is also made to the development of natural fissures through the soil. It is recognized that platy or laminar structured soils have a greater amount of horizontal fissures than the prismatic and columnar structured soils. One could conclude that this could also cause greater conductivities in a horizontal direction. This theory or concept is referred to as soil anisotropy. Soil holes and cracks due to worms and roots also affect the hydraulic conductivity; however, it is difficult to extract soil samples for

\[15Ibid., p. 48.\]
\[16Ibid., p. 50.\]
laboratory conducted measurements without physically altering these openings and cracks in the soil.\textsuperscript{17} Thus, laboratory measurements would seldom be duplicated in the field.

There is some indication that the soil permeability is affected by microbial activity resulting from prolonged saturation. This activity occurs during prolonged submergence, prolonged leaching operations, and/or extensive water spreading on agricultural soils. The soil pores probably become obstructed with microbial growth, cells, slimes, or poly-saccharides.\textsuperscript{18} It is difficult to determine just how much the permeability may be affected by the presence of colloids and micro-organisms, and by hydration of the clay particles. It appears that the soil permeability is decreased by any periods of prolonged wetting.

Permeability and hydraulic conductivity measurements may also be affected by presence of a second fluid within the porous medium. This condition exists whenever the fluid is a liquid and the second fluid is gas or air that is entrapped in the pores of the soil. The evolution of air from the liquid will continue until an equilibrium is reached. This process was described by Wycoff and Botset in 1936.\textsuperscript{19} Equilibrium may be reached or approached only after extended periods of time.


\textsuperscript{19}R. C. Reeve, \textit{op. cit.}, p. 409.
Christiansen suggested de-airing the soil with carbon dioxide before saturating with water. The carbon dioxide would become soluble in water and the soil would be air-free immediately. Reeve observed in 1953 that the rapid solution of carbon dioxide by the saturating water increases the structural breakdown of soils and the final permeability result may be lower.20

In the theory of flow, soil is usually assumed to be homogeneous. It is generally recognized that homogeneity in soil is not true. Permeability measurements of small samples would be representative of a large volume of heterogeneous soil only if an adequate number of samples were used. This fact alone favors taking measurements from larger samples.

Methods of Measuring Permeability and Hydraulic Conductivity

Various methods of measuring the hydraulic conductivity in the field have been devised. Formulas have been developed to translate the flow measurements into hydraulic conductivity. Some of the investigators have exact mathematical solutions, some have used approximate solutions (the soil was assumed to be heterogeneous), and there are others who have relied on the electrical analog method of solving the problems of three-dimensional flow.

The three most commonly adopted field methods of measurement are the piezometer cavity, the cased tube, and the auger-hole. Of the three methods which are compared in Figures 1, 2, and 3, the auger-hole method appears to be the least complicated.

20Ibid., p. 410.
Figure 1. The Piezometer, Tube, and Auger-hole Methods for Measuring Soil Permeability. (Redrawn from Reeve and Kirkham, Trans. AGU. 32:582-590. 1951.)

Figure 2. Relative Sample Sizes for Four Permeability Methods. Sizes for the Field Methods are the Volumes of Soil Through Which 80% of the Hydraulic Head is Dissipated. (Redrawn from Reeve & Kirkham, Trans. AGU. 32:582-590. 1951.)
The piezometer and auger-hole method both give a measurement that largely reflects the horizontal component of the hydraulic conductivity. The tube method more closely reflects a measurement of the vertical component of hydraulic conductivity.\textsuperscript{21} All three of these methods can be performed if there is a water table present in the immediate location of the soil for which hydraulic conductivity measurements are desired.

There are some distinct advantages of field measurements of permeabilities and of the particular method used. "Reeve and Kirkham (1951) showed that the effective sizes of sample associated with the small core (2-inch diameter by 2 inches long), a piezometer (1-inch diameter by 4-inch cavity), a cased tube (8-inch diameter with a cavity length equal to zero), and an auger hole (4-inch diameter by 30 inches deep), are in the ratio of 1, 35, 270, and 1400 respectively; . . . .\textsuperscript{22}

R. E. Winger, Jr., describes a method using ring infiltrometers to measure hydraulic conductivity in an area above the water table.\textsuperscript{23} One of limitations of this method is that the soil directly below the test zone must have an equal or greater hydraulic conductivity. When the test was performed a number of times it was arbitrarily determined that the test zone should be at least two feet above a less permeable layer. Where progressively tighter soils existed below the test zone, the steady-state conductivity was never reached.

\textsuperscript{21}Ibid., p. 413.

\textsuperscript{22}Ibid., p. 413.

\textsuperscript{23}R. J. Winger, Jr., "Field Determination of Hydraulic Conductivity Above a Water Table", A paper prepared for presentation at the Annual Meeting of the American Society of Agricultural Engineers; December 1956.
Figure 3. Schematic Flow Lines for Four Permeability Methods. (Redrawn from Reeve and Kirkham, Trans. AGU, 32:582-590, 1951.)

Figure 4. Infiltrometer Ring Method of Measuring Hydraulic Conductivity Above a Water Table. (Redrawn from R. J. Winger, paper presented to ASAE, Dec. 1956)
There are various indirect methods proposed for calculating soil permeability. The Kozeny-Carman equation related porosity and surface area to soil permeability. Aronovici and Donnan (1946) related the water transmission characteristics to soil texture. O'Neal (1949) proposed using texture, structure, and other characteristics for evaluating permeability. Later Uhland and O'Neal (1951) proposed the use of factors for field classification of soils as to their permeability. Pore size distribution has been proposed by various investigators.24 All of the proposed methods are relatively simple but require considerable skill and good judgement by the individual practicing them in the field. These proposed methods further illustrate the search for a simple and economical method of determining soil permeabilities.

**Determining Permeability from Field Measurements**

Basically all of the investigators have used Darcy's Law or some modification of it. They have taken the equation, \( v = k i \), and substituted for \( v \) in the continuity equation, \( Q = a v \), obtaining the relation

\[
Q = a k i
\]

(6)

where \( Q \) is the volume rate of flow, \( i \) is the hydraulic gradient, \( k \) the hydraulic conductivity, and \( a \) the cross-sectional area of the flow path.

In field determinations, \( Q \) can be obtained by measuring the volume of flow during an increment of time. The hydraulic gradient \( i \) is a relationship between the potential or head and the equivalent length of the flow path. The potential is measurable, however the length of the flow

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24 R. C. Reeve, _op. cit._, p. 413.
path would depend on the geometry of the flow pattern. Hence, it is most convenient to combine the equivalent flow cross-section with the equivalent flow path length into a composite factor, $A$. This factor $A$ would carry the dimensions of length.

The theory of seepage into auger-holes and measurements of soil permeability using auger-holes is fully discussed by Kirkham and van Bavel. They derived the potential function and then mathematically solved the problem of flow for the condition that an impermeable layer is located directly at the bottom of the hole. They have also indicated that a more practical solution of the flow problem for other conditions (no impermeable layer or impermeable layer at greater depths) could be solved using electric analog models.

Relating Darcy's flow equation to the theory of flow into an auger-hole, piezometer cavity, or a tube, certain modifications are necessary before permeabilities or hydraulic conductivities may be calculated. Within certain limitations it is relatively simple to measure (or calculate) the quantity of flow, the head causing flow, and the viscosity of the conducted fluid. However, it is necessary to solve the $A$ factor, either mathematically or by the electric analog method.

Since electricity flowing through a medium (electrolyte) is analogous to the flow of water from a high potential (source) to a lower potential (sink), this analogy can be used to solve complex three-dimensional flow problems. The electric current flow in a model will

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diminish in the ratio of the model to the corresponding field dimension. The current flow, the potential voltage drop, and the conductivity of the medium are all measurable quantities. The A factor can then be solved by the relationship

$$A = \frac{I_a}{I_m} \frac{r_m}{r} \frac{L}{\mathcal{C} E}$$  \hspace{1cm} (7)

where $I_a$ is current flow through the model, $r/r_m$ is the ratio of the model dimension to the field dimension, $\mathcal{C}$ is the conductance of the electrolyte (1/ohms-cms), and $E$ is the potential voltage drop across the model.\(^{26}\)

Investigators using the electric analog method have solved the A factors for the auger-hole method for practically any geometric pattern of flow into the auger-hole. Johnson, Prevert, and Evans\(^{27}\) presented a nomograph solution based on the following expression:

$$k = \frac{\Delta h \pi r^2}{4 t H A}$$ \hspace{1cm} (8)

where $k$ is the hydraulic conductivity in feet per hour or centimeters per second; where $(\Delta h \pi r^2)$ is the measured volume of water in the hole (radius $r$, height $\Delta h$); $At$ is the time increment; $H$ is the average potential or head during $At$; and $A$ is a factor (length) depending on the geometry of the flow pattern.


Although hydraulic conductivity is the more commonly used expression, it is convenient to compare air and water as the conducting fluids in terms of intrinsic permeability. The relationship of hydraulic conductivity to intrinsic permeability is expressed by

$$\bar{k} = \frac{k}{\gamma g}$$  \hspace{1cm} (9)

where \(\eta\) and \(\gamma\) are the viscosity and density of the specific fluid used, and \(g\) is the gravitational constant.\(^{28}\)

The equation for intrinsic permeability can be expressed as

$$\bar{k} = \frac{V \eta}{AT A_P A}$$  \hspace{1cm} (10)

where \(\eta\) is the viscosity of the fluid; \(A\) is a dimensional factor pertaining to the flow geometry; and \(V\) is the volume of fluid conducted through the medium in time interval \(AT\), at a differential pressure \(A_P\).\(^{29}\) This equation requires the proper corrections for viscosity changes due to temperature.

In measuring the permeability of a porous medium, the velocity of flow through a linear sample would be uniform along the entire length. This does not hold true for gases (or air). Muskat\(^{30}\) derives the following expression for permeability when the conducting fluid is a gas:

$$\bar{k} = \frac{D L \eta}{a(F_1 - F_2)}$$  \hspace{1cm} (11)

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\(^{28}\)Don Kirkham, and C. H. M. van Bavel, \textit{op. cit.}, p. 75.

\(^{29}\)Air Permeameter Notes, (mimeographed instructions for operation of the air permeameter possessed by South Dakota State College Agronomy Department).

where \( \overline{Q} \) is the volume outflow rate as measured at the mean pressure, \( L \) the flow path length, \( a \) the flow path cross-sectional area, and \( (P_1 - P_2) \) is the pressure difference, \( \Delta P \), across the length of the flow path. Using the air permeameter, \( P_2 \) would be the atmospheric pressure, and \( P_1 \) would be the applied pressure. The quantity \( \overline{Q} \) is not directly measurable, however the \( Q \) at pressure \( P_1 \) is a measurable quantity. The mean pressure, \( P_m \), is equal to \( (P_1 + P_2)/2 \). Since \( V/t \) is equal to \( Q \), and \( t \) is a constant time interval, the ratio \( \overline{Q}/Q \) can be also expressed by Charles' Law:

\[
\frac{V_2}{V_m} = \frac{P_m}{P_2}
\]

where \( P_2 \) is the atmospheric pressure in dynes per square centimeter, and where \( P_m \) is equal to \( P_2 \) plus \( \Delta P/2 \). Using air permeameter pressures where \( \Delta P \) is equal to 2000 dynes per square centimeter, \( V_2/V_m \) equates to 1.001. It can now be assumed, for practical purposes, that \( \overline{Q} \) is equal to \( Q \).

If the ground temperature is different from the temperature of the air transmitted through the medium, cooling or heating of the air will result. In measuring permeabilities in the field, it is more likely that the soil will be cooler than the air. It can be assumed that the pressure gradient will remain uniform or nearly so during the permeability measurement, and that the air temperature will approach the temperature of the medium almost instantaneously on contact. The coefficient of free expansion and contraction with pressure constant is 0.003671 per degree Centigrade.\(^{31}\) A 10 degree difference would result in approximately a 3.7 per cent change (due to expansion or contraction of air volume) in the

calculated permeability. When the soil temperature is less than the air temperature, a cooling of the transmitted air results. The measured volume would then be larger than the actual volume of air transmitted through the soil. In this case, the calculated permeability would be greater than the actual permeability by approximately 3.7 per cent per 10 degrees Centigrade temperature drop.

The cooling effect also causes a decrease in the viscosity of air. Lowering the temperature of the air transmitted 10 degrees Centigrade, results in a 2.7 per cent decrease in permeability due to a viscosity change. In this case, if the viscosity of air is figured at the initial air temperature, the calculated permeability would be greater than the actual permeability by approximately 2.7 per cent per 10 degrees Centigrade temperature drop. The total error per 10 degrees Centigrade temperature difference would now be approximately 6.4 per cent. Temperature corrections should be made whenever the soil is substantially cooler or warmer than the air temperature.

It can now be stated that equation (10) can be used, with some discretion, for calculating the permeability whether water or air is used as the conducted fluid.
CHAPTER IV

Discussion

The problem as presented in Chapter II is one of determining the hydraulic conductivity of a soil above the water table in situ. The desired features of the method to be adapted are:

1. May be conducted with relatively little expense.
2. May be conducted without time delays.
3. May be performed over extensive areas without great inconveniences.
4. May be conducted on a large soil sample in situ.
5. Can be easily adapted to measuring the horizontal as well as the vertical conductivity.
6. Must be reasonably accurate.

In reviewing literature and work of other investigators, it appears improbable that any method would possess all of the desired features. There are several approaches to the problem that could be used with varying degrees of accuracy. Calculating the permeability from some physical property of the soil is being deferred until the more direct measurement methods have been carefully investigated. This author believes that it may be difficult to quantitatively measure the macro-pore space and the natural fissures or openings of the soil in situ. It may also be difficult to relate these physical properties to the colloidal clay content of the soil to arrive at an acceptable permeability measurement.
Another approach to the problem would be to apply water to a small plot until a mound of water was actually established above a relatively impermeable layer of sub-soil. This procedure would work satisfactorily only where there exists an impermeable or relatively less permeable sub-layer below the soil to be measured. Its requirements are somewhat opposite those of the method described by Winger. If this water mound could be established, one of the field hole methods could be used to measure the hydraulic conductivity. One of the chief objections to adapting this procedure for field use would be its large water supply requirements.

Another procedure would be to measure the hydraulic conductivity above the present water table. Winger's method would fit into this class. Permeabilities can be measured with practically any fluid, using the proper dimensional units in Darcy's equation. Use of air as the conducting fluid should provide some value of permeability. Muskat states:

"... the permeability of a porous medium is a constant determined only by the structure of the medium and is independent of the nature of the homogeneous fluid passing through it, ..." [32]

Some of the advantages of using air are summarized as follows:

1. There will be little structural break-down of the soil due to repetition of the test.

2. There will be no problem of entrapped air as is the case with water.

3. The measurements can be easily obtained.

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4. The method can be readily adapted to measuring the horizontal as well as the vertical conductivity. Changes of air pressure accompanying slight changes of elevation are negligible, due to the low density of air. Unlike a liquid, which is greatly affected by gravitational forces, a gas will diffuse upward as readily as downward or horizontally. Air or gas flows directly from a higher pressure source to a lower pressure sink with a minimum of energy expended. Since air is of extremely low viscosity, the pressures required to produce laminar flow would also be very small. The result is that only relatively small volumes of air are required for the method.

The principle disadvantages of using air as a fluid are:

1. The uncertainty that all soil micro-pores are filled with moisture (air could move through micro-pores that normally would not conduct water).
2. The absence of passage obstructions normally caused by colloidal materials suspended in water.
3. The uncertainty that the hydration and soil swelling would be the same as when water is used as the conducted fluid.

Initial Field Trials

The first field trials initiated were an attempt to explore the possibility of building up a localized water table for measuring the hydraulic conductivity in situ. The three proposed methods of attempting establishment of the localized water table are as follows:
Method 1

Construct a six-foot depth trench around a minimum sized 12 by 12 foot island and install polyethylene cut-off membrane along the island side of the trench wall, backfilling behind the plastic cut-off. Construct a dyke along the cut-off and flood the island area.

Method 2

Construct a six-foot depth trench around a minimum sized 12 by 12 foot island. Construct a dyke along the outer perimeter of the trench and flood the entire trench and island area.

Method 3

Construct a shallow (one to two foot depth) trench and perimeter dyke around a minimum 12 foot diameter area. Flood the dyked area.

One each of the three methods was attempted in the preliminary trials, to determine which appeared to be the most suitable and economical method to be used for further investigation. A minimum number of wells were placed at all four sides of each plot area. One set (4) was placed 10 feet from the center of the plot, the other set 20 feet from the center. Upon satisfactory establishment of a localized water table, hydraulic conductivity measurements were made by the auger-hole method.

The initial field trials were attempted during June, 1959. The only suitable and readily available area at that time was located about 200 feet northwest of the Agricultural Engineering Building. The soil at this location is composed principally of glacial drift known as
Lismore silty clay loam. The permanent water table at the site was about six feet below the soil surface.

The deep trenching for Methods 1 and 2 were contracted to a local ditching contractor. The four-mil polyethylene cut-off membrane was installed, the ditch backfilled, and the dyke constructed with Agricultural Engineering Department labor. The estimated construction costs of the three methods are itemized as follows:

Method 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trenching</td>
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</tr>
<tr>
<td>Polyethylene</td>
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</tr>
<tr>
<td>Manual labor</td>
<td>$12.50</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>$36.10</strong></td>
</tr>
</tbody>
</table>

Method 2

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trenching</td>
<td>$15.60</td>
</tr>
<tr>
<td>Manual labor</td>
<td>$7.50</td>
</tr>
<tr>
<td><strong>Total</strong></td>
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</tr>
</tbody>
</table>

Method 3

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manual labor</td>
<td>$7.50</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>$7.50</strong></td>
</tr>
</tbody>
</table>

Additional labor was utilized to install the 1 1/4 inch diameter wells directly outside the dykes to study and compare the water table response. Additional expense was also incurred for water to flood the plots.

Figure 5. Plastic Cut-off Installed (Method 1) to Restrict Horizontal Seepage From Flot

Figure 6. Placement of the Plastic Cut-off Along the Island-side of the Perimeter Trench
A certain amount of equipment was necessary to facilitate obtaining precise measurements of the rise of the water inflow into the auger-hole. The equipment assembled for this purpose was modeled somewhat from the idea proposed by Hooghoudt. A fishing reel, a plastic covered aerial wire, a 45-volt dry cell, a 0-10 milli-ammeter, line resistors, and ground connections made up the electric circuit indicator. The equipment was mounted on a 14 inch by 20 inch plywood board equipped with scale rule, level tubes, clipboard, and fittings to attach the board to a standard survey plane-table tripod. A 3/4 inch electric conduit fitting was also attached to the board so that a piece of conduit could be extended from the board into the auger-hole to shield the measuring line from wind disturbances.

Some difficulty was encountered in getting water to the plots. As soon as water was available at the Agricultural Engineering Building, the problem was solved by using garden hoses to distribute the water. No individual measurements were made on the total quantity of water used by each plot; however, it was found that after the second day of flooding, a stream of about one-half gallon per minute was adequate for each plot. Flooding was continued for 10 days.

The water table was allowed to recede to about 1 1/2 feet below the soil surface. Auger-holes were constructed with a four-inch auger to the depth of 4 1/2 feet. Pump-out tests were conducted using the equipment illustrated in Figures 7 and 8. The pitcher pump with the garden hose and

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34C. H. M. van Bavel, and Don Kirkham, op. cit., p. 91.
screen filter attached to the suction and operated satisfactorily for evacuating the water from the holes.

Difficulty was encountered with the auger-hole tests. The holes were pumped, and as the water seeped into the hole some caving or sloughing occurred. It was readily apparent that the hole was enlarging at certain increments of depth. The tests indicated considerable error would result if the dimension of the holes kept changing during succeeding tests. Several preliminary pump-outs were performed before the actual test measurements were made. The extremely muddy water indicated that continual sloughing was taking place.
Two measurements of auger-hole inflow were made at each site. Only site three (Method 3) provided repeated results. The author does not believe that these results were caused by the particular flooding method, but strictly by chance.

Readings of the wells were taken at the time of the pump-out tests to determine the water table contours. The water table contours indicated there was considerable heterogeneity in the soil. The dispersion pattern of the water table between the plots indicated that the hydraulic conductivity was fairly good. The water contours at time of the pump-out tests are further illustrated in Figure 9.
Figure 9. Water Table and Ground Surface Contours of Plots in the Initial Field Trials
Daily readings were taken on the auger-holes to determine the decline of the water table at each plot. Generally, the water table receded at about the same rate in all three plots. No definite trend was established that would indicate the plastic cut-off was serving its intended purpose. Probably its failure to cause a slower decline in the water level within the plot can be attributed to its insufficient installation depth, and to seepage losses at the lap joints. Settlement of backfill material in the trench may have ruptured the plastic cut-off. Better installation procedures may yet prove the plastic barrier to be effective in containing the water table.

In briefly summarizing the initial field trials the following conclusions are offered:

1. Auger-hole tests in the initial trials were considered a failure because of sloughing into the auger-hole.

2. There appeared to be no superiority among the three methods in attempting to build up and temporarily retain a localized water table. A better procedure must be used in installing the plastic barrier.

3. A constant supply of water was necessary during the entire period of maintaining the water table regardless of the method.

4. It was evident from the water table contour map that the deep ditch around plots 1 and 2 did provide some equilibrium to the water table within the dyked areas.
Plan of Experiment

Based on the experience gained from conducting the initial field trials, it was decided to conduct additional field trials at a new location. The field trials would be located on the East Agronomy Farm and approximately 600 feet east of the farmstead buildings. The soil, Vienna loam, at this location is principally glacial drift.

It was also planned to repeat the auger-hole method of measuring hydraulic conductivity. Following the pump-out tests, and after the water table had declined, air permeability measurements would be made on the same auger holes. If some correlation were established, the air permeameter would certainly simplify field measurements of permeability.

To speed up the flooding procedure and reduce labor requirements, the field trials were set up with four replications of Method 3 used in the initial trials. It was planned to complete the water and air permeability measurements before severe cold weather threatened.

Instead of the water table indicating wells as used in the initial trials, 1/2 inch diameter piezometer tubes were installed. A pair of tubes were placed on each side of each individual plot. One tube would indicate the hydrostatic pressure at the nine-foot depth and the other at the four-foot depth. The water table level was maintained at the soil surface. The purpose of the piezometers was to check the hydrostatic pressure of the water table.

To conduct the air permeability measurements, it was necessary to construct an air permeameter instrument specifically for that purpose. Plans and specifications of permeameters in a paper by Grover were
valuable in constructing the air permeameter.\textsuperscript{35} It was necessary to make certain modifications to fit the size requirements of this investigation. Basically, the instrument is a float-can which is inverted in a reservoir of water. An outlet from the top of the reservoir tank transmits air to the auger-hole. The weight of the float-can provides a nearly constant pressure which causes the air to flow through the soil or medium. The instrument is illustrated in Figure 10. Calibration data and further details of the air permeameter are located in Appendix C.

The $A$ factor for air flow through an auger-hole is not the same as for water inflow into the auger-hole. For water inflow, the potential varies (due to gravitational effect on water) from the top of the water

table to the bottom of the hole. For air flow through an auger-hole, the potential (air can be considered weightless) can be considered the same from the top to the bottom of the hole. Therefore, the geometric flow pattern for the auger-hole method would change if air instead of water were used as the conducted fluid. To solve the A factor for this particular study, an electric analog tank and model was constructed. Using the relationship expressed by equation (7), the A factor for air flow through an auger-hole was obtained. Figure 11 illustrates the equipment used to obtain the values for calculating the A factor.

The A factor for the geometric pattern of flow of water inflow into the auger-hole was taken from Figure 12.
Figure 12. Graph for Use in Finding the A Factor for Water Inflow Into the Auger-hole. (Redrawn from H. P. Johnson, R. K. Frevert, and D. D. Evans, Agric. Engr., vol. 33:283-286. 1952.)
Field Procedure

Soil borings were taken on the available site area prior to setting up the plots. It was noted there was considerable variability in soil texture at different depths; however, no clearly defined stratification was observed. The water table was located at approximately the nine-foot depth.

The four plots were placed in a staggered double row with plots spaced 60 foot apart. This particular pattern of plot layout was used because it was desirable to keep all four plots on nearly the same elevation, and also to follow the general land topography of the available site area.

Dykes were constructed around each 12 foot diameter plot. All of the plowed topsoil from inside each plot was used to construct the dykes. Immediately following construction of the dykes, water was conducted to the plots with a portable irrigation pipeline and garden hose laterals. Flooding commenced on September 4, 1959 and continued through September 15. The water table was established and maintained at the soil surface. During the first two days, the four plots used a total of 22,000 gallons, or approximately two gallons per minute per plot. During the following eight days, the plots required 12,500 gallons, or approximately 16 gallons per hour per plot.

A pair of piezometers were installed along each side of each plot. Iron pipe of 1/2 inch diameter was driven into the ground with a pipe driving head and maul. A 5/16 by 2 inch carriage bolt was inserted in the bottom end of each pipe to prevent soil from jamming into the pipe. When driven to the desired depth, each piezometer pipe was pulled up about four
inches to extract the carriage bolt from the bottom end. All piezometers were then "flushed" with a jetting tool. A 10 foot length of 5/16 inch copper tubing fitted to a garden hose provided the hydraulic pressure. The jetting tool worked very satisfactorily, except when a fairly large pebble would wedge alongside the copper tubing and make it difficult to retrieve. All piezometers around the same plot were driven so that their tops were at a common reference elevation.

**Water Permeability Tests**

On September 15, a hole was augered in the center of each plot to a depth of 1 1/2 feet. The water table was maintained at ground surface without letting any water spill in directly from the top of the hole. Each hole was pumped several times prior to making inflow measurements.

Each hole was completely evacuated of water and measurements were made immediately by lowering the electric circuit indicator probe into the hole about 0.1 foot from the bottom of the hole. As soon as inflow water reached the probe, a stop watch was started and the probe raised 0.05 foot. Using two stop watches, it was relatively easy to measure successive increments of rise. All readings were taken before the water in the hole rose to the level where h/d exceeded 0.2; where h is the average depth of water in the bottom of the hole for any increment of rise, and d is the total depth of the hole below the water table. The temperature of the inflow water in each auger-hole was recorded immediately following

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Figure 13. Measuring Water Inflow Into Auger-hole.
(Water table was maintained at ground surface)

Figure 14. Flooded Plots Covered With Plastic Film
Upon Completion of Auger-hole Pump-out Tests
each test. All measurements of elevation were made to the nearest 0.01 foot. The average feet of head for each increment of rise of water into the hole was converted to dynes per square centimeter pressure for calculating \( k \), the intrinsic permeability.

All plots were covered with polyethylene plastic immediately following the pump-out tests. This was done to prevent excessive soil drying while the water table was receding.

Measurement of the static level of the water in each piezometer tube was taken using the same electric circuit indicator probe that was used for the auger-hole tests. It was evident that some of the piezometer cavities were located in impermeable material, as there was no appreciable change in the water level in some of the tubes since the time they were jetted.

**Air Permeability Tests**

In air permeability measurements on an auger-hole, the direction of flow would be reversed from the direction of water inflow into the auger-hole. Since the flow of a fluid through a porous medium is analogous to the flow of electrical current in an electrolyte, it should make no difference in the pattern of flow if the pressure source and sink were reversed.

A periodic field check indicated the water table had fallen to about three feet below the auger hole and that air permeability measurements could commence on October 14. Each plot was uncovered just prior to setting up the air permeameter. The air outlet was sealed to the top of the auger-hole with paraffin. It was found that a paint brush worked most satisfactorily for applying the paraffin seal.
Measurements were repeated three times at each auger-hole. Whenever the rate of fall of the float-can was quite slow, annular weights were added to increase the pressure, and additional measurements were taken. Results for different float-can pressures were obtained on most of the auger-holes.

The temperature of the soil and of the water in the permeameter at each measurement site was recorded. Soil samples for moisture determinations were also taken at each measurement site.
CHAPTER V

Results

Equation (10) was used to calculate the intrinsic permeability, \( \bar{k} \), for both water and air flow. Restating the equation:

\[
\bar{k} = \frac{V}{\Delta t \cdot \Delta p \cdot A}
\]

Calculation of Permeability from Water Inflow

The quantity \( V \), for permeability by water inflow measurements, was obtained by calculating the volume of an increment (0.05 foot) of auger-hole depth. Sloughing did make it difficult to measure the volume of the hole. A mean diameter was assumed for the hole from several measurements. The average head \( h \), for each increment of water rise in the auger-hole, was converted to \( \Delta p \) in dynes per square centimeter. The \( A \) factor was obtained from Figure 12 for the condition \( s/d = 2 \). The time increment \( \Delta t \), was obtained by the measurement of the time elapsed for each increment of rise. The viscosity of water, \( \mu_w \), was calculated from the equation

\[
\mu_w = \frac{.01779}{1 - .03368 T - .000221 T^2}
\]

where \( \mu_w \) is in poises and \( T \) is the temperature of the water in degrees Centigrade. 37

Calculation of Permeability from Air Flow

The quantity \( V \) is the volume of air forced through the soil for a measured increment fall of the float-can. The pressure \( \Delta p \) may be calculated from the unbuoyed weight of the float-can and the net cross-sectional area of the float-can. The time increment \( \Delta t \), is the elapsed time for each measured increment of fall of the float-can. The \( \Delta \) factor was obtained using an electric analog model. The viscosity of saturated air in poises, \( \mu_a \), is calculated from the equation

\[
\mu_a = 190.0 - 0.49(26.0 - T) \times 10^{-6}
\]

where \( T \) is the temperature in degrees Centigrade.\(^{38}\) Sample calculations and air permeameter calibration data are located in the Appendices.

Results

Intrinsic permeabilities as determined from auger-holes by air and water flow measurements are given in Tables I and II. Permeability by air plotted versus permeability by water in Figure 15 indicate that a relationship exists.

Moisture samples taken immediately following the air permeability measurements are tabulated in Table III. It was observed that the soil appeared very moist at the surface immediately after the plastic covers were removed. Some moisture condensation was noticed on the bottom side of the plastic plot cover. Earthworm activity was also noticed prior to conducting the air permeability tests.

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\(^{38}\) Air Permeameter Notes, (mimeographed instructions for operation of the air permeameter possessed by South Dakota State College Agronomy Department).
### TABLE I. AIR PERMEABILITIES: \( k \left( 10^{-8} \text{cm} \right) \)

<table>
<thead>
<tr>
<th>Increment of Rise</th>
<th>Hole #1</th>
<th></th>
<th>Hole #2</th>
<th></th>
<th>Hole #3</th>
<th></th>
<th>Hole #4</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ave.</td>
<td>Range</td>
<td>Ave.</td>
<td>Range</td>
<td>Ave.</td>
<td>Range</td>
<td>Ave.</td>
<td>Range</td>
</tr>
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<td>24.6</td>
<td>24.4-25.6</td>
<td>20.3</td>
<td>19.7-20.9</td>
<td>17.2</td>
<td>17.1-17.3</td>
</tr>
<tr>
<td>2</td>
<td>10.90</td>
<td>-----</td>
<td>24.6</td>
<td>24.4-24.8</td>
<td>20.4</td>
<td>19.8-22.4</td>
<td>17.1</td>
<td>-----</td>
</tr>
<tr>
<td>3</td>
<td>10.90</td>
<td>-----</td>
<td>24.8</td>
<td>24.6-25.3</td>
<td>20.7</td>
<td>20.0-22.5</td>
<td>17.0</td>
<td>16.7-17.3</td>
</tr>
<tr>
<td>4</td>
<td>10.95</td>
<td>-----</td>
<td>25.3</td>
<td>25.1-25.5</td>
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<td>20.5-20.8</td>
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<td>16.8-17.6</td>
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<td></td>
<td>20.5</td>
<td></td>
<td>17.1</td>
<td></td>
</tr>
</tbody>
</table>

*Average is taken of four trials of each increment.  
Mean is the mean of all trials and all increments at that particular auger-hole.*
## Table II. Water Permeabilities: $k \times 10^{-8} \text{ cm}^2$

<table>
<thead>
<tr>
<th>Increment of Rise</th>
<th>Hole #1</th>
<th>Hole #2</th>
<th>Hole #3</th>
<th>Hole #4</th>
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<td>Range</td>
<td>Ave.</td>
<td>Range</td>
</tr>
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<tr>
<td>3</td>
<td>0.89</td>
<td>0.87-0.91</td>
<td>2.14</td>
<td>1.93-2.27</td>
</tr>
<tr>
<td>4</td>
<td>0.83</td>
<td>0.80-0.85</td>
<td>1.91</td>
<td>1.82-2.01</td>
</tr>
<tr>
<td>Mean</td>
<td>0.88</td>
<td>1.98</td>
<td></td>
<td>1.46</td>
</tr>
</tbody>
</table>

*Average is taken of four trials of each increment.
Mean is the mean of all trials and all increments at that particular sugar-hole.
Figure 15. Permeability Measurements From Four Auger-holes
The piezometer readings on the outside perimeter of Plots 1 and 2 indicated that movement of water from the plot was primarily lateral. The nine-foot depth piezometers on Plot 1 did not come to equilibrium during the 30 days which indicates they were down in practically impermeable soil. Two of the four nine-foot depth piezometers on Plot 2 reacted in the same manner. All of the remaining piezometers indicated that water movement was primarily in a lateral direction from the plots. Readings of the 4 1/2 and nine-foot depth piezometers indicate that the equipotential lines were almost vertical at the edge of the plots. They also indicated that the equipotential lines were not perfectly symmetrical. Since there was no
particular water mound dispersion pattern trend among the four plots, soil heterogeneity seemed to be a plausible explanation.

There was considerable variability among successive inflow measurements. Some of this could be explained as sloughing or caving occurring between and during the successive measurements. It was difficult to determine the geometric dimensions (horizontal) of the auger-hole. An approximate diameter was determined from several measurements taken following the air permeability tests.

The air permeability measurements were relatively easy to obtain. Successive tests gave reproducible results. Changing the float-can pressure to speed the flow of air, did not significantly change the measured permeability $k$. Although air permeabilities did not coincide with the result obtained from the water inflow measurements, a definite relationship was indicated.
Summary and Conclusions

Summary

Initial field trials indicated that if an adequate supply of water was readily available, the installation of a six-foot depth plastic cut-off membrane around a plot did not greatly facilitate establishing and maintaining a localized water table. The plastic cut-off must also be more effective in containing the water table than indicated by the initial field trials, if the costs of installation are to be justified. No difficulty was encountered in establishing a localized water table without the plastic cut-off if the soil was underlain with an impermeable or progressively less permeable soil, and if an adequate supply of water was available.

Pump-out tests of the initial trials indicated that considerable sloughing or caving into the hole occurred wherever there were stones or rocks along the side of the auger-hole. Sloughing also occurred in some cases where soil lacked cohesion to support the vertical walls of the hole against the inflow of water. The auger-hole caving was reduced by decreasing the potential head causing inflow. Nevertheless, wherever a stone or rock was positioned in the wall of the auger-hole, invariably the hole caved after the first or second pump-out. As many as three and four holes had to be abandoned on Plots 3 and 4 before satisfactory auger-holes were obtained.

Differences in permeability as determined with water and air can be attributed to several causes. Any one or any combination of several
reasons may give a valid explanation of some of the differences. It is highly conceivable that the 10 day period of saturation was not sufficient to allow the entrapped air in the soil to come to equilibrium. A number of investigators have contended that the permeability of soil was greatly reduced by air that evolved from the saturating water. 39

It is also quite possible that the 30 day period between water and air permeability measurements may have resulted in some shrinking and dehydration of the clay particles. Colloids suspended in water may have had an immeasurable effect on the water inflow tests, which would not be present during the air flow measurements.

Earthworm activity noticed when the plastic plot cover was removed undoubtedly increased the rate of air flow.

The auger-hole A factor for calculating the permeability was not the same for air flow as for water inflow. Consequently, the soil sample on which the permeabilities were being measured, may not be identically the same.

In calculating permeability from water inflow into an auger-hole, the assumption was made that the water table could be considered static. Piesometers readings indicated that this assumption would be valid. However, in some cases the hydrostatic pressure may have changed at the time pump-out tests were being conducted. The permeabilities measured from a non-static water table (that was considered static) would tend to be smaller.

Conclusions

The following conclusions are offered from this study:

1. There appears to be no simple low cost method, possessing the desired features listed in Chapter IV of this paper, for measuring the hydraulic conductivity above a water table in situ. The air permeameter instrument offers promise if air flow permeabilities can be correlated with water flow permeabilities.

2. A localized water table can be raised if there is a readily available and adequate supply of water. However, there must also be an impermeable or progressively less permeable subsoil below the soil for which permeability measurements are desired.

3. A water table raised by flooding a small plot with water can entrap and compress air within the soil medium. This fact hints that a built-up water table should be maintained until most of the entrapped air has escaped.

4. The auger-hole method of measuring conductivities is not readily adaptable to all soils. Sloughing presents a major difficulty in obtaining water inflow measurements.

5. There was no evidence in the field trials of this study to indicate that a plastic membrane, if properly installed in a six-feet depth trench to contain an island of soil, would assist substantially to establish and maintain a localized water table.

6. The permeabilities as determined from air and water flow through an auger-hole did not coincide; however, there is adequate evidence to indicate that a relationship exists.

7. The air permeameter offers a relatively simple method of measuring field permeabilities. Its water supply requirements are small; only sufficient water to saturate the plot to field capacity is required. The measurements may be obtained from soil samples in situ. The air permeameter offers a relatively simple method of determining in situ if the soil is anisotropic.

This writer believes that further study and field trials are needed to establish the validity of using air permeability measurements for
determining the hydraulic conductivity of a soil above the water table in situ. The following suggestions are offered for continuation of study:

1. Additional field trials be conducted using the cased hole method of measuring permeabilities. The A factor of flow would be identical for air and water flow.

2. The soil should be saturated to field capacity followed immediately with air flow measurements taken at timed intervals. This would be to determine what degree of saturation, or soil moisture level, that air flow permeabilities most closely coincides with the water flow permeabilities.

3. Upon completion of air flow measurements, a water table be established and maintained at the soil surface to enable one to correlate the air flow permeabilities with water flow permeabilities.

4. Some method of de-airing the soil and saturating water be employed on some of the trials.

5. The air permeameter to be used to determine in situ if the soil is anisotropic. A suggested method is to first measure the air flow through the bottom of a cased hole, which would be more representative of the vertical component of conductivity. Then auger a cavity below the bottom of the casing and seal off the bottom of the cavity. The air flow through the sides of the cavity would now be more representative of the horizontal component of conductivity. A comparison of the results of several replications of the method should indicate the relationship between the vertical and horizontal components of conductivity.
LITERATURE CITED

Air Permeameter Notes, (mimeographed instructions for operation of the air permeameter possessed by South Dakota State College Agronomy Department).


Winger, R. J. Jr., "Field Determination of Hydraulic Conductivity Above a Water Table", A paper prepared for presentation at the annual meeting of the American Society of Agricultural Engineers: December 1956.
APPENDICES
APPENDIX A

Definition of Symbols

\( Q \)
volume rate of flow in cubic feet per second or cubic centimeters per second.

\( C \)
constant; characteristic of the porous media through which flow is caused.

\( a \)
cross-sectional area of the flow path.

\( L \)
length of the flow path.

\( H \)
energy or potential expanded to cause flow.

\( \mu \)
absolute viscosity of the fluid in poises.

\( \gamma \)
density of the conducted fluid, (grams per cubic centimeter).

\( v \)
average velocity of the fluid through the media; actually the quantity of fluid passing through a cross-section of medium during a unit of time.

\( d \)
the mean effective diameter of the pore size of the medium.

\( \Delta p \)
an increment of pressure expended in a flow column of length \( \Delta s \).

\( \Delta s \)
an increment length of flow column or path.

\( i \)
the hydraulic gradient in units of potential per length of flow path.

\( k \)
the hydraulic conductivity expressed in dimensions of a velocity; can be considered as the quantity of fluid transmitted through a cross-sectional area of flow path per unit of time.

\( k \)
intrinsic permeability expressed as the square of some dimension of length, \((L)^2\). Can be considered as the square of the mean effective pore diameter.

\( A \)
geometric factor (mean flux density of flow) of three-dimensional flow expressed as a dimension of length. Can be considered as the mean cross-sectional area of the flow path divided by the mean length of the flow path.

\( r_m \)
radius of cylinder (representing auger-hole) of the analog model.
\( r \) radius of the auger-hole in the field.

\( I_m \) current conducted through the analog model.

\( \sigma \) specific conductance of fluid used in analog model, \((\text{ohms-cm})^{-1}\).

\( E \) potential or voltage drop across analog model.

\( \Delta h \) increment of rise of water into auger-hole.

\( \Delta t \) increment of time during which an increment of rise \( \Delta h \) occurs.

\( g \) gravitational constant \((980.6 \text{ centimeters/second}^2)\)

\( V \) volume of fluid conducted through a medium during an increment of time \( \Delta t \).

\( P_1, P_2 \) initial and final pressures of flow through a medium.

\( Q \) air volume outflow rate measured at the mean pressure.

\( P_m \) mean pressure; also equivalent to one-half the sum of \( P_1 \) and \( P_2 \).

\( V_m \) corresponding volume of air at \( P_m \) conducted through a medium during an increment of time \( \Delta t \).

**Conversion Factors**

1 inch = 2.54 centimeters

1 foot = 30.48 centimeters

1 cubic foot = 28.32 \times 10^3 \text{ centimeters}

1 atmosphere = 1.0133 \times 10^6 \text{ dynes/cm}^2

1 foot of water = 29.891 \times 10^3 \text{ dynes/cm}^2

Converting intrinsic permeability to hydraulic conductivity of water at 20. degrees Centigrade:

\[ k/(1 \times 10^{-8}) = k/(3.5 \text{ cm/hr}) \]
Terms

Dyne: The equivalent unbalanced force acting on a mass of one gram giving it an acceleration of 1 centimeter per second per second. Can also be expressed as:

\[
\frac{gm}{cm^2 sec^2}
\]

Poise: An arbitrary unit of viscosity; also expressed as: \(\frac{dyne \cdot sec}{cm^2}\)

or as:

\[
\frac{gm}{cm \cdot sec}
\]
APPENDIX B

Basic Formulas and Sample Calculations

Intrinsic Permeability by air flow:

\[ \bar{k} = \frac{V}{\Delta t \cdot \Delta p \cdot A} \]

where \( V \) is the increment of volume passing through the medium of viscosity \( \mu_a \) during the period of time \( \Delta t \), and at pressure (dynes/cm²), \( \Delta p \). Factor \( A \) is the geometric factor of the particular three-dimensional flow pattern.

Sample Calculation:

\[ V = 3330. \text{ cm}^3 \]
\[ \mu_a = 185.6 \times 10^{-6} \text{ poises} \]
\[ A = 273. \text{ cm} \]
\[ \bar{k} = \frac{(3330. \text{ cm}^3)(185.6 \times 10^{-6} \text{ poises})}{(15.3 \text{ sec})(860. \text{ dynes})(273. \text{ cm})} = 17.2 \times 10^{-8} \text{ cm}^2 \]

Intrinsic Permeability by water inflow:

\[ \bar{k} = \frac{V}{\Delta t \cdot \Delta p \cdot A} \]

where \( V \) is the increment of volume inflow into the auger hole during the period of time \( \Delta t \), and pressure (dynes/cm²), \( \Delta p \). Factor \( A \) is the geometric factor for the particular three-dimensional flow pattern for water of viscosity, \( \mu_w \).
Sample Calculation:

\[
V = \Delta h \pi r^2 = (1.542 \text{ cm}) \pi (5.58 \text{ cm})^2 = 152. \text{ cm}^3
\]

\[
\mu = 0.00975 \text{ poises}
\]

\[
\Delta p = 1.385 \text{ feet} = 0.0414 \times 10^6 \text{ dynes/cm}^2
\]

\[
A = 99. \text{ cm}
\]

\[
\Delta t = 39 \text{ seconds}
\]

\[
\overline{k} = 0.93 \times 10^{-8} \text{ cm}^2
\]

Converting intrinsic permeability to hydraulic conductivity of water at
20 degrees Centigrade:

\[
\frac{\overline{k}}{1 \times 10^{-8}} = \frac{k}{(3.5 \text{ cm/hr})}
\]

\[
k = \frac{(3.5 \text{ cm/hr})(0.93 \times 10^{-8})}{(1 \times 10^{-8})} = 3.25 \text{ cm/hr}.
\]

Sample Calculation of absolute viscosity of water: \( T = 20^\circ \text{ C} \)

\[
\mu_w = \frac{0.01779}{1 - 0.03368T - 0.000221 T^2}
\]

\[
\mu_w = \frac{0.01779}{1 - 0.6736 - 0.0884} = 1.01 \times 10^{-2} \text{ poises}
\]

Sample Calculation of viscosity of saturated air: \( T = 17^\circ \text{ C} \)

\[
\mu_a = [190 - 0.49(26 - T)] \times 10^{-6} \text{ poises}
\]

\[
\mu_a = [190 - 0.49(26 - 17)] \times 10^{-6} = 185.6 \times 10^{-6} \text{ poises}
\]
APPENDIX C

Air Permeameter Calibration

$\Delta P$ = dynes/cm$^2$ pressure above atmosphere pressure
Can area = 666. cm$^2$

<table>
<thead>
<tr>
<th>Reading (cm.)</th>
<th>Can Weight (gms.)</th>
<th>$\Delta P$ (dynes/cm$^2$)</th>
<th>Average $\Delta P$ (dynes/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>597</td>
<td>878</td>
<td>860</td>
</tr>
<tr>
<td>5</td>
<td>575</td>
<td>846</td>
<td>830</td>
</tr>
<tr>
<td>10</td>
<td>553</td>
<td>814</td>
<td>800</td>
</tr>
<tr>
<td>15</td>
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<td>765</td>
</tr>
<tr>
<td>20</td>
<td>508</td>
<td>748</td>
<td></td>
</tr>
</tbody>
</table>

Can + 300 grams weight

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<th>Reading (cm.)</th>
<th>Can Weight (gms.)</th>
<th>$\Delta P$ (dynes/cm$^2$)</th>
<th>Average $\Delta P$ (dynes/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>1320</td>
<td>1305</td>
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<tr>
<td>5</td>
<td>875</td>
<td>1287</td>
<td>1270</td>
</tr>
<tr>
<td>10</td>
<td>853</td>
<td>1255</td>
<td>1235</td>
</tr>
<tr>
<td>15</td>
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<td>20</td>
<td>808</td>
<td>1188</td>
<td></td>
</tr>
</tbody>
</table>

Can + 599 grams weight

<table>
<thead>
<tr>
<th>Reading (cm.)</th>
<th>Can Weight (gms.)</th>
<th>$\Delta P$ (dynes/cm$^2$)</th>
<th>Average $\Delta P$ (dynes/cm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1196</td>
<td>1762</td>
<td>1745</td>
</tr>
<tr>
<td>5</td>
<td>1174</td>
<td>1728</td>
<td>1710</td>
</tr>
<tr>
<td>10</td>
<td>1152</td>
<td>1695</td>
<td>1680</td>
</tr>
<tr>
<td>15</td>
<td>1129</td>
<td>1662</td>
<td>1645</td>
</tr>
<tr>
<td>20</td>
<td>1107</td>
<td>1630</td>
<td></td>
</tr>
</tbody>
</table>

Can + 965 grams weight

<table>
<thead>
<tr>
<th>Reading (cm.)</th>
<th>Can Weight (gms.)</th>
<th>$\Delta P$ (dynes/cm$^2$)</th>
<th>Average $\Delta P$ (dynes/cm$^2$)</th>
</tr>
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<td>0</td>
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</tr>
<tr>
<td>5</td>
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<td>2270</td>
<td>2250</td>
</tr>
<tr>
<td>10</td>
<td>1518</td>
<td>2235</td>
<td>2215</td>
</tr>
<tr>
<td>15</td>
<td>1495</td>
<td>2200</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>1473</td>
<td>2170</td>
<td>2185</td>
</tr>
</tbody>
</table>

* 0 reading coincides with the float-can at the highest position.

For each additional gram weight placed on top of the float-can, add
1.472 dynes/cm$^2$ pressure to the average $\Delta P$ for each increment of volume,
V.
AIR PERMEAMETER
RESERVOIR CAN

1" o.d. smooth iron pipe
Markings spaced 1 cm. apart

Handles replaced
on side of can

Top of 10 gal.
milk can removed

1/2" nipple

5/16" to air outlet

Handles from
neck of can

Float
guide

15"

12 1/2"

2 3/8" o.d.
iron pipe

2 1/2" 4"

11"

65
AIR PERMEAMETER
FLOAT CAN

10. gage aluminum plate

\[ \frac{1}{8} \text{ i. d. aluminum conduit} \]

2\(\frac{1}{2}\) gage aluminum

AIR OUTLET SUPPORT

\[ \text{set screw} \]

2\(\frac{3}{8}\) o.d. iron pipe

2\(\frac{3}{4}\) i. d. iron ring

sheet metal

anchor legs (3)

\[ \frac{1}{2} \text{ rod} \]

6"

5"
APPENDIX D

Methods Used For Determining Permeabilities in the Lake Plains Area

Disturbed Permeability

Disturbed samples were obtained with a regular barrel-type soil auger or from sections of an undisturbed core taken with the Utah sampler. Most of the samples used for disturbed permeability tests were taken with the Utah sampler, because better depth control could be obtained and there was less mixing of soil samples from the lower depths with material scraped off the sides of the hole as the auger was pushed down the hole. Samples from each depth were placed in quart cartons to insure that none of the fines would be lost in transporting the sample from the field to the laboratory. The apparatus used and the procedure followed in the laboratory for making a disturbed permeability test is outlined in the Reclamation Manual - Vol. 5 - 2.10.8G.

Undisturbed Permeability

Undisturbed soil samples were collected to the 10-foot depth for laboratory permeability analysis with the Utah sampling machine. Since there had been some criticism about the method of collecting and preparing the samples for the laboratory tests a slightly different procedure was established for the samples taken on the East Lake Plain. When the soil cores were taken in the field they were packed at once in permeameters instead of being encased with cheesecloth and cellulose acetate. These permeameters were then transported only a short distance, usually less than four miles, to a field laboratory where the undisturbed test was run. By this method the test was being run in the laboratory within two or three hours from the time the sample was taken in the field. This insured that the sample did not dry out and that the transportation damage was kept at a minimum.

A brief description of the procedure used in preparing the sample in the field is as follows:

As soon as the casing and core was removed from the sampler it was placed on two sawhorses and the outside casing removed. The core was then measured and examined and any portion that was broken or distorted was rejected. No core shorter than six inches or longer than 14 inches was packed. If the core was found satisfactory it was carefully rolled onto a spare heavy split soil tube.

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lined with wax paper and supported by three sawhorses. To preserve the moisture while the lower half was being packed, the upper half of the core was covered with a dampened cheesecloth, the waxed paper was taped shut around the core, and the section was covered with a five-foot length of sawdust.

The permeameters were either metal or plastic tubes or cylinders 4 1/2 to 5 inches in diameter with a concave bottom in which a 3/8 inch hole was drilled. The length of each tube varied from 16 to 18 inches. To keep the material from falling out of the 3/8 inch hole in the bottom, a one inch square of stiff nylon curtain material was placed over the hole. Fine gravel, 1/4 to 1/2 inch thick was placed on top of the nylon curtain material and a number 11 filter paper was placed over the gravel. The tubes were prepared for packing by one crew member while the others were drilling the core.

The selected length of core was then carefully cut away and separated from the rest of the core with a sharpened spatula. A masonite circle was then placed over the top of the core and a piece of stiff curtain material two inches wide and approximately 36 inches long was centered over the bottom of the core and brought up the sides and over the top where the ends were wrapped together until the proper tension was obtained to lift the core without cracking it. One hand was placed at the top and the other at the bottom of the core and it was lifted off the trough and placed inside of the permeameter tube, care being taken to center it as closely as possible. The cloth was then unwrapped and cut off as short as possible without disturbing the core. The masonite top was then removed and a thin rod was used to push the remaining cloth to the bottom of the tube. One half to one inch of crude cotton batting was then tamped around the core at the bottom of the tube. This was to keep the sand-bentonite mixture which was then poured around the core from going down and plugging the nylon screen. A special device similar to an inverted funnel was placed over the top of the core to prevent any of the bentonite from getting on top of the core. MX-80 bentonite was used at the rate of one part bentonite to four parts fine sand passing a 14-mesh screen. Sufficient sand-bentonite mixture was added to bring it to within approximately one inch of the top of the core in the tube. A rod was then used to pack the sand down and additional sand and bentonite was added until it was again within one inch of the top of the core. Fine gravel was then poured on top of the sand-bentonite mixture until it was even with the top of the core sample.

A number 12 filter paper was placed on top of the core and the packed permeameter was then placed in a specially prepared box on a pickup truck. Immediately after the last section of the core was packed all of the permeameters were taken to the field laboratory which was located only a few miles away.

As soon as the permeameters arrived at the field laboratory they were placed in a pan of water. They remained in this pan until capillary action had brought the water to the top of the core.
The permeameter was then removed from the pan of water and placed on the permeability rack. A two-inch head of water was used and the test allowed to run until such time as the permeability rate became constant. The time required to reach a constant permeability rate varied from three days to two weeks.

Field Permeability Tests

Field permeability tests were made on 16 of the 37 sites in the East Lake Plain and on six sites in the West Lake Plain.

Two tests were made at each site. The shallow test was usually within the top three feet and the deep test between five and seven feet. The section to be tested was selected after a careful study of the disturbed and undisturbed permeability data and soil texture at that site, and was usually at a depth where there was an appreciable change in the permeability rate.

The outside hole was excavated with a Buda earth auger, a 42-inch diameter helix bit, and a nine-inch frost point. Each hole was augered to within 9 to 12 inches of the level at which the test was to be performed. The remaining 9 to 12 inches of soil were carefully removed with a small hoe and a short handled shovel to avoid compaction or disturbance of the horizon to be tested, and also to make certain that the exact depth was reached. A carpenter’s level was used to check the bottom of the hole to be sure that it was level before the cylinder was placed in the hole. A steel cylinder, 18-inches in diameter and 20-inches in length was placed in the middle of the 42-inch hole and driven exactly six inches into the soil, after which the soil around the inside and outside of the ring was carefully tamped to insure a good seal. Burlap sacks were placed on the surface inside of the ring to dissipate the force of the water as it was applied in order to avoid disturbing or puddling the soil.

There were two criticisms brought up in connection with calling this test a permeability test instead of an infiltration test. One was that the tension from the drier soil below and laterally from the bottom of the cylinder exerted a force on the water moving through the six-inch test zone, thereby increasing the amount of water moving through the test area. To check this tension while the test was being conducted, two tensiometers were installed on opposite sides of the cylinder about three to four inches outside of the cylinder. The porous cup of each tensiometer was installed approximately three inches below the bottom of the cylinder. For the first two or three days these tensiometers had readings of 200 to 500 cm of water. As the soil below and laterally from the cylinder became saturated the tension gradually reached zero which indicated that the area immediately surrounding the bottom of the cylinder was near a saturated state. This saturated state brought up the second criticism, which was that when the wet front reaches an impermeable or a less permeable layer than that being tested a mound of water will start forming. When this mound has risen into the six-inch test area the hydraulic gradient will be reduced to less than unity, which results in not only a reduction in the movement of water through the six-inch
test zone, but also presents a question on what value to use for "h" in the calculations for permeability. To check the possibility of a mound forming under the test area, two piezometers were installed on two sides of the 18-inch diameter cylinder, and to a depth three inches below the bottom of the 18-inch cylinder. As long as the tensiometers indicated zero tension and the piezometers showed the entire column to be at atmospheric pressure, which are the circumstances under which water will not rise in piezometers placed in the saturated zone, the resulting rate of water movement through the six-inch test zone should approach the permeability rate.

To complete the installation a calibrated tank was connected to a carburetor float attached to the side of the 18-inch cylinder. This float could be raised or lowered to maintain any head of water desired. For these tests a six-inch head was used. All water coming from the calibrated tank was run through a fine mesh so that the carburetor needle valve would not become fouled during the test. This screen was cleaned at least once every day while the test was being run. The tank was placed at the ground surface and secured to a post driven into the ground to keep it from falling over during a high wind. Metal covers were placed over the 18-inch cylinder to prevent rainfall entering the ring and also to slow down evaporation losses.

At the beginning of the test, measured amounts of water were added to the area inside the ring to bring the water surface up to the six-inch head after which the float was put into operation. Water was also added to the area outside of the ring to bring the level up to the six-inch depth, but the amounts added were not measured or recorded. The six-inch head on the outside of the ring was not maintained throughout the test, but water was added two or three times each day. This helped to saturate the soil laterally from the test zone and reduced the length of time that the test required to reach a steady state.

The test was continued until a constant rate was obtained. The time varied from one to three weeks, depending upon the permeability of the soil. After the test was completed the 18-inch rings were dug out and the soil column broken off so that the material remaining inside the ring could be examined and a brief description made of any cracks, worm or root holes or any other channels that might have had some influence on the permeability rate.