USE OF THE VELOCITY-HEAD PERMEAMETER TO DETERMINE THE SATURATED HYDRAULIC CONDUCTIVITY AND FOR SEPTIC SYSTEM SITE EVALUATION

By

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A THESIS

Submitted to
Michigan State University
in partial fulfillment of the requirements
for the degree of

MASTER OF SCIENCE

Department of Agricultural Engineering

1988

ABSTRACT

USE OF THE VELOCITY-HEAD PERMEAMETER TO DETERMINE THE SATURATED HYDRAULIC CONDUCTIVITY AND FOR SEPTIC SYSTEM SITE EVALUATION

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The Velocity-head permeameter (VHP) is a relatively new instrument, developed by Dr. George Merva Agricultural Engineering Department Michigan State University, to measure the saturated hydraulic conductivity (K_s) in situ. Currently few methods exist that are capable of measuring the K_s in situ without extensive supportive equipment. The VHP is fully portable and can be used in remote areas quite easily.

The VHP was compared to the constant-head outflow method in the laboratory on the same undisturbed cores to determine if a correlation between the two methods exists. Further consideration was given to the applicability of the instrument for routine use during site evaluation for on-site wastewater soil absorption systems. The two methods show a very good correlation demonstrating that the VHP will accurately predict the results of the constant-head laboratory method. Usefulness of the instrument during site evaluation for on-site wastewater soil absorption systems was determined to be good when site limitations such as small layers or inclusions and general soil horizons that may be expected to yield low K_s values, are encountered.

Approved: George E. Merva
Major Professor

Approved: Donald M. Edwards
Department Chairperson

In loving memory of my father **Forest J. Rose** who always looked toward my accomplishments with great pride always believing in me no matter what the circumstances. Through his patient teaching I gained a practical knowledge that could not be taught in any classroom. Though he did not live long enough to enjoy this accomplishment with me, I know that one day we will share our joy together again in heaven.

> " These things I have spoken to you, that in Me you may have peace. In the world you have tribulation, but take courage; I have overcome the world."

The Words of Jesus (John 16:33)
New American Standard Bible

ACKNOWLEDGMENTS

I would like to express my appreciation to my major professor Dr. George E. Merva for his guidance, encouragement, and assistance throughout my research and studies for my program. His methods of teaching and research have been instrumental in shaping my approach to education.

I also appreciate the help and guidance offered by others on my committee, Dr. Ted Loudon and Dr. Del Mokma. Their help both in the office and in the field proved very beneficial to my education.

The help of Bruce Grunner and Mark Banghardt from the Ingham County Health Department was essential to the collection of data in the field. Likewise the encouragement of John Long of the State of Michigan Health Department was very helpful. I would also like to thank John Harvey for his encouragement and patience in the development of the production model of the velocity-head permeameter.

Finally I would like to thank all my colleagues for their emotional and spiritual support during my studies at MSU especially Cindy Phelps, my chief editor, and Kirk Dolan, my statistical editor.

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I. INTRODUCTION

With the large number of rural homes in Michigan, septic wastewater disposal systems, Comprised of a septic tank for primary treatment followed by a soil absorption system for purification and disposal, have become increasingly important. The redistribution of the population to the rural area coupled with increasing environmental concerns has led to the State of Michigan requiring more stringent control of design and installation of these systems. Each site or location for potential septic systems must be considered as a separate design problem based on several established criteria. The goal of such careful consideration is to insure safe working systems that will provide adequate treatment of wastewater for several years with minimum maintenance.

Bouma (1974) defined two important terms to describe a good working system, disposal and treatment. Disposal is defined as placing the effluent into the ground in such a way that it is never allowed to surface, whereas treatment occurs underground as a result of filtration, adsorption, and oxidation which together reduce the contaminants in the wastewater. In order to provide adequate systems that provide environmental protection, treatment becomes a critical design factor.

Although all criteria used as standards for site inspection are important, the hydraulic conductivity of the soil stands out as being critical in developing a good system. Hargett, Tyler, and Siegrist (1982) pointed out that the saturated hydraulic conductivity is of little use when proposing a design except through the use of some empirical design constraints. These authors discuss the development of a clogging mat as being a factor which reduces the saturated flow to a lower, unsaturated value. Ideally then, the unsaturated hydraulic conductivity would be most desirable as a design parameter. The crust method as presented by Bouma et al. (1971) is suggested as being capable of determining the unsaturated conductivity in situ. Otis et al. (1977a) pointed out the fact that to use this method requires a highly trained technician and a large amount of time making it an economically unfeasible method. Since economics plays such an important role in site inspection and design, other methods of soil evaluation must be considered. The determination of the saturated hydraulic conductivity (K_s) is much less expensive and can be used in empirical equations to determine the expected unsaturated hydraulic conductivity. Gardner (1958) and Fritton, Stahl, and Aron (1982) have developed such empirical relationships for determining the unsaturated hydraulic conductivity given the saturated hydraulic conductivity and comparisons of the soil with other similar soils already evaluated.

For many years in Michigan the percolation test was used to determine saturated hydraulic conductivity. Not until recently was the accuracy of the percolation test questioned. McGauhey and Winneberger (1964) found the percolation test to yield wide variations between test locations only a meter apart. Mokma and Whiteside (1973) demonstrated a seasonal effect on the results of the percolation test and suggest that to prevent this the soak period of the test must be extended. Davis and Prince (1984) defined the percolation test to be a crude approximation of the hydraulic capabilities of the soil. Another drawback of the method is the time required for this test making it quite costly to use on a individual site basis (Bouma, 1971). Currently, the State of Michigan has removed the requirement of a percolation test for site inspections by county sanitarians. Even though the requirement of the percolation test has been removed it is still used periodically to overrule county sanitarians at the local government level.

In place of this test, the soils are examined from soil maps, borings and/or back-hoe cuts. Although for most soils this method is adequate, there are many situations in Michigan were the soils may be questionable. Such sites are usually disallowed (failed) and thereby turned over to private consultants for further investigation. Mokma and Whiteside (1973) suggested that the soil mapping unit, used to create soil maps, makes it difficult to predict the expected saturated hydraulic conductivity for small wastewater tile fields. The authors state that the scaling of these maps is such that the validity of the corresponding conductivity estimates are questionable when applied to areas as soil absorption systems.

Considering the limitations mentioned above, it seems essential that in situ measurements of the saturated hydraulic conductivity (K_s) be taken for soils that have special limitations or are questionable. Fritton, Ratvasky, and Petersen (1986) suggested there is no instrument to replace the percolation test. A review of the current methods available for in situ determination of the saturated hydraulic conductivity reveals two very adaptable methods, the Guelph Permeameter and the Velocity-head Permeameter. Of these two, the Velocity-head Permeameter (VHP) was suggested as having small time and water requirements to complete tests (Merva, 1987).

The objectives of this study are to: 1) establish the reliability of the Velocity-head Permeameter compared to the constant-head laboratory method for the range of soils tested and 2) investigate the application of the Velocity-head Permeameter to site inspection of potential soil absorption systems for septic tank effluent.

In this thesis the term permeability (k) is taken to mean the intrinsic permeability of a porous media. Therefore, it is defined as a parameter which is a function of the media itself and not a function of the viscosity of the liquid flowing through that media. The permeability (k) is related to the hydraulic conductivity (K) by the term f (fluidity) where,

$$K = kf. \tag{a}$$

For this study the fluidity term (f) for distilled water is approximately equal to one over the range of conditions encountered during testing. Therefore, the permeability (k) and the hydraulic conductivity (K) are considered synonymous. Finally the hydraulic conductivity (K) is a function of the water content of the soil and would be expected to attain some maximum value when the soil is saturated (based on Darcian behavior). The term saturated hydraulic conductivity (K_s) is defined as this maximum value at soil saturation.

The soil absorption system is defined as the secondary part of the septic wastewater disposal system involved in purification and disposal. Design of the soil absorption system requires some knowledge of the hydraulic conductivity of the soil in and around the proposed system. Therefore the soil's saturated hydraulic conductivity (K_s) for the proposed soil absorption system in this study was the major concern during site evaluations. Furthermore, the terminology used to describe the soil absorption system tends to vary significantly from the researcher to the installer. Terms commonly used to describe the soil absorption system include septic waste distribution fields; waste tile distribution fields; leach beds; leach fields; leaching systems; home septic waste systems; septic systems; etc. In order to provide continuity the term soil absorption system is used to refer to this secondary part of the septic wastewater disposal system.

NOTE: Since many of the reviewed papers used different units, the use of SI units was overshadowed by the need to allow better comparisons (e.g. length dimension (cm) and hydraulic conductivity dimension (cm/hr)).

II. LITERATURE REVIEW

A. Site Requirements for Soil absorption Systems.

The State of Michigan, along with many other states, has established requirements and guidelines for the granting of permits for septic waste water disposal system installation. The purpose of such guidelines was reported by Bohunsky et al. (1977) to be the disposal of human wastes in such a way that it will not: 1) contaminate any present or future drinking water supply, 2) give rise to public health hazard or present the potential for hazard, 3) pollute or contaminate surface or groundwater, 4) give rise to nuisance, odor, or unsightly appearance, and 5) violate laws or regulations governing water pollution or sewage disposal. With such purposes in mind, further inquiry into these guidelines is necessary to understand the importance of hydraulic conductivity in waste water absorption by soil. McKeague, Wang and Topp (1982), described the saturated hydraulic conductivity as an important parameter for evaluating the potential uses of the soil, while Bouma (1974) and Bouma et al. (1975) further described the lack of permeability data as being a serious problem in evaluating site suitability.

1. Site Evaluations

According to Magner (1984), there is no cookbook approach to site evaluation. Rather each site evaluation tends to require its own approach. Many of the criteria used in site evaluation have been established through system monitoring and modeling of commonly found soil situations in the field (Bouma,1974). Although the use of the soil survey technique is useful for evaluating soils with no or slight limitations, a problem tends to develop as the limitations for a given soil type increase. According to Bouma's definitions of disposal and treatment it is quite simple to dispose of waste water, but much more difficult to adequately treat the same waste water. Further discrepancies arise as to what is adequate treatment. Tyler et al. (1977) described purification as the elimination of fecal indicators and adequate reduction in concentrations of suspended solids, BOD, nitrogen, and phosphorous. Bouma suggested that standards for purification have not been rigidly established and have tended to be diverse depending on the compound being considered. He also mentioned the fact that soils tend to vary in their ability to adsorb certain elements and compounds. Through his research he established broad minimum criteria for soil suitability: (1) the hydraulic capacity of the soil must be greater than 2.54 cm/hr, (2) a minimum of 90 cm of unsaturated soil should be present between the bottom of the system and a high groundwater table or bedrock (note: water table is strongly affected by season), and (3) limiting site characteristics such as excessive slope and location within a flood plain should be avoided. Machmeier, Hansel, and Anderson (1982) included the following six items for site evaluations in the State of Minnesota as cited from the "On-site Sewage Treatment Manual":

1. Depth to highest known water table or bedrock.
2. Soil conditions, properties, and permeability.
3. Land topography or slope.
4. Existence of lowlands, surface depressions, rock outcrops.
5. All legal setback distances maintained from wells, lakes, streams, buried pipes and lines, etc.
6. Surface water flooding probability.

The State of Michigan also uses these same items in its consideration of site evaluations, (Bohunsky et al., 1977).

a. Soil Profile Evaluation

Although all six of these items are equally important, the soil conditions are critical to both the design and function of any system (Anderson et al., 1977). Otis et al. (1977a) listed several factors affecting soil permeability to include; a) soil compaction, b) size and continuity of soil pores, c) peds formed by repeated wetting, drying and freezing, and d) aggregation or channeling formed by soil fauna activity and plant roots. When considering a site for waste water treatment the soil strata must be carefully examined for soil conditions that could adversely affect the flow of waste water through the soil and eventually lead to possible system failure. In support of this, Otis et al. listed poor site evaluation as one cause of system failure. The guidelines of Machmeier, Hansel, and Anderson (1982) for soil borings when evaluating a site are as follows:

Soil Boring Considerations

a) Borings should at least extend three feet deeper than proposed bottom of the system.

b) Record soil texture by depth and note where changes occur.

c) Record the highest known water table by noting the presence of mottling. If no mottling occurs in clay or heavy loam, then let the test hole stand 16 hours and record depth to standing water.

The state of Michigan, to further assist in identifying soils, also adds the use of soils maps and optional excavations to investigate the soil horizon (Bohunsky, 1977). Otis et al. (1977b) described the use of soil maps to be helpful in determining relative capacities of soils to transmit liquid, but limited in their ability to quantify the actual K_s. Anderson et al. (1977) stated that National Soil Maps could not be used for site evaluation due to limited site specific accuracy. Research carried out by Conta, Richardson, and Prunty (1985) demonstrated that field values of permeability fall outside ranges given by SCS soil map data. The hydraulic conductivity has also been described by Wagenet, Knighton, and Bresler (1984) and Nassehzadeh-Tabrizi and Skaggs (1983) to vary several orders of magnitude in a relatively small field. Another problem of the soil maps is their inability to describe limitations due to management practices. Bouma and Hole (1971) demonstrated the presence of compacted layers due to extensive agricultural usage and a consequent reduction of the saturated hydraulic conductivity in these layers. This aforementioned research demonstrated that this compacted layer may occur at different depths, e.g. 20-60 cm for a clay soil and 50-90 cm for a silt loam.

b. Determination of the Hydraulic Conductivity

According to Bohunsky et al. (1977), in order for a designed system to function properly, the rate of infiltration and percolation must exceed the rate of application. Conditions found in the soil during septic waste water disposal can, according to Bouma (1974), be entirely predicted based on the hydraulic conductivity and moisture retention data. The saturated hydraulic conductivity according to McKeague, Wang, and Topp (1982), is important in determining succeptability to ponding and runoff as well as defining the adequacy for septic soil absorption.

The ability of the soil to absorb or transport liquid has been commonly evaluated by the percolation test (Otis et al. 1977a). Machmeier (1985) established guidelines for the use of the standard percolation test as follows:

Percolation Test Guidelines

a) Test hole dimensions and locations
   (i) Each test hole 15 to 20 cm diameter, with vertical sides dug or bored to depth of the bottom of the proposed system.
   (ii) Soil texture descriptions should be noted as well as changes.

b) Preparation of the test hole
   (i) Bottom and sides should be carefully scratched to reduce any smearing.
   (ii) Remove all loose material and add 5 cm of 0.6 to 1.9 cm diameter gravel to prevent surface sealing and scouring.

c) Soil saturation and swelling
   (i) Fill hole to 30+ cm of water and maintain level for at least 4 hours.
   (ii) Allow hole to swell for 16-30 hours except in sand

d) Percolation rate measurements
   (i) Measure water drop from 20 cm reference for 10 minute intervals in sands or 30 minute intervals in heavy soils.

e) Calculation of rate
   (i) Divide time by the water level drop ($\Delta t/\Delta H$).
   (ii) Average all rates for final design value.

f) Submit worksheets for reporting purposes.

g) Do not perform percolation tests where frost exists below the depth of proposed system.

At the present time, many states use the standard percolation test to determine site acceptability, while other states (e.g. Maine) have abolished its use due to certain discrepancies that can be found in the validity of such a test (Bouma, 1971; Hoxie and Frick, 1984). Machmeier, Hansel, and Anderson (1982) also stated the percolation test is merely a measure of the rate of drop of water in a given diameter test hole and asserted that it does not measure the rate of water movement through the soil. Davis and Prince (1984) go so far as to say that the percolation test is a crude approximation of the hydraulic abilities of the soil. An attempt has been made to relate the results of the percolation test to the hydraulic conductivity (Fritton, Ratvasky, and Petersen, 1986). They suggested this attempt to be necessary due to the lack of an instrument to replace the percolation test. Bouma (1971) demonstrated statistically that the coefficient of variance for the falling head percolation test averaged 54% and the coefficient of variance for the constant head percolation method averaged 35%. The soils used ranged from clay loam to silt loam to loamy sand. In a study by Conta, Richardson, and Prunty (1985) wide spatial variations demonstrated that the precision of the percolation test was low. Otis et al. (1977a) concluded that the percolation test may vary in the same soil as much as 50%, indicating a need for a more accurate method of testing. Furthermore, Bouma found that the percolation test when compared to the double tube method, yielded significantly higher values for the saturated hydraulic conductivity. Further limitations develop from the fact that most data require a great deal of time to obtain and sufficient data are not available to make adequate estimates of all soils (Bouma, 1974).

In light of these limitations of the standard percolation test, it becomes desirable to consider other methods of determining the saturated hydraulic conductivity to enable complete site evaluations. These methods include laboratory measurements, estimation based on other similar site data, and in situ determination through morphologic classification and actual measurement of the hydraulic conductivity. Magner (1984) depicted the conductivity values obtained from laboratory cores as being conservative. Other researchers have also pointed out as well that undisturbed cores are actually "relatively" undisturbed cores (Hillel, 1982). Comparisons of similar soils through the use of soil maps is defined by Davis and Prince (1984) to be useful "background information", although Topp, Zebchuk, and Dumanski (1980) noted an individual soil series to be highly variable with respect to the hydraulic conductivity. They have also suggested that only in the last decade has much attention been given to measuring and recording field values. Soil Interpretation Records used for estimation of the hydraulic conductivity are, for the most part, estimates and reflect very few actual measurements (King and Franzmeier, 1981). They suggest that more in situ determinations be made to enable better estimations in the future. Also the study by Conta, Richardson, and Prunty (1985) mentioned earlier, depicted permeability to at times, fall outside predicted ranges on SCS records. Given this variability, consideration should be given to methods which determine the hydraulic conductivity in situ.

i. Hydraulic Conductivity by Morphologic Classification

Classification of soil groups, based on morphologic parameters were compared to the in situ hydraulic conductivity as measured by the piezometer method, King and Franzmeier (1981). The spatial variability of their results demonstrated the need for more extensive testing of soil horizons. Their saturated hydraulic conductivity data was grouped into classes based on texture, origin of parent materials, and horizon development. This resulted in dividing the loam texture, massive structure class into three classes; water-worked till, compact till, and other C horizons yielding K_s ranges of 28 to 99.17, 0.02 to 0.28, and 0.03 to 11.21 cm/hr respectively. Soils with similar groupings tested in other locations seemed to fit into the range of hydraulic conductivity fairly well given the wide ranges of K_s. Hydraulic conductivity measurements were found to vary widely in apparently similar soil horizons.

McKeague, Wang and Topp (1982) developed a method for better estimation of the saturated hydraulic conductivity using soil morphology. Their findings concluded that the major factors contributing to high saturated conductivities were the presence of biopores, textures coarser than loamy fine sand, and strong, fine to medium blocky structure. They grouped the morphological features into eight classes and soil horizons were then classified in the field according to these classes. This was later compared to actual saturated hydraulic conductivities obtained using the air entry permeameter. The results of the study demonstrate that soils finer than loamy fine sand must be carefully evaluated according to their structure to best estimate their saturated hydraulic conductivity. The authors also suggest that for many soils morphology by itself does not appear to be a usable criterion. A further conclusion of this study was the need for more field values of the saturated hydraulic conductivity for use in comparisons. Conta, Richardson, and Prunty (1985) have further concluded that there is no clear relationship between percolation rate and any measured soil parameter for soils with greater than 0.35 kg/kg clay content.

ii. In Situ Measurement of the Hydraulic Conductivity

Anderson et al. (1977) reviewed field tests commonly used in conjunction with site evaluation. All of the methods presented by the authors are reviewed in section B of the literature review. A major limitation of most methods, presented by McKeague, Wang, and Topp (1982), is the large amount of time required to test or evaluate any given soil. As pointed out in the following section on system operation, consideration should be given to both the vertical and horizontal components of the hydraulic conductivity. With these points in mind the field method used to determine the hydraulic conductivity should be considered carefully from both an accuracy and economic standpoint (Nassehzadeh-Tabrizi and Skaggs, 1983).

2. System Operation

In order to properly design a quality waste disposal system, considerations must be given to the potential conditions which could be found in the system after years of operation. These conditions could be entirely predicted based on the hydraulic conductivity and moisture retention data (Bouma, 1974).

If considering the use of K_s for design purposes, Hillel and Gardner (1969, 1970) reported that an impeding layer at the top of an infiltrating profile may prevent the subsequent saturation of the strata below. It is commonly understood that anaerobic conditions encourage the formation of organic compounds from microorganisms thereby creating a clogging mat (Otis et al., 1977a). The process by which this clogging mat develops was subdivided by these authors into the following phases:

Phase	Condition
1	Compaction, puddling and smearing of soil during construction.
2	Puddling caused by constant soaking of soil during operation.
3	Blockage of soil pores by solids filtered from waste effluent.
4	Accumulation of biomass from growth of microorganisms.
5	Deterioration of soil structure by exchange of ions on clay particles.
6	Precipitation of insoluble metal sulfides under anaerobic conditions.
7	Excretion of slimy polysaccharide gums by some soil bacteria.

Although mat formation occurs aerobic bacteria decompose many organic solids which keeps soil pores open, but this happens only when periods of drying and aeration take place (Otis, 1984). Bouma (1971) also demonstrated that if aerobic conditions are reached in the soil, organic compounds can be oxidized and a increase in infiltration through the impeding layer would result. Therefore, Otis et al. (1977b) suggested that when designing a system for the distribution of septic effluent into the soil the unsaturated hydraulic conductivity should be considered. Bouma suggested that a precise K value on which to base a design is difficult to determine, since little is known about the formation and subsequent removal of these clogging mats. Not only is the unsaturated vertical conductivity below a trench system important, but as Otis et al. (1977a) pointed out, a clogging mat would produce ponding in the trench and thereby cause more extensive use of the side walls through horizontal conduction or flow. Pask et al. (1984) have also discussed horizontal conductivities of upper soil layers in slowly permeable soils to be significant in maintaining a good operating system. In conjunction with this, Wang, McKeague, and Topp (1985) demonstrated that in some soils the horizontal and vertical hydraulic conductivities may vary by a factor of 9.

The desire for a value of the unsaturated hydraulic conductivity point to the need for field methods which make in situ determinations of this parameter both in the horizontal and vertical directions. Jaynes and Tyler (1984) concluded that no existing methods for evaluating the unsaturated hydraulic conductivity are economically feasible for routine site evaluations. Otis et al. (1977a) stated specifically that the crust test method is not economical from the standpoint of standard site evaluation. They remarked that the unsaturated conductivity is a function of tension and follows trends based on similar soil types. Although it is difficult to determine the exact expected conductivity, Bouma (1971) measured soil moisture tensions ranging from negative 30 to 40 millibars for operating systems below a clogging mat under continuous effluent application. Fritton, Stahl and Aron (1981) suggested the use of some empirical model to determine the unsaturated hydraulic conductivity. This model was based on similar soil classifications, soil moisture tensions and the field measured saturated hydraulic conductivity values.

The facts surrounding both site evaluation and system operation suggest that if hydraulic conductivity is to be measured as a soil property for evaluation and design, it should, ideally, be measured in both the horizontal and vertical direction. Since, as pointed out earlier by Jaynes and Tyler (1984), no economical methods exist for determining the unsaturated hydraulic conductivity, methods of measuring the saturated hydraulic conductivity should be considered. The question then posed is, "What method or methods, to be used in the field in conjunction with site evaluation and system design, could accomplish this economically and accurately?"

B. In Situ Methods for Determination of K_s

Several methods exist for determining the saturated hydraulic conductivity (K_s) of a porous media in situ both above and below a water table. More commonly used methods below a water table include the auger-hole, piezometer, multiple well, and measured drawdown (Nassehzadeh-Tabrizi and Skaggs, 1983). Methods used above a water table include: twin ring (Scotter, Clothier and Harper, 1982), double tube (Bouwer, 1961), ring and shallow well permeameter (Winger, 1965), air-entry permeameter (Bouwer, 1966), and in situ falling head permeameter (Sommerfeldt and Chang, 1980; Bouwer 1966). Other methods have also been developed which tend to be refinements of previously mentioned methods such as the Guelph Permeameter (Elrick et al., 1984) and the Velocity-head Permeameter (Merva, 1979). Bouwer and Jackson (1974) have reviewed the more commonly used methods for in situ measurement of the hydraulic conductivity.

1. Below Water Table Techniques

a. Auger-hole Method

The auger-hole and piezometer methods have been described by Winger (1965) to be methods by which conductivity is determined from the rate of water entry into a cavity below the water table, following water removal from that cavity. The auger-hole method is considered to be the easiest and simplest of the in situ procedures (Nassehzadeh-Tabrizi and Skaggs, 1983). Winger defines the actual test to measure the average horizontal conductivity of the soil profile from the static water table to a small distance below the bottom of the hole. Talsma (1960), Boersma (1965a), and Buckland, Harker, and Sommerfeldt (1986) all reported on the use of different field methods for determining hydraulic conductivity. They found that the auger-hole method yielded conductivity values similar to those determined from actual performance of drainage systems.

Diserens (1934) first applied this technique and developed the equation for the hydraulic conductivity (K) to be:

$$K= (233/Ht) \log(y_o/y_t) \tag{1}$$

where,

K = Hydraulic conductivity in m/day
t = time in minutes
H = distance in meters between water table surface and hole bottom
y_o = distance in meters between water table and water level in hole at t equal to zero
y_t = distance in meters between water table and water level in hole at t equal to time t.

Ernst (1950), used relaxation techniques to numerically solve for the flow around the auger-hole. This enabled a reduction of the K equation to:

$$K = C(\Delta y/\Delta t) \tag{2}$$

where,

C = coefficient determined from nomographs
Δy = change in water level for Δt
S = Shape factor chosen based on hole conditions
Δt = time increment
r = radius of the auger-hole

The analysis of the solution was then expressed in the form of monographs from which C could be read as a function of y,H,r, and S. Other more recent developments are a series of exact mathematical solutions for variable depths to an impermeable layer, (Boast and Kirkham, 1971). The volume of soil tested with this method is suggested as being approximately 0.4H m^3, (Bouwer and Jackson, 1974), where H is the height of the water table from the bottom of the test hole. Procedures for field use of the auger-hole method are referenced in van Beers (1958).

Winger (1965) described two conditions which decrease the accuracy of values obtained by the auger-hole method: 1) very low or high permeability rates and 2) proximity of the water table to the soil surface and, Olson and Daniel (1981), suggested further that the auger-hole method could only be used in moderately permeable soils due to the slow rise of the water table in less pervious soils. Van Bavel and Kirkham (1949) noted that if the water table is significantly low inaccurate measurements would result. Boersma (1965a) found that soils with small sand lenses which drain quickly yield erroneous results. Talsma (1960) described two more conditions which could influence the auger-hole method's accuracy: 1) A possible existence of a cone of depression around the hole. 2) Difficulty in choosing shape factors for each condition. Soils which are non homogeneous, non isotropic or have variable K_s values are difficult to evaluate with this method (van Beers, 1970). Boersma reported measurements from the auger-hole method to vary by as much as 100% between holes just a few feet apart. He also pointed out the difficulty of preparing holes and taking measurements in rocky soils.

b. Piezometer Method.

The piezometer method is similar to the auger-hole method, but is considered to be a point measurement (Buckland, Harker, and Sommerfeldt, 1986). Winger (1965) described the usefulness of this method in measuring horizontal permeability of thin layers below a water table. Talsma (1960) also noted the piezometer method yielded more accurate results in layered soils. Luthin and Kirkham (1949) described the hydraulic conductivity, which governs flow into a small diameter pipe installed as a piezometer, by the equation:

$$K = \frac{\pi r^2}{At} \ln \frac{y_o}{y_t} \tag{3}$$

The terms are similar to those used in the equation for the auger-hole and A has the dimension of length and is dependent on the geometry of the system. Youngs (1968), is credited with the use of an electric analog to determine A for various effects of impermeable layers. The volume of soil effectively tested is approximated to be 1 dm^3, (Bouwer and Jackson, 1974). The practical field use of this method is referenced by Winger (1965). Bouwer and Jackson described the measurement obtained by this method to be horizontal or vertical depending on the specific geometry of the installed piezometer.

Potential limitations of the piezometer method may be the same as those associated with the auger-hole method (Talsma, 1960). Winger (1965) noted a problem tube sealing in coarse soils. He further states that less permeable layers of 10 to 12 inches thick existing between more permeable layers do not yield reliable results with the piezometer method. Boersma (1965a) mentioned the limitation of using graphs in place of calculations as not providing exact solutions with this method. He further suggests that the presence of slow permeable layers and the necessity of cavities of proper dimensions make field applications of the method difficult.

c. Multiple Well Technique

The two-well technique proposed by Childs (1952), consists of two small diameter wells located about one meter apart. Water is pumped from one well into the other until a constant gradient between the two wells is maintained. The hydraulic conductivity is then calculated, as referenced by Bouwer and Jackson (1974), based on the equation:

$$K = \frac{Q}{(\pi)(\Delta H)(H+L_f)} \cosh^{-1} \frac{D}{2r} \qquad (4)$$

where,

Q = the pumping rate
ΔH = equilibrium water level difference
D = the distance between the center of the wells
r = the radius of the well
H = the water depth in the hole
L_f = end correction factor

Later the two well method was extended to a radial symmetrical array of wells by Smiles and Youngs (1963). In this method the wells are arranged on the circumference of a circle in an alternate pumping and receiving fashion. The hydraulic conductivity equation per Bouwer and Jackson (1974) then becomes:

$$K = \frac{Q}{n\pi(\Delta H)(H+L_f)} \quad \ln \frac{4D}{nr} \qquad (5)$$

where,

Q = becomes the total flow rate in the system
n = number of wells
other parameters same as above

Snell and van Schilfgaarde (1964) used an electric analog to analyze this system and thereby simplified the equation for the hydraulic conductivity based on the geometric conditions of the wells. As the number of wells increases the soil sample volume increases as well to enable a "field" measurement of the horizontal component of the hydraulic conductivity, (Bouwer and Jackson 1974).

Limitations of the multiple well technique may include problems similar to those described under the auger-hole approach. Further difficulties arise due to sealing of the bottom of the receiving well. Kirkham (1954) proposed the use of two additional wells to observe the piezometric head between the two wells in overcoming the problems of sealing. Bouwer and Jackson (1974) state that heterogeneity in a horizon can create some problems in the multiple well method and hence produce an averaging of values. These authors also reported a dependence of the correction term, L_f, on the capillary fringe, as the water level increases above the water table and unsaturated soil conditions prevail.

d. Drainage Method

Several equations have been developed as models of water flow to a drain tube. Of these, one of the most popular is van Schilfgaarde's transient equation. The equation was developed to determine drain spacing given the physical parameters of the soil and not necessarily to determine the physical properties, (van Schilfgaarde, 1974). DeBoer (1979) demonstrated the ability of this equation to determine the hydraulic conductivity of the soil. The equation developed by van Schilfgaarde to solve for S and transformed by DeBoer to solve for K, is of the form:

$$K = \frac{fS^2}{9td_e} \ln \frac{M_o(2d_e+M)}{M\,(2d_e+M_o)} \qquad (6)$$

where,

K = the saturated hydraulic conductivity
f = the drainable porosity
S = the drain line spacing
M_o = water table height above drain line at time zero
M = water table height above drain line at the end of the time interval
t = time interval in days
d_e = equivalent depth to impermeable layer below the drain line

Equation 6 has been pointed out by the author to be effective in some field studies, providing that the physical properties of the soil be known to enable accuracy of the equation. Limitations pointed out by Nassehzadeh-Tabrizi and Skaggs (1983) are the increased requirements for time and equipment over other methods. Bouwer and Jackson (1974) described the measurement of the hydraulic conductivity using the drainage method as being most effective for determining a effective field K but, they further mentioned the method to be generally impossible for common use.

All of the methods described above for measuring the hydraulic conductivity are limited by the necessity of a water table above the soil sample to be evaluated. van Bavel and Kirkham (1949), pointed out that many times such measurements could only be performed a few times a year. This leaves the option of artificially producing a high water table in the soil or to explore other methods devised to measure the hydraulic conductivity above a water table.

2. Above the Water Table Techniques

a. Twin Ring Method

One method for determining the hydraulic conductivity at or near a surface is by the twin ring method, (Scotter, Clothier, and Harper, 1982). This method begins with the equation developed by Philip (1969) for modeling steady-state infiltration from a circular pond. Through linearization of Philip's equation by the use of a term called the matric flux potential (Φ), a solution can then be arrived at for the flux density (q) into the soil, (Raats, 1971). Where,

$$q = \alpha(\phi_s) + 4(\phi_s) / (\pi r) \qquad (7)$$

and by the assumption of the saturated hydraulic conductivity being greater than the unsaturated, an approximation can be made such that,

$$\phi_s = (K/\alpha) \tag{8}$$

and equation 7 can then be written as:

$$q = K + 4(\phi_s) / (\pi r) \tag{9}$$

Finally, if two simultaneous rings are used equation 9 can then be solved simultaneously using two rings to determine K by:

$$K = (q_1 r_1 - q_2 r_2)/(r_1 - r_2)\ . \tag{10}$$

Scotter, Clothier, and Harper (1982) described a deviation from the assumption that α is independent of the matric potential by showing the resulting value of the hydraulic conductivity to be less than the actual saturated value. The authors tested the method on three soils at different locations and compared the results to soil cores taken near field test locations. The results demonstrated a close correlation between the ring and core method for two of the three soils which were sand to sandy loam. The third soil that did not demonstrate a close correlation was a silt loam. This indicates a problem of anisotropy, as mentioned by the authors. Vertical hydraulic conductivities may be expected to be greater in the sandy loams, whereas the horizontal hydraulic conductivities may be expected to dominate in the silt loams (Harr, 1962). Scotter, Clothier, and Harper (1982) stated that the theory may not apply to cases with low conductivity crusts over samples. They also note that in some cases the steady-state response of the flux density is unattainable.

b. Double Tube Method

Bouwer (1961) developed another method for measuring the hydraulic conductivity in situ known as the double tube method. Boersma (1965b) declares this method to be free of simplifying assumptions used in other methods. The method, as described by Bouwer and Jackson (1974), saturates a limited soil region below an auger-hole in which two concentric tubes are located. The hydraulic conductivity is then calculated based on the rate of change of water in the inner tube while the outer tube water level is held constant. Conceptually, the outer tube serves to maintain "saturation" in the soil and aides in obtaining a vertical hydraulic conductivity by setting up a saturated boundary. The inner tube is then inserted into the bottom of the auger-hole to delineate the soil sample. Hence a balance can then be obtained on the water input versus the water output, which is used to determine the saturated hydraulic conductivity. Through the use of a resistance network flow factors were established for various soil conditions to be expected in the field (Bouwer, 1961). Through the simultaneous measurement of flow rates and water level differences in situ, the proper flow factor can be selected based on a similar geometry to that depicted in the laboratory. Bouwer determined the equation by which the hydraulic conductivity is determined as:

$$K = \frac{2.3R_v 2}{R_c F_f t} \log \frac{H_o'}{H_t'} \tag{11}$$

where,

K = saturated hydraulic conductivity
R_v = radius of measurement tube for inner tube
R_c = radius of inner tube
t = elapsed time
H = difference in water levels between tubes (positive or negative)
H_b = H at balanced flow
H' = difference in water level in inner tube and balanced flow level ($H-H_b$)
H_o' = $H-H_b$ at initial time (t=0)
H_t' = $H-H_b$ at some time t
F_f = dimensionless flow factor determined experimentally from a flow resistance network.

This procedure if used at the soil surface simply reduces to a buffered cylinder infiltrometer. Another special case of the instrument's use is when the underlying soil has a much higher conductivity the inner tube functions as a permeameter (Bouwer, 1961). The volume of soil that is tested would depend on the dimensions of the inner tube but is approximately 3 dm^3 (Bouwer and Jackson, 1974). Bouma and Hole (1971) found that this method measured K in the vertical direction if the soil structure was predominately coarse prisms and measure K in the horizontal direction if the soil structure was predominately small peds. Bouwer and Jackson define the method required hole size to be from 20.3 to 106.7 cm in diameter, required water to be about 200 liters, and a required time of 2 to 3 hours to complete each test. Bouwer (1961) assumed the area below the inner tube to be significantly saturated to a depth of at least $2R_c$ (twice the radius of the inner tube). This assumption was further extended to suggest that suction head below this would have no effect on the conductivity determination. Bouwer and Rice (1967) defined the requirement of the outer tube diameter to be 2 times the diameter of the inner tube although, in theory, a factor of only 1.7 is suggested. They attributed this difference to soil disturbance.

Boersma (1965b) found values obtained by this method to compare favorably with laboratory values.

The limitations of this method are similar to those associated with the percolation test and other tests that require boring a test hole in the soil. The auger-hole must have sand or stone placed in the bottom to avoid sealing at the soil water interface, (Boersma, 1965b). A special method is also needed to overcome the problem of smeared side walls in the test hole (Bouwer, 1962). Bouwer (1961) also identified the problem of isolating a leakage flow component for any given value of head and the difficulty of predicting the time to saturation. Finally, Conta, Richardson, and Prunty (1985) suggested that the double tube method is not well suited to routine use and possesses disadvantages for use in determining site suitability for septic waste disposal.

c. Ring Permeameter Method

Winger (1965) defined the ring permeameter method as a specialized measurement of the vertical permeability of some critical zone in question. Winger (1960) described the exact procedure used when taking field measurements. He further defined the calculation of the saturated hydraulic conductivity based on Darcy's equation of the form:

$$k = \frac{VL}{tAH} \qquad (12)$$

where,

k = the permeability in inches per hour

V = volume of water passed through the soil in cubic inches
A = the cross-sectional area of the test cylinder in square inches
t = time in hours
L = the length of the soil column in inches
H = the height of water above the base of the ring in inches.

The hydraulic conductivity is related to the permeability (k) by the term called fluidity (f) a correction term for fluid properties (Hillel, 1982), so that

$$K = kf, \tag{13}$$

where k is a property that is intrinsic to the porous media while the hydraulic conductivity can be a function of the fluid used. Since water is the principle fluid used, the permeability and conductivity are commonly used synonymously although, corrections must be made if the viscosity of the water changes significantly, (Winger, 1965). The ring permeameter method was later modified by Bouwer for use with two concentric tubes in an auger-hole, (Bouwer and Jackson, 1974).

The principle limitation described by Winger (1960) is the necessity of equal or greater permeability directly below the test hole. The procedure calls for the use of tensiometers to overcome this problem. Furthermore, unless tensiometers are used to satisfy the requirements of Darcy's law (a pressure gradient of one) the results can be expected to depend on the unsaturated boundary conditions (Bouwer, 1961). In the procedure, as described by Winger (1965), it is required that the soil be tamped along the cylinder to prevent flow along the wall. This might be considered to be a disturbance of the in situ sample. Another limitation described by Winger was the large quantity of water needed to complete each test. Boersma (1965b) listed the difficulty of use in rocky soil and deep permeable soils to be a further limitation of this method. The test method also requires a great deal of time, not only for installation, but also to obtain zero pressure at the tensiometers (Bouwer and Jackson, 1974).

d. Shallow-well Permeameter or Pump-in Method

Yet another method developed for the determination of the saturated hydraulic conductivity is the shallow well permeameter method. The procedures are described in detail by Winger (1965) and Boersma (1965b). Zanger (1953) developed two equations for determining the conductivity based on the depth to the impermeable layer:

for $S > 2H$,

$$K = \frac{Q}{2\pi H^2} \left(\ln \left\{ \frac{H}{r} + \left(\frac{H^2}{r^2} - 1 \right)^{0.5} \right\} - 1 \right) \qquad (14)$$

and for $S < 2H$

$$K = \frac{3Q}{(\pi H)(3H+2S)} \ln \frac{H}{r} \qquad (15)$$

where,

Q = flow rate at equilibrium conditions
H = the depth of water in the auger-hole
r = the radius of the auger-hole
S = the depth from the auger-hole bottom to the impermeable layer.

Further evaluation by Boersma (1965b), has led to the development of nomographs to aid in rapid determination of the saturated hydraulic conductivity in the field. Bouwer and Jackson (1974) described this method as measuring predominately the horizontal component of the saturated hydraulic conductivity. The measured horizontal value is further stated by Winger (1965), to be a average value of the auger-hole with exceptions for layers of relatively high permeability. Buckland, Harker, and Sommerfeldt (1986) compared three methods for determining the hydraulic conductivity and found the constant-head well permeameter method to give the best results compared to drainage trial values. DeBoer (1979) demonstrated differences in K_s between the pump-in and auger-hole methods. He also compared these two methods to the drainage method, the result was that a composite profile value obtained with the pump-in method compared favorably with that obtained using the drainage method.

Some limitations of the shallow well method relate to those of the double tube method in that care must be taken to protect the soil surface. Talsma (1960), Winger (1965), and Boersma (1965b) all suggested that this method yields lower values than several other commonly used methods, including the auger-hole method. Talsma attributed these errors to approximations in the well permeameter method or to puddling affects giving rise to surface sealing. Winger and Boersma defined further limitations to include time, water, and equipment needed to accomplish the test. Winger stated that the hole must be augered carefully to yield accurate results, a task which can be quite difficult in the field. Boersma finally suggested that the results of this method reflect the conductivity of more permeable layers as opposed to an overall average.

e. Air-Entry Permeameter Method

Another method, the air-entry method, was developed by Bouwer (1966) to determine the saturated hydraulic conductivity using Darcy's equation and data obtained from infiltration at high pressures. Bouwer and Jackson (1974), Aldabagh and Beer (1971), and Bouwer (1966) have all described both the equipment and its use in the field. The major difference between this method and others as stated by Bouwer is the use of the velocity of the falling water column as it infiltrates into the soil. The determination of the hydraulic conductivity is based on the equation:

$$K = \frac{(dH/dt)\ (R_r/R_c)^2\ L}{H_t + L - 0.5\ P_a} \tag{16}$$

where,

K = half the value of the saturated hydraulic conductivity due to entrapped air.
dH/dt = rate of fall of water level in reservoir prior to closing supply valve.
H_t = height of water level in reservoir after closing supply valve.
R_r = radius of reservoir.
R_c = radius of cylinder.
L = depth of wetted front when supply valve is closed.
P_a = air-entry value of soil as evaluated from minimum pressure head inside cylinder after closing supply valve.

Equation 16 is based on one-dimensional flow; Bouwer and Jackson (1974) stated that the wetted front must not leave the cylinder to enable such an assumption. Topp and Binns (1976) recognized increased speed, accuracy, and lower water requirement as making this method more desirable over other methods. Aldabagh and Beer (1971) described the procedure as requiring two men to operate the instrument, approximately one hour to run a test, and about one gallon of water being necessary for each reading. Topp and Binns suggested one limitation of this method as being the difficulty in determining the depth to the wetted front. The authors incorporated a small pencil size tensiometer into the system to evaluate the depth of the wetted front. This addition is described as beneficial but difficult due to certain soil conditions that may lead to tensiometer breakage. Aldabagh and Beer further note this method to be a point measurement and describe methods to help overcome such a limitation in the field.

f. In Situ Falling-head Permeameter

A final method developed to measure the in situ hydraulic conductivity is the falling-head permeameter (Sommerfeldt and Chang, 1980). This method is simply an extension of the falling-head laboratory procedure used to determine the saturated hydraulic conductivity of soil cores. The difference between the laboratory and field methods is the use of a head driven into the soil. The laboratory method is used to measure the vertical hydraulic conductivity of soil cores (Klute, 1965), whereas the field method was adapted to measure only the horizontal conductivity (Sommerfeldt and Chang, 1980). The equation for the hydraulic conductivity is given by Klute as:

$$K = (al/At) \ln(H_1/H_2) \qquad (17)$$

where,

a = the cross-sectional area of the manometer
l = the length of the soil sample
A = cross sectional area of soil sample
H_1 = the head at time zero
H_2 = the head at time = t
t = time from start to finish.

Sommerfeldt and Chang (1980) described in detail the field procedures required to use this method for the determination of K_s. They further described the necessity of system stabilization before readings can be taken, which may take approximately one or more hours to read. This need for stabilization was attributed to the contact of the soil sample with the extended soil horizon, whereas in the laboratory method the soil sample is a core with no soil boundary interface. Buckland, Harker, and Sommerfeldt (1986) described the results from the in situ use of the falling-head permeameter to be a point measurement. The authors considered the method to be accurate, but noted the need for several measurements, due to spatial variability, if a field measurement is desired.

Further limitations of this method and the air-entry permeameter have not been sufficiently discussed due to its relatively recent adaptation for field measurements and the lack of published data.

g. Extensions

The previously described methods, though not all inclusive, are the standard, accepted methods used for determining the saturated hydraulic conductivity. Other methods tend to be extensions, developments or refinements of these basic methods. Examples of two such methods are the Guelph Permeameter, an extension of the shallow well method, and the Velocity-head permeameter, an extension of the falling-head permeameter method.

i. Guelph Permeameter

A recently developed variation of the constant-head shallow well permeameter is the "Guelph Permeameter" (Elrick et al., 1984). One of the changes made to the shallow well permeameter was the reduction of the effective volume of soil tested to approximately 1000 to 2000 cm^3, changing it from an average to a point measurement (Reynolds, Elrick and Topp, 1983). Likewise the required time, water and supporting equipment needed to run a test in situ were also significantly reduced. The testing range of the Guelph Permeameter is defined as 10^{-8} to 10^{-4} ms^{-1} and can be obtained through the use of two different permeameters with varying reservoirs (Reynolds et al., 1984). Elrick et al. proposed a change in the Guelph's theory such that, a more recent model used to determine the saturated hydraulic conductivity (K_s) includes a term for the influence of gravity in the system. They further noted more accurate numerical procedures as being instrumental in developing better coefficients used in the model. The model used prior to this consideration was stated as under estimating the actual value of K_s. Reynolds, Elrick, and Clothier (1985) further developed a model to include the effects of unsaturated flow around the well. They stated that prior to the incorporation of unsaturated flow effects the value of K_s measured by the instrument was over estimated. Reynolds, Elrick, and Clothier defined the flow factor (C^*) used in the evaluation of K_s, as being estimated for a homogeneous and isotropic porous media and that the potential is assumed to be constant throughout the unsaturated flow domain.

There are several advantages of and applications for the Guelph Permeameter. The instrument itself is considered inexpensive to construct, simple, reliable, has a relatively small water requirement, and a relatively small test time requirement to obtain a reading (Elrick et al., 1984). The aforementioned authors also give several practical applications for this device including: 1) The design of more efficient drain spacing through the use of K_s in theoretical equations; 2) The classification of soils from the in situ use of the instrument; 3) The recognition of weak cap material used for land fill covers and; 4) The design of septic waste water soil absorption systems. With changes in the model used to incorporate capillarity, other in situ parameters can be estimated as well (Reynolds and Elrick, 1985). These parameters include soil sorptivity (S), matric flux potential (ϕ_m), and the α-parameter of the exponential hydraulic conductivity-pressure head relationship.

Limitations through field application and testing must be considered from both a theoretical and physical standpoint. Reynolds and Elrick (1985) demonstrated a dependence of the field saturated hydraulic conductivity (K_{fs}) on the model used to calculate it, resulting in possible over-estimations or under-estimations. They also described a limitation due to soil layering where the hydraulic conductivity would, as expected, be significantly influenced by the layer of greatest conductivity. Physical limitations were attributed to the nature of the test hole and prevailing soil conditions (Elrick et al., 1984). These physical limitations were described as including possible smearing of side-walls when augering the test hole and surface sealing of the side walls due to the presence of finer soil particles. A solution to the side-wall smearing is to use a brush to remove smeared soil from the walls (Reynolds, Elrick, and Topp, 1983). This method was later suggested as being only 50% effective in soils that were considered to be easily smeared, during augering (Lee et al., 1985). Other physical limitations are related to the instrument itself and particularly to the reservoir (Reynolds et al., 1984). These authors have stated that a reservoir must be large enough to allow a test to be run without opening the reservoir, since the reservoir serves as a in-hole Mariotte bottle, yet sufficiently small to accurately measure the flow rate (q). A syringe device was added to the permeameter to help alleviate the problem of lack of water and to help eliminate surface sealing by initially dislodging of the finer particles. This same syringe device is also stated as having the capability of testing for air leaks in the system during measurements. A comparison by Lee et al. between three different methods the Guelph permeameter, the air-entry permeameter, and laboratory cores (falling-head method) demonstrated an interaction between soil type and the technique for measuring the saturated hydraulic conductivity. As pointed out by the authors any method used to determine the saturated hydraulic conductivity would be dependent on several factors such as required type and accuracy of desired K_s measurement, soil type, and various practical constraints of the investigation.

ii. Velocity-head Permeameter

This method was developed through careful evaluation of the equation used with the falling-head permeameter to determine the saturated hydraulic conductivity, (Merva, 1979). Merva, through detailed analysis of results obtained from the falling-head method and through the availability of high speed, hand held calculators was able to develop the velocity-head permeameter (VHP) method.

The theory by which this method functions can be easily understood by careful examination of Darcy's equation of saturated flow through a porous media. The more common scalar form of Darcy's equation solved for the saturated hydraulic conductivity (K) is:

$$K_s = -\frac{Q}{A}\left(\frac{dh}{ds}\right)^{-1} \tag{18}$$

Where,

K_s = saturated hydraulic conductivity with units of L/t
Q = specific flow volume with units of L^3/t
A = cross sectional area of the soil sample with units of L^2
dh/ds = the change in pressure over the length of the sample volume in the direction of flow and is unitless.

This equation can also be represented in another form by:

$$v = -K_s \frac{h}{S} \tag{19}$$

Where,

v = the flux Q/A or the velocity
K_s = same as above and considered to be constant by Darcy's law
h = the change in pressure in the direction of flow
S = the length of the saturated core and is constant by definition of the soil sample length.

Next, the equation can be differentiated with respect to pressure (h) and then solved for the saturated hydraulic conductivity, hence taking the form:

$$\frac{dv}{dh} S = -K_s \tag{20}$$

Since the distance S is known and K_s is desired all that remains is to evaluate dv/dh or the slope of the velocity versus pressure function.

This slope dv/dh can be thought of for some interval Δh as $\Delta v/\Delta h$ and since the velocity (v) can be described by $\Delta h/\Delta t$ (Δt = the time interval) the slope for the same interval Δh becomes:

$$\frac{\dfrac{\Delta h_1}{\Delta t_1} - \dfrac{\Delta h_2}{\Delta t_2}}{\Delta h} \tag{21}$$

Then if Δh is held constant for each consecutive measurement of v then the slope $\Delta v/\Delta h$ for the i^{th} value can be estimated by the following equation:

$$\frac{\dfrac{1}{\Delta \tau_i} - \dfrac{1}{\Delta t_{i-1}}}{i - (i-1)} \tag{22}$$

Finally by plotting 1/t versus the consecutive integer i, the estimate of the slope $\Delta v/\Delta h$ can be found by a least square linear regression of 1/t versus the consecutive integer i. The resultant slope then multiplied by the distance S yields the value for the saturated hydraulic conductivity (K_s). It is shown here by the derivation that the resulting K_s is now determined only by consecutive measurements of Δt but, it is suggested that the value of Δt must be quite accurate. Merva (1979) further demonstrated the use of such an equation in the field and the independence of the method from initial head conditions in the soil. He describes this independence as due to the use of the slope rather than the initial head value. Procedures for field use of the instrument are described in detail by Merva (1979).

The advantages as pointed out by Merva (1979) include the speed at which a measurement can be made and the minimum amount of water required. Further advantages, though not published, can be found in the simplicity of the instrument which enables one person to use the instrument easily in the field and removes requirements for additional supportive type equipment, (Merva, 1986[1]).

Limitations to this method, although seemingly few, have yet to be published. As with the falling-head permeameter or any core method, the result is a point measurement. This point measurement may or may not be considered to be a limitation, depending on the application.

C. In Situ Determination of the Unsaturated K Value

The last method to be reviewed is one described as being capable of measuring the unsaturated hydraulic conductivity in situ. Several methods used to determine the unsaturated hydraulic conductivity have been reviewed by Klute (1972). The crust test method developed by Bouma from Hillel and Gardner's laboratory technique for long columns (Bouma et al., 1971), is chosen for review due to its adaptability to field use in site evaluation for soil absorption systems.

Bouma et al. (1971) implemented an in situ procedure for determining the vertical component of the unsaturated hydraulic conductivity based on Hillel and Gardner's laboratory process of infiltration into soil profiles capped with crusts or impeding layers. The authors point out that as of yet there is no reliable, established method for determining

1. Merva, G.E. 1986. The velocity permeameter. Memorandum on operation, strengths, and weaknesses of the technique.

hydraulic conductivity from fundamental, physical soil properties and therefore suggest that K must be determined experimentally. Furthermore, it is evident that most water movement through the soil around soil absorption systems takes place under unsaturated conditions (Hargett, Tyler, and Siegrist, 1982). Conceptually then, it would be better to know the unsaturated hydraulic conductivity as a function of the matric potential ψ for purposes of design and flow modeling. The method used by Bouma et al., was to carefully carve out soil columns at least 30 cm high with a diameter of 25 cm. Then a ring infiltrometer 10 cm high was fitted onto the top of the column and the column wrapped with aluminum foil then soil packed around it to serve as a vapor barrier. Since the column was under negative head conditions, this was determined adequate to prevent moisture loss except vertically through the soil column. Then a crust of soil, formed by wetting and kneading various soil materials into a thick paste, was applied to the surface in varying thickness to obtain unsaturated conductivities at different soil moisture tensions. A cover was then placed over the crust and water applied to a depth of 3 mm with a Mariotte device. Determination of the hydraulic gradient below the crust was accomplished using pencil size mercury-type tensiometers located just below the crust and 3 cm deeper, both in the center and at the periphery of the column. Once a constant infiltration rate had been reached for a minimum of 4 hours, the unsaturated conductivity was calculated as the infiltration rate divided by the hydraulic gradient below the crust ($K=v/i$). The method was later modified to better measure the saturated hydraulic conductivity by sealing the outside of the column with dental plaster and obtaining a hydraulic gradient of 1 cm/cm (Baker and Bouma, 1976).

This method of determining the unsaturated hydraulic conductivity can be very useful as a tool for design. Limitations of such a method include the excavation of a large amount of soil to accomplish the desired results (Bouwer and Jackson, 1974). Furthermore, the amount of time required to run such a test and the need for a skilled operator would not prove economical for site specific testing (Otis et al. 1977a). Bouma (1982) pointed out that a light crust will not induce unsaturated conditions in the sub-crust soil. Another consideration would be the unstable flow phenomena which could lead to invalid tests for some situations as reported by Hillel (1982). This phenomena has been observed in transition zones from fine textures to coarse textures. As the wetted front proceeds through these zones, the water advancement is not observed to be even but has sudden breakthroughs in specific locations.

With the previously stated criteria for site evaluations, the evaluation and design need to make use of the saturated hydraulic conductivity. Since evaluations occur from spring to fall and since soils with seasonally high water tables are generally unacceptable, only above water table methods for determining the saturated hydraulic conductivity could seriously be considered for common use in site evaluation. With economic and time constraints also brought into consideration the only two methods which hold promise for field applications are the Velocity-head Permeameter and the Guelph Permeameter. The purpose of this research is to consider the accuracy as well as potential use of the Velocity-head Permeameter for site evaluation.

III. PROCEDURES AND METHODS

The procedures carried out to evaluate the effectiveness of the Velocity-head Permeameter (VHP) as a tool for septic system site evaluation consisted of two parts. The first was a laboratory comparison of the VHP to the standard constant-head method. Through this comparison the VHP's accuracy of predicting the results of the constant-head method was statistically analyzed. The second was field testing of the VHP in conjunction with county health department site inspections, to test the instruments feasibility as an on site tool.

The goal of these comparisons is to be able to predict K_s as determined by the constant-head method given a value of K_s determined by the VHP. The VHP values were chosen as the independent variable and the constant-head values as the dependent variable. Due to the wide range of values in the data, a log-log transformation was incorporated to give more even weighting of all data for regression purposes.

A. Laboratory Procedures

The laboratory tests were used as a method to test the validity of the VHP instrument. This portion of the study allowed for comparison of the instrument to standard 7.62 cm cores. It further enabled a comparison of laboratory values to field values obtained in situ. Cores in the laboratory were first used to determine the saturated hydraulic conductivity using standard constant-head outflow procedures (Klute, 1965). Once this was accomplished the same core was tested using the VHP to determine a "VHP" saturated hydraulic conductivity. The cores were then dried to determine bulk density and finally disassembled to determine contents.

1. Laboratory Cores', Constant-head Method

Undisturbed cores were sampled in the field as described in section B, Field Procedures, labeled and brought to the laboratory. In the laboratory a 2.54 cm ring of PVC pipe was added to the top of each core and a cheesecloth type material rubber banded to the bottom of each core. The cores were then saturated by placing them in a container and adding distilled water to the container until it was approximately 1.27 cm from the top of the cores. The cores were allowed to saturate for a minimum of 24 hours at approximately 25 degrees Celsius before determining the hydraulic conductivity. After saturation the cores were placed in a stand and a disc of No. 1 filter paper was placed on the surface to prevent surface sealing during the addition of water. Siphon tubes (4 mm dia.) were then added to provide 0.5 to 2.5 cm of water over each core. Water was maintained at a constant level using a Mariotte bottle. Water was allowed to flow through the cores for a minimum of 15 minutes to establish equilibrium. Water was then collected at 15 minute intervals and measured to determine the flow-through volume. The depth of water was also measured to determine the amount of head acting on the core. A minimum of 4 intervals were recorded and if significant change was noted (approximately 5 ml) between the first and last interval, a 5th interval was then measured. When less than 3 ml of water was collected for the 15 minute interval, the core was re-tested using a modified technique.

The modified technique used for testing low conductivity ranges was to increase the constant head from approximately 2.54 cm to approximately 1 meter. This was done by adding a one hole rubber stopper to the core ring and then placing a 1.25 cm diameter stand tube, fitted with a hose barb fitting, into the stopper. Again, siphon tubes and a Mariotte device were used to maintain constant head while testing. The same procedure was then used to determine flow-through volumes as mentioned above.

2. Laboratory Velocity-head Permeameter

After the cores were tested using the standard constant-head procedure, they were adapted for testing with the VHP. This was done by placing a two hole rubber stopper into the core ring. This two hole rubber stopper was fitted with two glass tubes and a second inverted rubber stopper placed over the tubes. The glass tubes were inserted such that the end of one was elevated about one inch above the other. This allowed for water entry through one and air escape through the other during the filling of the instrument. A P-T Quick Coupler[2] was then placed on the rubber stopper to obtain a perfect seal. The earlier model instrument attached directly to this PT coupler. Finally steps 6 through 8 of the procedure for running the instrument, as outlined below in section B, Field Procedures, was used to determine the saturated hydraulic conductivity of each core. Step number 9 was omitted since the core was very near saturation following the standard constant-head test.

2. P-T Quick Couplings. Cathey Company, 4917 Tranter St., Lansing, Michigan 48910.

Therefore, the first acceptable reading (usually the first or second reading as determined by an acceptable R^2 and the provision of no visible leaks) obtained with the VHP was used. Special attention was given to the connections each time to detect the presence of leaks. If a leak occurred, the connections were reworked, dried, and the test was run a second time.

3. Bulk Density Determination.

Bulk density determination for each core was carried out following the determination of the saturated hydraulic conductivity. The first step in the procedure was to drain the cores until they could easily be removed from the PVC cylinders. Then, after removal, the soil samples were dried for a minimum of 24 hours at 105°C and weighed. Using the volume of the PVC cylinders, bulk density was calculated and recorded. The reason for removing soil from the PVC cylinders was to protect the PVC from heat damage. Special care was used to make sure all the soil was removed from the cylinder by brushing the cylinders clean and then cleaning the brush.

4. Core Break-Down

Finally cores were broken-down to observe their contents. This enabled observations about the presence of large pores or channels and the presence of materials which may adversely affect the results. The cores were screened through a sieve (2.83 mm) to determine the presence of stones in the cores. The materials found was estimated and recorded by percent volume for future reference. Finally, the approximate diameter of large stones was recorded to show possible influence on the hydraulic conductivity. Further information such as presence of roots and other materials were recorded where applicable. If any cores gave unique results (e.g. large fluctuations between consecutive readings) they were saved intact for future reference.

B. Field Procedures

Currently in the State of Michigan there is no instrumentation used routinely to determine the in situ hydraulic conductivity when evaluating a potential site for a soil absorption system. Therefore, the VHP was used tested to determine if it could be used for this application. This involved working with the county sanitarians to compare their estimation of pass/fail of a site with values of the hydraulic conductivity obtained in situ with the VHP.

1. Current Evaluation Procedures

The county sanitarians follow State Department of Public Health guidelines when evaluating a potential site for a soil absorption system. These guidelines have been reviewed earlier and consist of six items for acceptance (Machmeier, Hansel, and Anderson, 1982). The six items that need to be considered are as follows:

i) Depth to highest known water table or bedrock.

ii) Soil conditions, properties, and permeability.

iii) Land topography or slope.

iv) Existence of lowlands, surface depressions, rock outcrops.

v) All legal setback distances maintained from wells, lakes, streams, buried pipes and lines, etc.

vi) Surface water flooding probability.

Each individual site must meet certain basic requirements based upon these six items. The basic requirements for the state of Michigan have been outlined by Bohunsky et al. (1977). The use of the VHP focused on only one aspect of the second item, the soil permeability.

Presently in Michigan, the soil permeability is estimated by two possible techniques, soil borings and/or back-hoe excavations. Before arriving at the site, the sanitarian will review available soil maps and soils data. If the review of the soil maps suggests soils with high percentages of sand, the sanitarian may evaluate the soil using only soil borings commonly made with a hand auger. Also, based on the review of soils maps, a sanitarian may require a back-hoe excavation to locate soils that can provide adequate treatment of wastes or to observe the soil profile for possible limiting conditions (e.g. compacted layers). It is not uncommon for both borings and excavations to be performed on the same site at different times depending on earlier findings. These borings or excavations must be at least 1.2 meters below the bottom of the proposed system (Bohunsky et al., 1977).

Next the soil's texture is evaluated and changes which occur in the horizon are noted. Once the soil has been analyzed through field textural identification procedures, an estimate for the hydraulic conductivity is determined from the soil maps data and consideration of how the textural classification coincides with these mapping units.

Prior to this method of estimation, a percolation test was required by the State to evaluate the soil permeability. As mentioned earlier, the percolation test, although useful, produces discrepancies and has a general lack of accuracy which lead to the elimination of this test method as a requirement in the state of Michigan. Ironically, if a site is turned down based on the soils evaluation it is still possible to overrule a sanitarian's decision through local governmental units by using the results of percolation tests.

2. Field Use of the Velocity-head Permeameter

The use of the VHP in the field was aimed at determining the permeability or saturated hydraulic conductivity of soils evaluated by county sanitarians for potential soil absorption systems. Since limited funding was available to carry out this research, guidelines were developed to aid in the evaluation of each site. Although for most sites a backhole was used to evaluate the soils, time and money did not permit evaluation of the conductivity in these same backhole excavations. Therefore, the conductivity was evaluated in a small hand dug pit adjacent to the back-hoe excavation. Each pit was approximately one meter square and 46 to 71 cm deep. This allowed for evaluation of the soil near where the actual system might be located in the soil. In some cases the soil was evaluated at different depths if time allowed or if the sanitarian and/or land owner requested it. The saturated hydraulic conductivity was first measured in the vertical direction at the bottom of each test pit, estimated to be near the depth of the proposed system. A minimum of three separate values were measured (since the VHP is a point measurement) and then averaged to determine the in situ saturated hydraulic conductivity. This value was then used for comparison with pass/fail values given by the sanitarians in the field. Finally, a horizontal measurement was taken near the same depth for the purpose of comparison since the hydraulic conductivity tends to vary directionally due to the soil lacking homogeneity (Harr, 1962).

The VHP comes with three different size coring devices for use in soils with very slow to rapid hydraulic conductivities. A schematic diagram of the instrument used can be found in figure 1. For purposes of comparison and limiting the introduction of possible error, the smaller coring device (3.81 cm diameter) was used almost entirely during field testing. This decision tended to avoid including the effect of biopores or worm channels on the hydraulic conductivity. Since the coring device's inside diameter was only 3.81 cm, it was easy to locate the coring device between observed biopores. The undisturbed cores that were collected for laboratory measurements were also of small diameter (3.81 cm) to similarly avoid the same influence of biopores.

Distilled and deionized water was used in the VHP for testing to eliminate any ionic effects on K_s. The water temperature was kept near 25^oC. Since relatively little water is needed for the VHP instrument (Merva, 1979) it was possible to transport enough distilled water (approximately 18 liters) to evaluate 2 to 3 sites at a time. All field and laboratory tests were carried out by the same person, thereby enabling a reduction in possible experimental error which could arise with testing by different operators.

Small changes were made in the original VHP instrument described by Merva (1979). These changes were separation of the instrument from the coring device via extendible tubing and the addition of a specialized valve which allowed for faster conductivity determination. The earlier

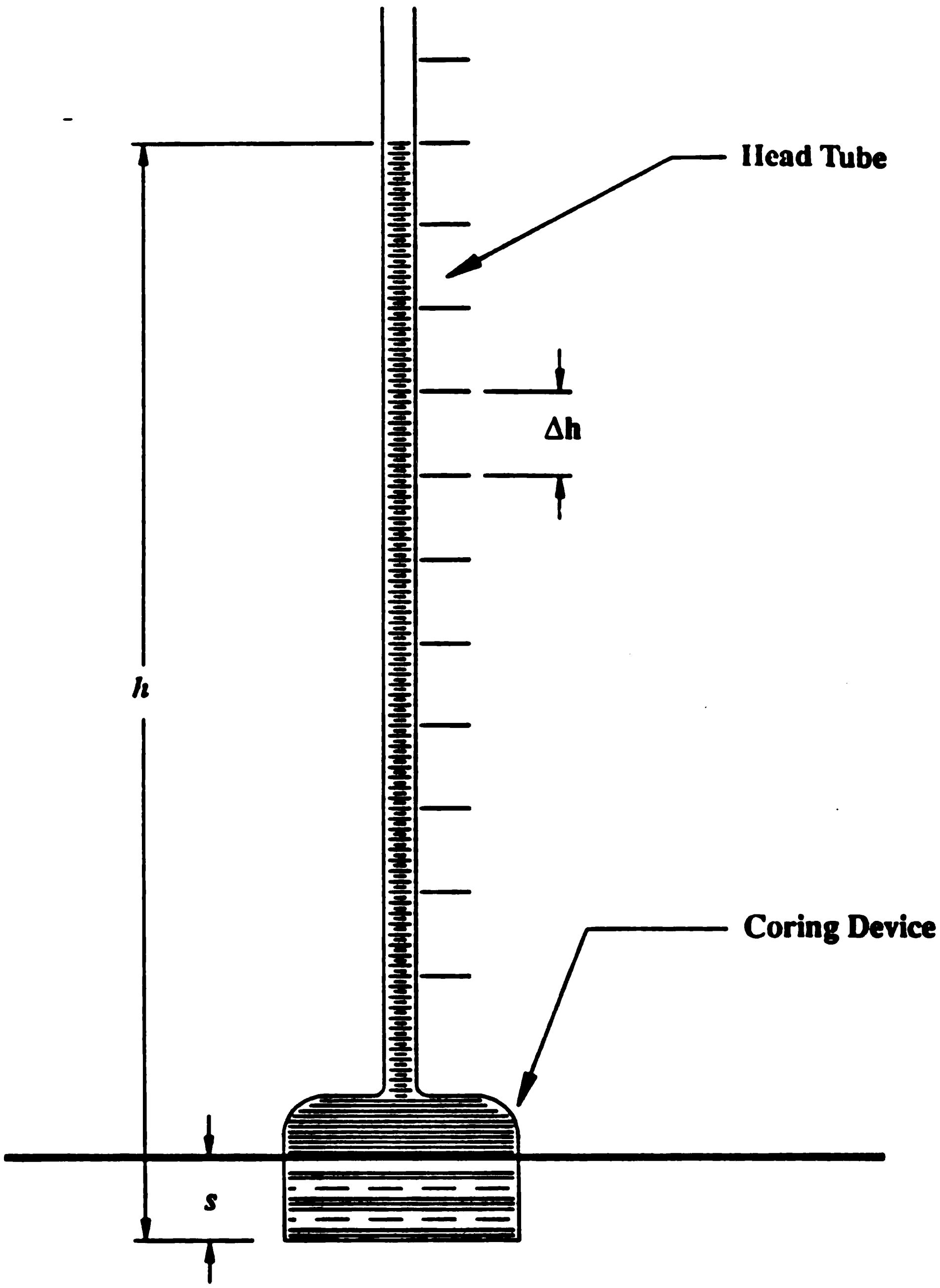


Fig. 1. A schematic of the basis on which the Velocity Permeameter operates. The velocity of entry of water into the soil core of length *s* is magnified by the decrease in cross sectional area of the head tube which supplies the head *h*. The velocity is fall of the head of water is determined by the time required for the water column to fall through the head increment Δh.

instrument was mounted directly on the coring device which made determinations difficult when strong winds prevailed. Winds would rock the instrument which would loosen the coring device in the soil. The valve addition allowed reservoir tubes to be more easily matched to soil conditions. Water was added through the use of a small 7.6 liter garden sprayer, that could be pressurized. By adding water to the coring device and forcing the water back through the instrument, with a minimum of pressure, entrapped air was expelled from the system. Once the instrument had been filled, the valve to the pressurized sprayer was closed and the readings were taken at the pressure determined by the height of the water in the head tube.

The exact procedure for use of the instrument was as follows:

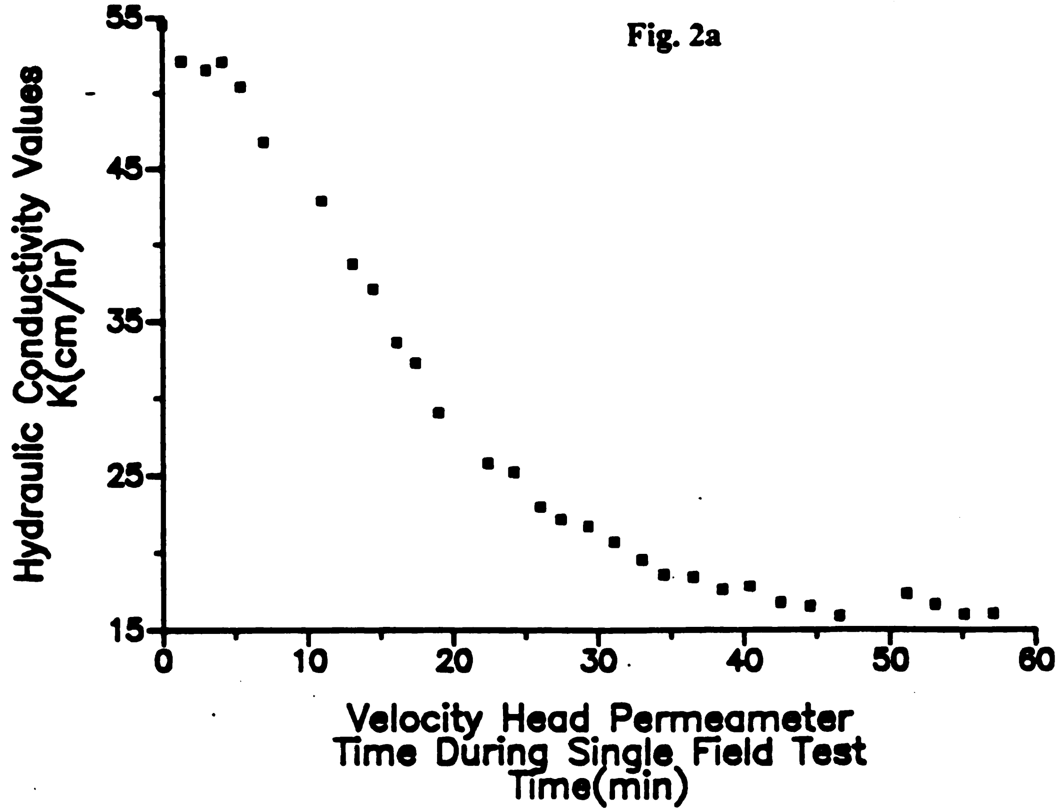
STEP	PROCEDURE
1)	After excavation of the test pit, the bottom of the pit was cleaned with a hoe.
2)	Once the surface was cleaned, the coring device is placed in the desired location of testing (for this study this meant between biopores).
3)	The coring device was then driven a distance of 1 to 3 cm into the soil, using the driver. For horizontal measurements the core was driven into the soil using a jack device.
4)	Once the coring device had been driven to this depth, the coring device was rocked back and forth in a circular fashion and the coring device lifted removing a thin core of soil to provide a clean shear face on the soil surface for testing (the coring device and test area were then cleaned and small soil particles removed by simply blowing across the shear face). This allows an undisturbed soil surface and thereby prevents soil smearing which could significantly reduce the measured K_s.
5)	The coring device was then placed back in the exact same location and driven approximately 2 to 5 cm into the soil (this distance must be measured accurately) using care to drive the coring device straight and provide a minimum of soil disturbance.

STEP	PROCEDURE
6)	Once the coring device was in place, the extension tubes were attached to the coring device and water was added to the to purge air from the system.
7)	After filling the system with water, the fill valve was closed and the column of water allowed to descend from the head tube as water was conducted through the soil.
8)	The rate of change of velocity with respect to coring device was measured using a hand held programmable calculator (Hewlett Packard (HP) 41-CV with timing module). A program has been developed to accomplish measurement of the rate of change of velocity which yields a value of K_s based on Merva's (1979) equation. The values were recorded using the HP hand held printer which connects to the CV hand held computer.
9)	Steps 7 and 8 were repeated until the value of K_s did not change significantly*, usually no more than 4 or 5 runs were necessary.

*NOTE: The value of K_s decreases as the wetted front approaches the distance the coring device is driven into the soil and then stabilizes, (Merva, 1979). A plot of this phenomenon can be found in figure 2a,b.

3. Undisturbed core sampling

A core to be used in laboratory testing was removed from beside the location of each vertical VHP measurement using procedures outlined by Blake (1965). The device used to extract core samples was based on Blake's design, but had minor modifications. These modifications were the reduction of the diameter of the core from 76.2 mm to 38.1 mm and the use of PVC pipe as the outer cylinder for each core. The diameter reduction was adapted to reduce the impact of large biopores and channels in the soil and match the size of in-situ VHP tests. The use of small samples would be expected to diminish the effect of macropores pointed out by Bouma (1982), but at the same time enabled the



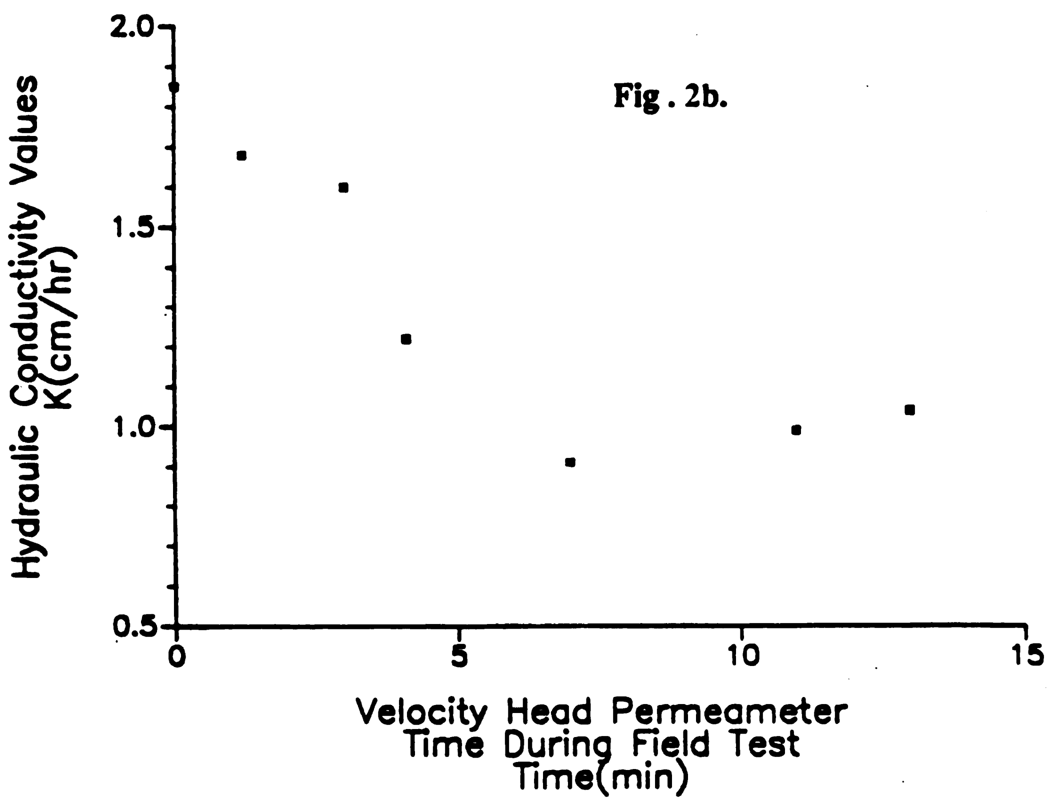


Fig. 2. Figures 2a and b illustrate the decreasing apparent hydraulic conductivity with time as measured with the Velocity Permeameter. This is due to time required for the soil core to saturate. When the core is fully saturated, in most soils, a slight increase in measured hydraulic conductivity is noted. The least value of hydraulic conductivity is assumed to be the conductivity of the soil.

measurement of minimum values which could occur should biopores and channels be eliminated through continuous saturation of the soil. The procedures for core extraction were as follows:

STEP	PROCEDURE
1)	After determining the vertical hydraulic conductivity with the VHP, cores were removed adjacent to each VHP location.
2)	The exact location for the core was determined by observing the soil surface and then placing the sampling device between large biopores and channels.
3)	The coring head was then driven into the soil until the soil filled the PVC cylinders.
4)	The core was extracted from the coring head and carefully trimmed with a knife to 7.62 cm in length.
5)	Finally, the core was carefully placed into a padded box to minimize disturbance during transportation back to the laboratory.

Each core was carefully handled to minimize disturbance prior to determination of the saturated hydraulic conductivity in the laboratory.

4. Additional Field Testing

During the period of field research, several additional applications and testing of the VHP were performed. These applications included testing a soil absorption system with known history, evaluating a field to be used in subsurface irrigation research, and defining problems in ground water flow for three separate situations.

a. Known History Absorption System

The system tested was located in Southeastern Michigan and serviced a typical three-bedroom home. The owner built the house and had the septic soil absorption system installed in 1959. At this writing he still owned the house and contrary to all recommendations, had never had the septic tank pumped or cleaned. The system was installed in medium fine yellow sand and considered to be a typical trench type system. The effluent from the septic tank is distributed into three separate trenches that are approximately 2.4 to 3.1 meters apart and 0.61 meter wide. The tile used were common 10.2 cm clay field tile with minimal spacing between tile. The system is completely connected at the distal end of the three tile lines. 15 to 20 cm of coarse stone is distributed in the trenches below the tile lines. System loading consists of the waste water from two bathrooms and a kitchen with dishwasher but no garbage disposal. There was no loading of washing machine waste water at any time to the system. There were 5 people contributing to the system until approximately 10 years ago at which time the number of people was reduced to two, due to the children moving away.

The hydraulic conductivity of the system was tested in two different locations. The first was at the distal end of an outside lateral which was near the surface and the second was outside of the distribution box on the center lateral. Both vertical and horizontal measurements were made at the soil/stone interface. While excavated, the system was loaded with over 350 liters of water to observe possible ponding conditions. Observations and measurements were recorded. Finally, the parent material between two trenches near the distal end was tested to determine the saturated hydraulic conductivity of this soil.

b. Subsurface Irrigation Site Evaluation.

This special case was to evaluate the horizontal hydraulic conductivity of several back-hoe pits in a research field. This field is located near St. Johns, Michigan and is part of Michigan State University's ongoing subsurface water management research. The horizontal saturated hydraulic conductivity was determined using the procedures described above. For comparative analysis, undisturbed cores were taken in the horizontal direction using similar procedures outlined in section B.3, Undisturbed Core Sampling, with the only difference being that the sampler was driven in the horizontal direction. The results of the VHP testing will be compared to the piezometer method (see the literature review) following installation of drain tubes.

c. Groundwater Flow Problems

This application involved three different situations dealing with groundwater flow. The first was a potential site for home septic waste disposal system with the presence of a high water table. A mound type system was designed for the location to overcome site limitations. The VHP was used to look at the surface layer of soil to determine whether the A horizon should be removed to allow a more permeable layer at the interface. Two additional sites were tested to determine saturated hydraulic conductivities that could be used for design on each.

A second situation was at the Muskegon Waste Water Facility in Muskegon, Michigan. The problem which existed at this location was surface ponding on a leveled portion of the site to which waste water is applied via sprinklers for water treatment and filtration. The ponding was considered to be a hindrance to workers in this location. The VHP was used to determine where the actual problem existed in the soil.

The third situation was a site evaluation at Michigan State University where a subsurface irrigation system had been installed and was thought to be malfunctioning. The system had been designed based on values obtained through the auger-hole method. The method of evaluation was to look at two back-hoe pits; one where the system was working effectively and another where the system was functioning poorly. Once conductivity values were obtained the two pits were compared to determine differences.

C. Data Analysis Method.

The data were analyzed from several perspectives. The first analysis involved comparing laboratory measurements obtained with both the VHP and standard constant-head method using 7.64 cm cores. The transformed VHP method vs. standard constant-head method data were plotted and fit by least squares linear regression. This fit gave an indication of the accuracy of the instrument as compared to the standard constant-head method. Further analysis involved comparisons between field measured values and laboratory values. For application of the VHP to home septic site evaluation, a comparison of sanitarian pass/fail determinations were made to VHP field measured values. The bulk density versus hydraulic conductivity and vertical versus horizontal conductivities were plotted to enable comparison as well. The results of these comparisons can be found in chapter IV, RESULTS AND DISCUSSION.

The method of statistical analysis was based on the experimental design and the comparisons used to verify the results. The experimental design consisted of three treatments: a) Constant-head laboratory method; b) Velocity-head permeameter in the laboratory; and c) Velocity-head permeameter in the field. Treatments a and b had the same number of samples since they were applied to the same core. Treatment c used 3 to 5 samples per site, depending on the site and time required to obtain samples in the field. Blocking according to location allowed the separation of each site. If two potential sites existed on the same piece of land, they also were separated by blocking. The wide variations found in Michigan soils as well as the impact of soil management on the hydraulic conductivity required such a separation. Once all the data had been collected and organized based on this experimental design, then analysis of the data was undertaken.

The first step in the analysis was to look for the presence of outlying data points. This analysis consisted of determining the mean for a given subset and then determining if any point in that subset was significantly different from the mean based on the given amount of error of the subset. The subsets included in the analysis were as follows: a) sample within a repetition, b) repetition within a treatment, c) treatments within a block, and d) all blocks within the data set. The equation for testing for such outliers is described by Gill (1978) as

$$\text{Test Statistic:} \qquad e_L/((MSE)^{0.5}) \qquad (23)$$

Once the statistic is calculated, it is then tested against a critical value, presented by Gill in tabular form, to determine if the test statistic is greater than the critical value. For this research a 95% confidence limit was chosen (an α value of 0.05) for all statistical analysis. If a point was determined to be an outlier, then examination was made to determine if sufficient reason existed to remove the data point from the subset and subsequent overall analysis. No point was removed only because the test statistic determined it an outlier. Reasons for removing data points included: 1) difficulty in obtaining good measurements in the laboratory (e.g. system leakage, large variations in consecutive point measurements, obvious surface sealing, and possible core damage); and 2) field problems such as large stones, possible inclusions within a core, and uncertainty of the interaction of the point measurement with a layer of dissimilar soil. If such problems were not noted during data collection then the point was not removed. The total number of cores considered outliers and removed from the data was 16 out of a total of 155 cores sampled, a 10.3% reduction. In the field-to-laboratory comparisons, 4 out of 40 sites were removed, a 10% reduction. A separate analysis for outliers was done on the bulk density of each core, a completely independent variable also measured during testing. In this analysis, no cores were found to have significant variation in the bulk density.

The second part of the analysis was to determine if a correlation existed in the comparisons. Based on the theory outlined by Merva (1979), the VHP should yield the same results as the constant-head method. Using this assumption one expects a linear relationship between values of the saturated hydraulic conductivity measured with the two different methods. Hence, the ideal model for comparison is a 45 degree line through the origin. In order to compare the measured results against the expected model, linear regression was performed on the data using a statistical package known as Plot-It (Eisensmith, 1988). Upon evaluation of the data it was quite obvious that the wide range in values (0.01 cm/hr to 76 cm/hr) necessitated some transformation of the data to eliminate bias from the larger values of K_s. A transformation similar to that used by Mason, Lutz, and Petersen (1957) was employed for this analysis and necessitated taking the Napierian logarithm of all the data prior to regression. The only difference from the previously mentioned authors' technique and this analysis was the multiplication of the data by 200, rather than their use of 100, to eliminate negative numbers in the regression calculations. Transformations are as follows:

Constant-head Laboratory Values

$$Y_T = \log(K_s \times 200) \tag{24}$$

Velocity-head Permeameter Values

$$X_T = \log(K_s \times 200) \tag{25}$$

After transformations of the data were performed, linear regression of the data using the least squares method was initiated. The model chosen for linear regression fixes the intercept at zero and then determines the best fit line through the data. The first order linear model for this case is:

$$Y' = B(0)(X'). \tag{26}$$

where,

$Y' =$ transformed Y values (log(200Y)).
$X' =$ transformed X values (log(200X)).
B(0)= slope of the least squares best fit line.

The results yielded the coefficient of determination and slope for the regressed line to enable comparison with the ideal slope of 1. For purposes of further comparison the linear regression model was changed to a non-fixed zero intercept and regressed again. This model is represented by the equation:

$$Y = B(0) + B(1)(X) \tag{27}$$

The residuals were also analyzed in order to determine certain biases expected in the comparison. Finally, to fully understand the adequacy of the regression analysis, the analysis of variance (ANOVA) tables were constructed to evaluate the error involved. The results of this analysis are found in chapter IV, RESULTS AND DISCUSSION.

IV. RESULTS AND DISCUSSION

A. Selection of an In Situ Method

1. Limitations of Several Methods

The use of the VHP in the field led to several qualitative observations. The instrument itself is relatively easy to use and requires only one person to both operate and transport. This alone is an advantage over most other methods described earlier. Most of the above water table methods require a large amount of water to obtain steady state readings since the amount of water required for testing is directly related to the volume of the test sample. Requirements for excess water would also introduce the undesirable need for additional equipment. Since in most field situations this is either impossible or too costly, consideration of other field methods is necessary to provide an accurate yet feasible and economic method of testing. Methods described as requiring small amounts of water include the Crust Test, Guelph Permeameter, Air-Entry Permeameter, Falling-head Permeameter, and Velocity-head Permeameter. The reduction in the amount of water required to run tests is attributed to a significant reduction in the volume of soil tested for all five methods. The VHP requires very little water and usually was found to be adequately supplied by the 7.6 liter garden sprayer, although a 11.4 liter sprayer could also be used with the instrument to allow extended testing in areas where water availability is limited.

Another major limitation to consider in field use is the time requirements for testing. This becomes even more significant when considering site evaluations of proposed absorption systems since the cost versus benefits must be carefully considered for these small size home-systems. This, according to Otis et al. (1977a), would tend to eliminate the Crust Test from routine use in site evaluations. Unfortunately, this eliminates the one test capable of determining the unsaturated hydraulic conductivities. The remaining four methods are described as requiring little time to evaluate the saturated hydraulic conductivity. Of these methods, the air-entry permeameter method, requires two pencil size tensiometers to be installed with the coring device of the instrument to prove fulfillment of the requirements of Darcy's law. Problems associated with the use of tensiometers are soil manipulation resulting from tensiometer insertion and a problem of tensiometer breakage under certain conditions. If this instrument is ruled out, that would leave three remaining for evaluation.

The falling-head permeameter fulfills the requirements of relatively small time and little water, but is limited in its use by the dependence on the matric suction at the wetted front in determination of the hydraulic conductivity. This dependence can be seen by the use of head (H) in the equation for the hydraulic conductivity:

$$K = (al/At) \ln(H_1/H_2) \qquad (17)$$

where,

a = the cross-sectional area of the manometer
l = the length of the soil sample
A = cross sectional area of the soil sample
H_1 = the head at time zero
H_2 = the head at time = t
t = time from start to finish

This equation, by the natural logarithm term, would suggest a dependence on the soil matric suction. The total head (H_1,H_2) used at the beginning and end of each test may vary significantly from the pressure added by the column of water used to determine t. This variation could be attributed to the matric suction and the location of the wetted front. This dependency would then lead to possible underestimation of the saturated hydraulic conductivity, or to an extended length of time needed to attain steady state situations. Such a discrepancy could tend to restrict the accuracy and limit the use of this instrument in the field.

Of the two remaining methods, the VHP is considered a point measurement and the Guelph permeameter more of a average value measurement. From this statement alone it would seem that both instruments would have their own place in field use. Although this is true, there are still limitations to the Guelph Permeameter method that deserve consideration. The first of these limitations is in theory. The parameters that govern the use of the instrument were developed for laboratory situations. As mentioned earlier, these parameters first underestimated and later overestimated the saturated hydraulic conductivity. Although the method of parameter estimation has been refined by the use of resistance networking, it may still be a source of error. Such modeling has assumed homogeneity and isotropy, an approximation for most soils in situ. The second limitation is the smearing of the test hole walls which frequently occurs when test holes are installed in a wet to saturated clayey soil. These conditions result in surface sealing and erroneous results.

2. Limitations of the Velocity-head Permeameter

This leaves the final method, the VHP, as possibly more desirable than the others. Although this method tends to have its limitations, the order of magnitude of these limitations would be suggested to be much less especially due to the independence on the total head of the system. Like most other methods described in this research, the VHP has advantages and limitations. These limitations may or may not be limitations for other methods and, as with other methods, there is some small amount of trade-off involved.

The first limitation of the VHP is based on the expansion of the natural logarithm in the equation as per Merva (1979). In his development, the author assumes no influence of the second derivative term in the Taylor expansion on the hydraulic conductivity. For most situations this would be a valid assumptions. However, a review of literature on the influence of clay particles (Swartzendruber, 1969) suggests that clay soils may have significant impact on this second derivative or exhibit non-Darcian behavior. Before throwing away this method because of such a problem it is important to note that other methods reviewed do not take into account this problem either.

A second limitation is the consideration that the instrument yields a point measurement, a problem of several of the above water table methods. This has been pointed out by Anderson and Bouma (1973) to be a problem when a field value is desired. In order to characterize a soil horizon, many values must be taken to determine some average value for the given location or field. Although this sounds tedious, many times only a few measurements are required for conformation of a range of conductivities. The five methods mentioned above could also be considered point measurements with regard to obtaining field measurements. As mentioned earlier, by reducing the volume of the field sample, the time and water requirements can be reduced but at the same time the number of tests necessarily increases. The use of the VHP in the field demonstrated that for field size characterization, if one excavation was carefully evaluated with the instrument, other spot checks in the field could determine if the same order of magnitude existed or if the soil had changed due to soil series or management practices. Many times such a change could be predicted by a minimal amount of morphologic classification and then tested using the VHP to determine a value for the saturated hydraulic conductivity.

In contrast to the factor of point measurement being a limitation, it can also be viewed as an advantage. Many times in the field it is desirable to look at compacted or other limiting layers in the soil horizon. The VHP has been used by two other researchers as an effective means of identifying such layers and compaction in a quantitative sense (Perry et al., 1986 and Kanwar et al., 1985). The saturated hydraulic conductivity, in such cases, can be linked to a decrease in soil porosity, decrease in the voids ratio, and an increase in the bulk density. These factors all relate to the problem of soil compaction from various sources and practices.

The use of the VHP in the field also requires a fair amount of excavation to accomplish deep measurements. This is due to the instrument again yielding point measurements. In certain cases this may be considered a limitation but in most cases the soil profile needs to be examined as well and therefore presents no limitation. The use of the VHP is such that it allows classification of the soil horizon very easily after testing is completed to the desired depth. Such excavation usually allows the user to easily define the layers in the horizon enabling measurement in specific layers to evaluate the entire soil horizon.

Two physical limitations to the VHP include the use of the instrument in stony soils and the influence of biopores and macro-structure. These problems tends to plague most of the above water table methods. The main difficulty in stony soils is the potential damage to the coring device of the instrument. The coring devices used in this research were hardened to prevent damage but care while operating the instrument is also necessary to prevent such damage. While using the VHP in the field, it was found that if the coring device was driven in slowly and carefully, a sudden stop in movement upon impact could signify the presence of a stone beneath the wall of the coring device. The general procedure used in the field was to remove the coring device, relocate it and try again. It is estimated for this study that this procedure is about 70-80% effective in stony soils.

3. Overcoming the Limitation of Stony Soils

Another concern in soils of this type is how to get a measurement if the percentage of stones or rock fragments is high. Brakensiek, Rawls, and Stephenson (1986) demonstrated that the saturated hydraulic conductivity of soils with rock fragments can be estimated from the saturated hydraulic conductivity of the fine earth fraction and the rock fragment content by weight. The equation derived by these authors is of the form:

$$K_b/K_s = (1-R_w) \tag{28}$$

where,

K_b = field soil saturated hydraulic conductivity
K_s = fine-earth fraction saturated conductivity
R_w = rock fragment content by weight (decimal)

These findings help to support use of the VHP in such soils since the instrument can be easily used to determine the saturated hydraulic conductivity of the fine earth fraction. This can be accomplished by carefully excavating to find an area where small numbers of rock fragments exist and by using the small coring device to reduce the sample volume. If these steps prove successful, the result can be used in the equation above to estimate the field soil saturated hydraulic conductivity. This also demonstrates the usefulness in some situations of having a point measurement rather than some larger average that could be more greatly effected by rock fragments.

4. Overcoming Biopore and Macro-Structure Problems

Large biopores have also presented some difficulty in taking field measurements. In this research it was decided to measure the hydraulic conductivity without the influence of large macropores or biopores. There is a concern here as to what effect such pores have on the soil horizon. If the channels or pores extend to the drain tube depth in drained soils they would obviously effect the drainage rate in some fashion. Likewise if these pores or channels exist in the soil below a distribution system they would have some impact on the rate at which water could be applied. For this research a decision was made to attempt to neglect the pores or channels as with continued saturation of the soil it might be expected that the percentage of such structure could significantly decrease with time. Anderson and Bouma (1973) suggested the use of large sample volumes to incorporate the effect of this structure into the saturated hydraulic conductivity. The VHP is capable of doing this to a certain degree by using the largest diameter coring device in such situations. The major difficulty lies in the fact that if such structure within the sample, is continuous with other large pores or channels it becomes difficult to get a reading as the water quickly disappears to parts unknown. This can also occur due to soil fracture when driving the coring device into the soil. If continuity of macro structure exists two things can happen; the water reappears a small distance away from the coring device (described as a blow-out), or simply disappears deep in the soil. If either situation prevails the, R^2 of the hydraulic conductivity becomes very low. To help alleviate the problem of soil fracture by the sample coring device, the coring device wall thickness is being reduced. The other procedure is to use a smaller coring device neglecting the large pores and channels then estimate the effect or impact on the measured value, using the measured value as a minimum value.

B. Conversion of K_s to an Unsaturated Value

As mentioned earlier, the unsaturated hydraulic conductivity is strongly desired for design purposes. Since field measurement of this parameter tends to be time consuming and difficult or tedious, a method of estimating it based on the saturated, in situ value would be quite useful. Gardner (1958) developed an equation relating the unsaturated hydraulic conductivity at a specific soil moisture tension to three other parameters related to the soil type. The equation is of the form:

$$K_p = a/(-p^n+b) \tag{29}$$

where,

K_p = the unsaturated hydraulic conductivity at some suction pressure
p = the suction pressure at which the conductivity is to be estimated
a,n,b = parameters used for modeling based on laboratory testing of several soils.

Bouwer (1964) listed some standard values for a,n, and b for three different soil types, shown in table 1 . Given the standard values, an expected value for the unsaturated hydraulic conductivity can be achieved using the above equation. Although at present there are few values of K_s for which a, n, and b exist, more values could be developed. Since K_s is easily obtained with the VHP, further development of Gardner's equation or similar methods could be quite valuable.

TABLE 1*

Typical parameter values for three soil types to be used in Gardner's equation to predict the unsaturated hydraulic conductivity from the measured saturated value.

Soil Type	K_s cm/day	a	b	n
Medium Sands :	500	$5x10^9$	10^7	5
Fine Sands, Sandy Loams :	50	$5x10^6$	10^5	3
Structureless Loams : Clays	1	$5x10^3$	$5x10^3$	2

* Taken from Bouwer 1964.

In order to apply Gardner's equation in reference to waste septic systems a value for the expected matric potential is required. Bouma (1971) found the matric potential in operating systems just below a clogging mat to be negative 30 to 40 millibars. This value along with K_s and using table 1 provides the necessary parameters to use Gardner's equation to estimate the expected unsaturated hydraulic conductivity.

C. Laboratory Results

Figure 3 is a scatter diagram of the laboratory data used for analysis with outliers removed. A plot of the transformed laboratory K_s, on undisturbed cores, with the best fit least square regression line is found in figure 4. For the linear regression with the intercept fixed at 0, the slope was 1.02 ±0.03 with a standard error of 0.013 at the 95% confidence limits, a coefficient of determination equal to 0.98, and a residual (observed value - regression line value) standard deviation was 0.43 (10% of full scale). The F value from the ANOVA (table 2) was 6248 with a significance below 0.005 %. If a direct relationship existed between the two methods, the expected slope would be exactly equal to 1.00. The results show that at the 95% confidence limit the slope (1.02) is not significantly different from 1.00 (the confidence interval for the slope contains the value of 1.00). Therefore, the VHP would be expected to accurately predict the constant-head method value 95% of the time given the residual standard deviation of 0.43 about the predicted value in the range of 0.01 to 76 cm/hr.

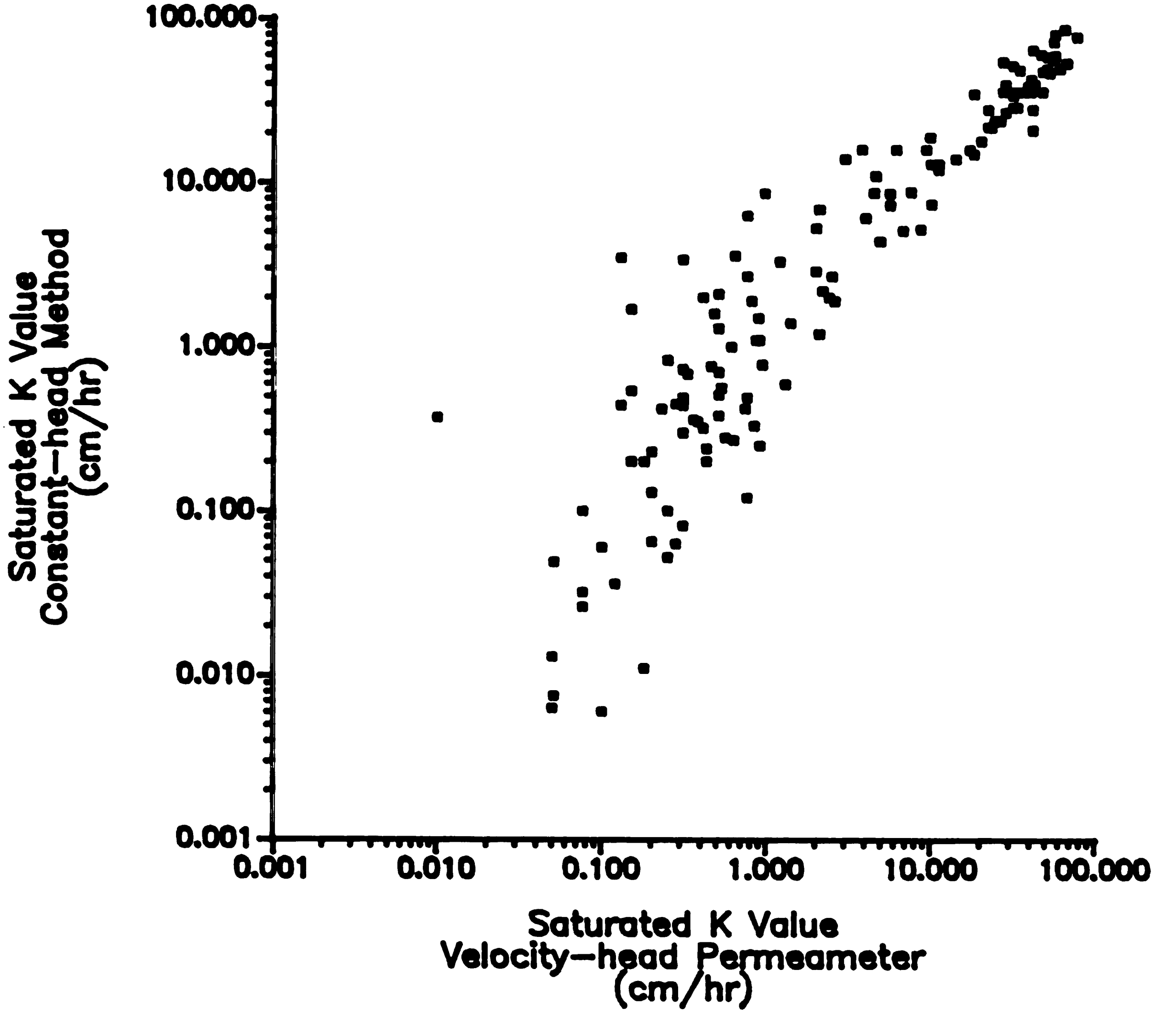


Figure 3

Scatter diagram of saturated hydraulic conductivity values obtained with velocity-head permeameter in the laboratory versus, the same value for the same undisturbed core obtained with the standard constant-head outflow laboratory procedure.

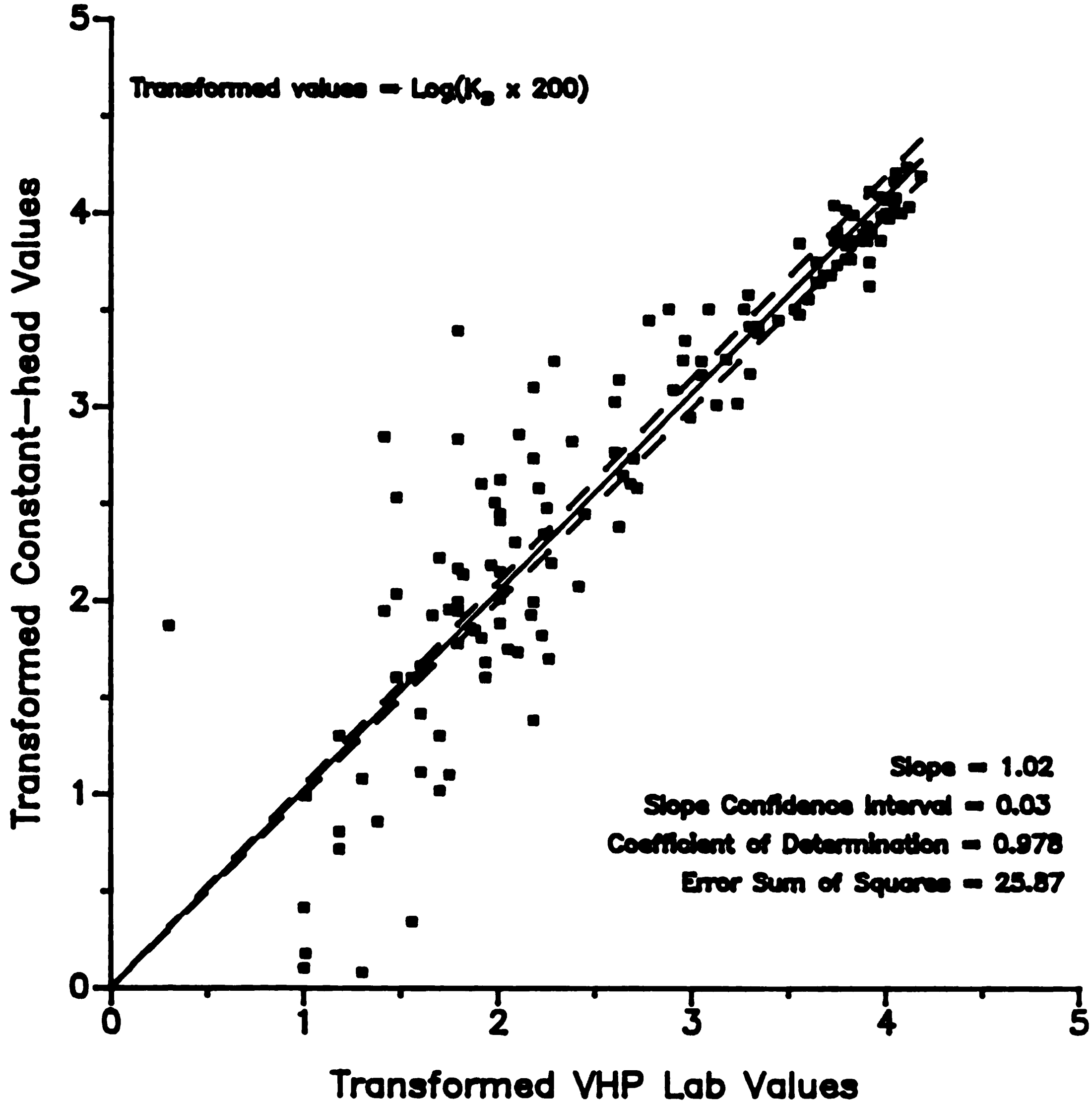


Figure 4

Linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter versus, the same value for the same undisturbed core obtained with the standard constant-head outflow laboratory procedure, for the range of 0.01 to 76 cm/hr.

TABLE 2

Analysis of the variance for linear regression of transformed saturated hydraulic conductivity values obtained with both the velocity-head permeameter and the constant-head methods on the same undisturbed cores, for the range 0.01 to 76 cm/hr.

ANALYSIS OF VARIANCE				
Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Regression	1	1171	1171	6248
Residual	138	25.87	0.188	
Total	139	1197	Sig. of F Value: < 0.00005	

The regression analysis using the non-zero intercept model produced similar results. The slope was 0.99 with a standard error of 0.038, the intercept 0.09, the coefficient of determination 0.84, and the residual standard deviation was 0.43. The lack-of-fit test suggests that the non-zero intercept regression line does not fit the data as well as the forced zero regression line (lack-of-fit F for forced zero = 0.949 ; F for non-forced zero = 0.952)

Figure 5 demonstrates the accuracy of the instrument in the range from 0.6 cm/hr to 76 cm/hr. The resulting statistical analysis (see table 3) demonstrates that a large portion of the variance in the data can be attributed to values below 0.6 cm/hr, although some reduction in variance could be attributed to a reduction in data points from 138 to 89 or a 36% reduction. The best fit line through the data has a slope of 1.02 ±0.02 with a standard error of 0.009 at the 95% confidence

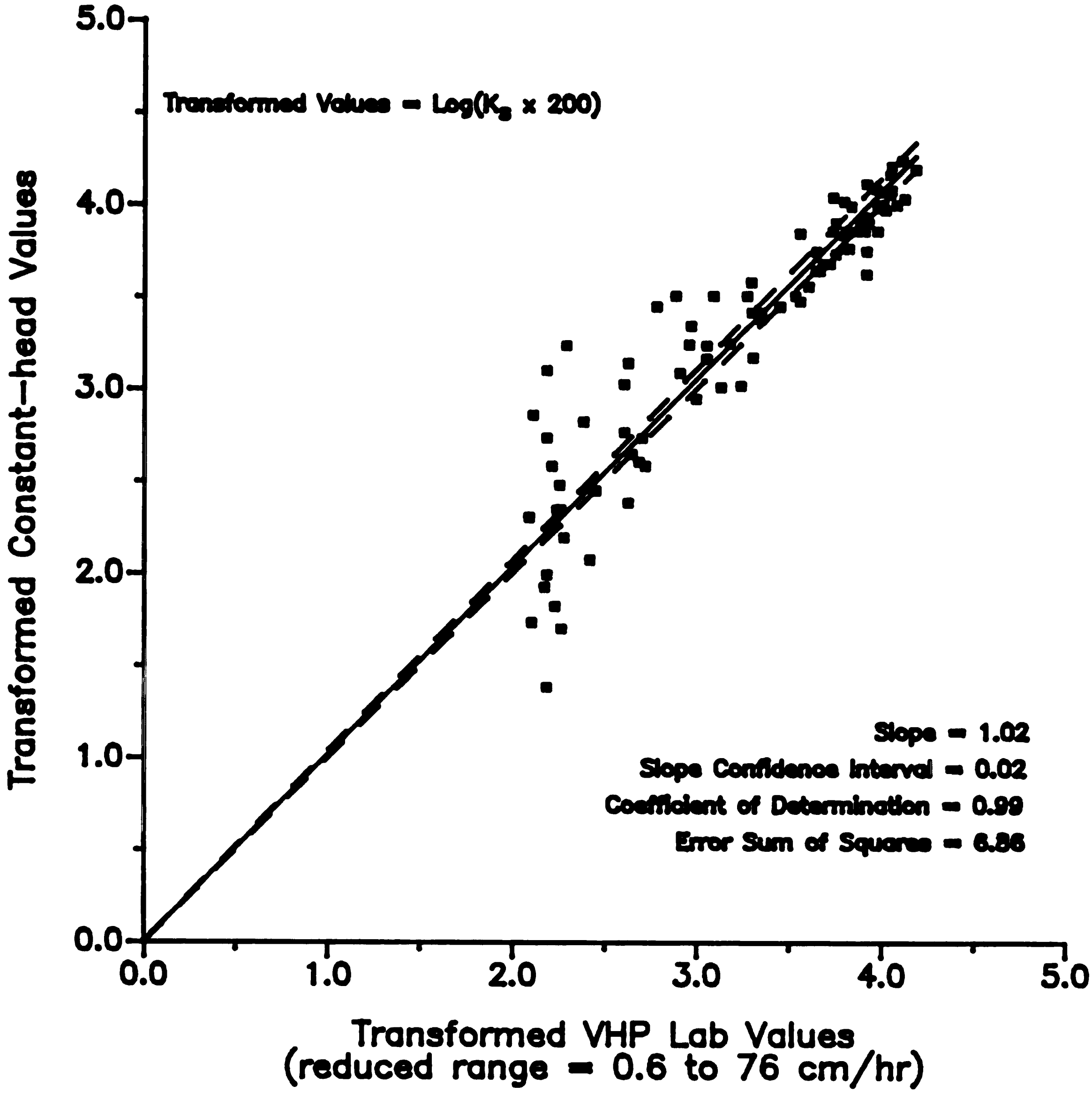


Figure 5

Linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter versus, the same value for the same undisturbed core obtained with the standard constant-head outflow laboratory procedure, for the range of 0.6 to 76 cm/hr.

limits, coefficient of determination equal to 0.99, and a residual standard deviation of 0.28 (7% of full scale). The F value from ANOVA (table 3) was 13,100, significant below 0.005%.

For linear regression with the non-forced zero intercept the results were similar. The slope was 0.94 with a standard error of 0.043, the intercept 0.26, coefficient of determination 0.85, and the residual standard deviation was 0.28. The lack-of-fit test suggests that the forced zero intercept regression line does not fit the data as well as the non-forced zero regression line (lack-of-fit F for forced zero = 0.585 ; F for non-forced zero = 0.560). Although the non-forced zero intercept model demonstrated a better fit, the forced zero model was used to enable comparison to the larger range analysis and was also considered to have an adequate fit for the data.

TABLE 3

Analysis of the variance for linear regression of transformed saturated hydraulic conductivity values obtained with both the velocity-head permeameter and the constant-head methods on the same undisturbed cores, for the range 0.6 to 76 cm/hr.

ANALYSIS OF VARIANCE				
Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Regression	1	1020	1020	13,100
Residual	88	6.86	0.078	
Total	89	1027	Sig. of F Value: < 0.00005	

The results show that at the 95% confidence limit the slope (1.02) is not significantly different from 1. Therefore, the VHP would be expected to accurately predict the constant-head method value 95% of the time given the residual standard deviation of 0.28 about the predicted value in the range of 0.6 to 76 cm/hr.

Figures 4 and 5 and tables 2 and 3 demonstrate that the VHP accurately predicts the value expected from the standard constant-head method. Although the results demonstrate a higher degree of accuracy in the range of 0.6 cm/hr to 76 cm/hr, it cannot be concluded that the VHP produces inaccurate results below 0.6 cm/hr, since the constant-head method is suggested as not being accurate below 0.6 cm/hr (Klute, 1965). Further comparisons between the VHP and methods considered to yield accurate results in this range should be carried out in the future.

The data from the laboratory (figure 4) tend to demonstrate two trends upon visual inspection. The first trend is that for values below 0.1 cm/hr the VHP tends to overpredict the standard constant-head method and the second is that for values above 1.0 cm/hr the VHP tends to slightly underpredict the standard constant-head method. The statistical results demonstrated that the best fit line through all 138 data points is not significantly different from 1.00, suggesting that such trends are not significant enough to warrant changing the model. However, the trends exist and, therefore, must be evaluated when considering the variance and standard error of the data.

Before any conclusions are to be made regarding these trends a consideration of the variance of the data must be reviewed. Mason, Lutz, and Petersen (1957) evaluated the variance between individual cores considered to represent the same soil. The standard deviations of their data for the A, B, and C horizons were found to be 0.32, 0.36, and 0.42 respectively. This deviation, according to the authors, was attributed to variation both in laboratory technique and between cores supposed to represent the same soil. The standard deviation in the laboratory comparison for this study was found to be 0.43 for all values and 0.28 for the range from 0.6 cm/hr to 76 cm/hr. These values compare favorably with the previously mentioned authors' results. Direct comparisons, however, are difficult to make, for although the use of the same core for both test methods should have reduced one source of variation, it may have introduced another source. The introduced source of variation could be attributed to the experimental procedure of always testing the same method first. Klute (1965) points out that continued application of water to a soil sample could have adverse effects on the measured hydraulic conductivity.

The comparison of the VHP to constant-head method indicates that the VHP will accurately predict the constant-head method with a residual standard deviation of 0.43 in the range of 0.01 cm/hr to 76 cm/hr. This residual standard deviation is considered to be acceptable when compared to those described in research by Mason, Lutz, and Petersen (1957). The reduction of the range to 0.6 cm/hr to 76 cm/hr resulted in an increase in the coefficient of determination from 0.86 to 0.95 and a residual standard deviation reduction from 0.44 to 0.28 or a 36.4% reduction. This supports the suggestion of Klute (1965) that the constant-head method is not considered accurate below 0.6 cm/hr. As can be seen in the scatter diagram of all data (figure 4), the variance or residual increases as the saturated hydraulic conductivity decreases. Such findings support the need to further compare the VHP to other methods considered to be more acceptable at lower values of the saturated hydraulic conductivity, such as the falling-head laboratory method. With the evaluated variance for the data from this study in mind, consider now the two previously mentioned trends in the data.

First, consider the variations at very low values (below 0.1 cm/hr) of the saturated hydraulic conductivity. An unpublished article by Merva[1] suggested, based on a field study of the VHP at Ohio State University, that overprediction by the VHP at relatively low values was due to smearing of the soil surface during preparation of the laboratory standard cores. Since this study demonstrates a similar response for both methods on the same core in the laboratory, although at a much lower magnitude, another source of variation must be considered.

Swartzendruber (1969) cites the possibility of non-Darcian flow behavior, where flow is a function of pressure. He makes several references to data that exhibit this phenomenon and suggests that in some way it is associated to the clay particles themselves. He cites four possible theories that could explain such a phenomenon: 1) the modification of water properties within the soil strata resulting in a change in the viscosity of the internal soil water causing non-Newtonian behavior, 2) a change in the porous-medium fabric, suggesting a reversible structural change that would enlarge flow paths and limit dead-end voids, 3) electrical streaming potential demonstrating variation in flow based on the transport coefficient for electroosmotic water flow and the electrical potential streaming gradient, and 4) The osmotic effect of salts. Swartzendruber also reminded us that experimental error could also account for some of this variation in the data.

The underlying assumption of the VHP method is that the equation describing the saturated hydraulic conductivity is a first order phenomenological approach and neglects a possible second order behavior or possible dependence of the saturated hydraulic conductivity on pressure. Before rejecting the VHP method due to the possible second order phenomena, it is noted that other standard methods which are readily accepted for determination of the saturated hydraulic conductivity are based on a similar application of the Darcian approach and also assume a first order response. Though insufficient data exist from this study, other research demonstrates a dependence of K_s on pressure at low saturated conductivities and high clay concentrations (Swartzendruber, 1969 and Baumer[3]).

Figure 6a and 6b demonstrates an interaction of the initial pressure of the water column and a rest period on the K_s determined for the same clay loam core with the VHP. Although, these results are only a instantaneous picture of this interaction they provide some interesting observations. First the possible dependence on the initial pressure of the water column used for testing may adversely affect the measurement of the K_s. This could support the findings of Swartzendruber (1969) and could suggest that the saturated flow for these type soils is non-Darcian. The Darcian approach states that K_s is independent of the

3. Baumer, O., Soil Scientist, USDA, SCS, National Soil Survey Laboratory, 1987. Personal Communication.

Fig. 6a

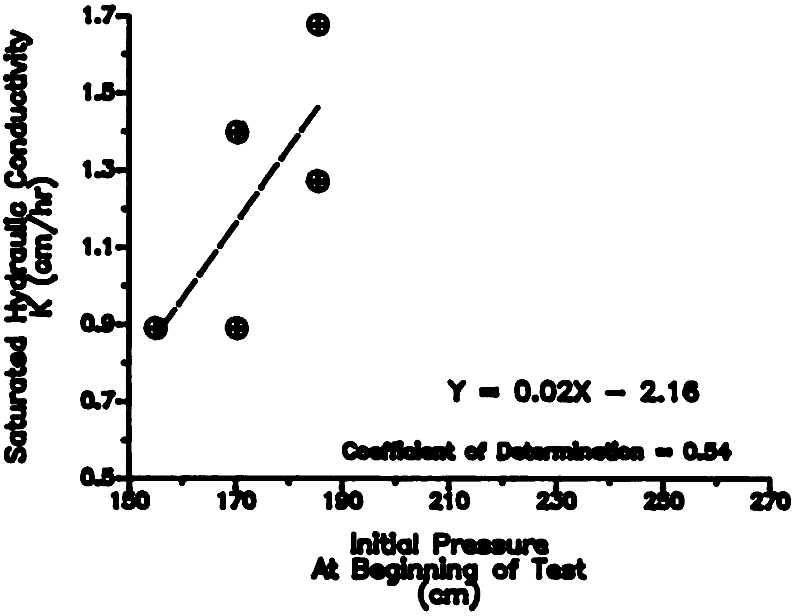


Fig. 6b

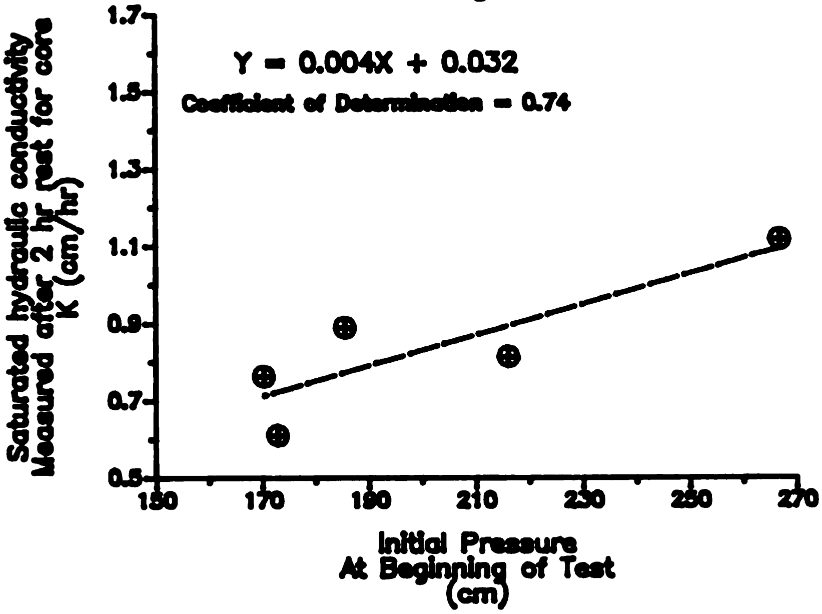


Figure 6

Figure 6a and 6b represent the interaction of initial pressure and time on the results of the measured saturated hydraulic conductivity for a clay loam (determined by the velocity-head permeameter). Both 6a and 6b represent the same core with a elapsed time of 2 hours between test periods where the core was allowed to dry during this time.

pressure or head at which it is measured. The results of figure 6a and

6b may suggest that for some soils a second order phenomena exists where K_s is independent of the pressure or head at which it is measured. Secondly is the effect of repeated wetting and drying of a soil core on the K_s. The difference between 6a and 6b (although, measured at different pressures) might suggest some irreversible phenomena occurring during wetting and drying of the soil. Since this phenomena occurs continuously in the soil regime this phenomena suggests that further study be done to characterize these affects. The VHP could be a useful tool to enable further evaluation of this interaction of time and initial pressure both in the laboratory and in the field.

The second noted variation in the data is the slight underprediction of the VHP at higher saturated conductivity values. Again, the variation is small compared to the prediction model and can be disregarded, but none-the-less must be explained. Little information is available in the reviewed literature as to possible causes of this phenomenon. Considerations must include: 1) the interaction of clay particles (Swartzendruber, 1969), 2) entrapped air and/or microbial effects as outlined by Klute (1965), and 3) lack of validity of Darcy's law for values above 60 cm/hr (Klute, 1965). This study included 4 values measured by the VHP that were above 60 cm/hr and 7 values measured by the constant-head method that were above this value. Klute (1965) defines the deviation as an expected decrease in K_s with an increase in pressure. A plot of the values obtained in the laboratory with the VHP is shown in figure 7. This figure, shows that the saturated sand core hydraulic conductivity decreased with time or consecutive run number. It is also important to note that the initial value obtained with the

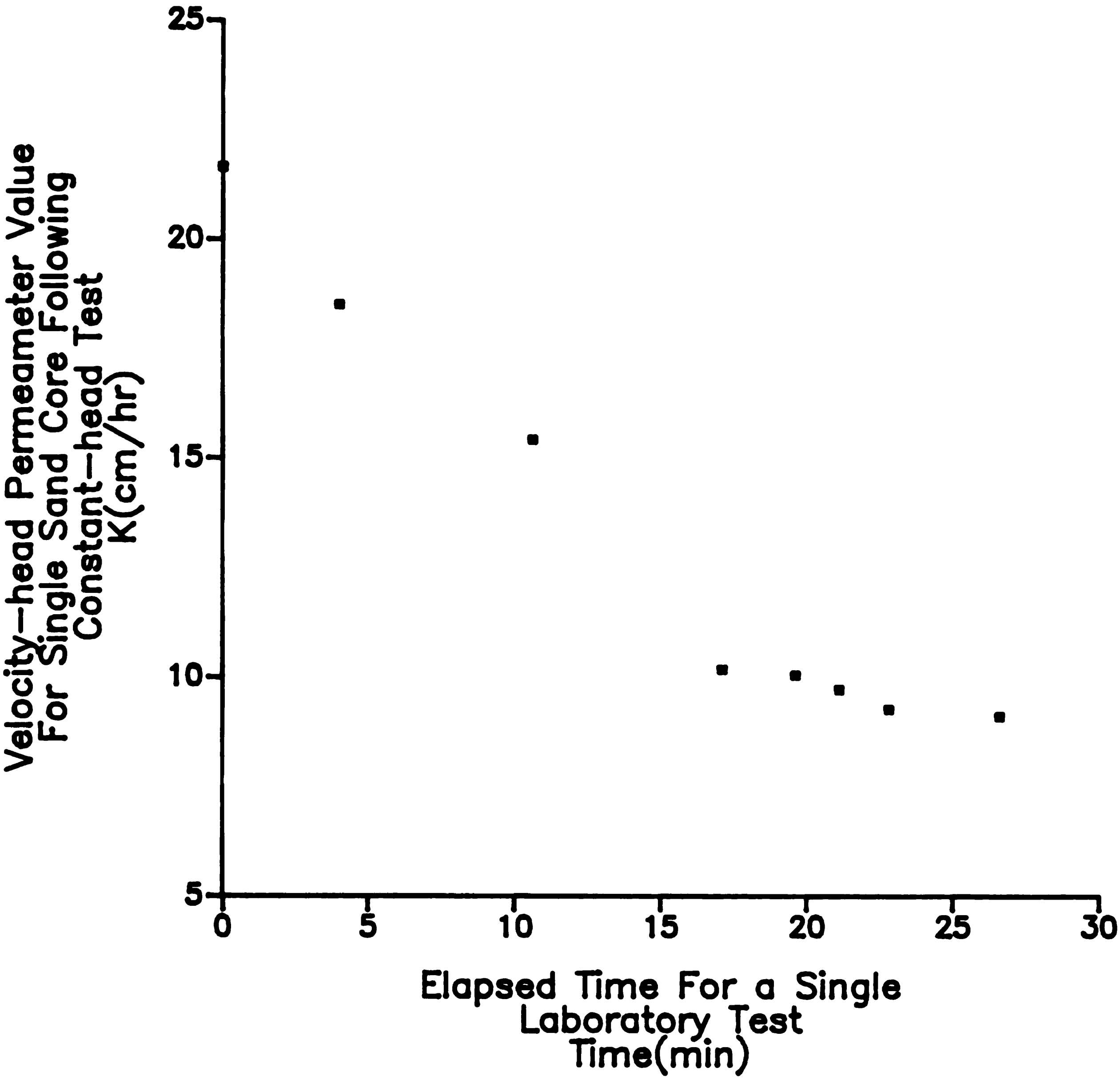


Figure 7

Plot of values obtained with the velocity-head permeameter during consecutive runs on an undisturbed sand core immediately following the constant head test.
VHP is very close to the value obtained with the constant-head method.

This aided the correlation since, as mentioned in chapter 3 under A. Laboratory Procedures, almost entirely the first or second reading was used for this comparison.

Possible sources of error introduced by this study may include the use of smaller diameter cores and the sequence of testing. The smaller diameter core (3.81 cm) as explained earlier, was determined necessary to eliminate the influence of macroscopic structure in the soil such as biopores. This decision, although necessary, may have introduced a new type of error into the experimental procedures. With shrinking and swelling of clay soils, the undisturbed soil core is considered to be influenced by the wall effect to some degree (McIntyre et al., 1979). In this study the soil cores, except for a few sand cores, were tested within 48 hours to prevent unnecessary drying and subsequent shrinkage of clay. The influence of the wall effect, if present, would be considered to increase as the diameter of the core decreased. In order to quantify this wall effect, more research would need to be conducted and comparisons made between small and large diameter cores. Such comparisons should be done with homogeneous soil cores ranging from clay through sands to enable thorough evaluation. It is likely that such comparisons could be related through the use of a K_R term (the potential change in the hydraulic conductivity related to core radius). Such a term could be directly related to the difference between the dimensionless ratios of cross sectional area to the wall area for cores of different radii.

The reason for suggesting such a correction factor was based on the observation that, for sand cores, the VHP tended to produce a lower value than the constant-head method with increasing runs while the first run appeared to approximate the constant-head method. This might be attributed to microbial action in the soil except for the observation that the four consecutive runs of the constant-head method did not produce a similar decrease in the hydraulic conductivity. In addition, visual inspection of cores revealed no evidence of microbial presence. The difference in readings may be attributed to the increased pressure created by the VHP possibly forming a seal at the wall, thereby overcoming the wall effect. If, in the future, the VHP were compared to the falling-head method with similar initial pressures, this type error would be constant between the two methods.

The second suggested source of error relates to the uniqueness of this research. Each core was tested via two methods, first with the standard constant-head method and second with the VHP method adapted to fit onto the same core. After the review of literature and verbal communication with the Department of Crop and Soil Sciences at Michigan State University it appears that this "double test" method has not been done before. Most comparisons tend to use the averages of a group of disturbed or undisturbed cores rather than results from the same core. As mentioned above, this may have introduced a new type error since the literature suggests that continued water flow through the soil may reduce the hydraulic conductivity (Klute, 1965). A review of the collected data suggests that such a reduction was not a problem in this research since the data seem to be distributed both above and below the best fit line. Future testing could include reversing the order of testing on cores from "homogeneous soils" that produce good replication or on hand-packed, disturbed cores to verify this hypothesis. The statistical results of the comparison of the VHP to the constant-head method did not indicate a need for such testing. Figures 6 and 7 however, demonstrate that K_s is in some way related to the time and continued wetting by the reduction of K_s with time.

D. Field Results

The second part of the accuracy testing of the VHP was to compare in situ values to those determined in the laboratory using the constant-head method. Figure 8 is a scatter diagram of the transformed in situ values versus laboratory values obtained using the constant-head method. The in situ values represent an average of 2 to 3 measurements with the VHP and the laboratory measurements represent the average value of 3 to 5 cores. The analysis of variance for the zero forced intercept model is found in table 4. The resultant slope of the best fit line (figure 8) was 1.04 ±0.05 with a standard error of 0.025 and a residual standard deviation of 0.42 (12% of full scale). The F value from ANOVA (table 4) was 1686, significant below 0.005%.

The regression line using the non-zero forced intercept yielded a slope of 1.12 with a standard error of 0.089, coefficient of determination 0.75, a intercept of -0.23, and a residual standard deviation of 0.42. The lack-of-fit test suggests that the non-zero intercept regression line does not fit the data as well as the forced zero regression line (lack-of-fit F for forced zero = 0.752 ; F for non-forced zero = 0.754).

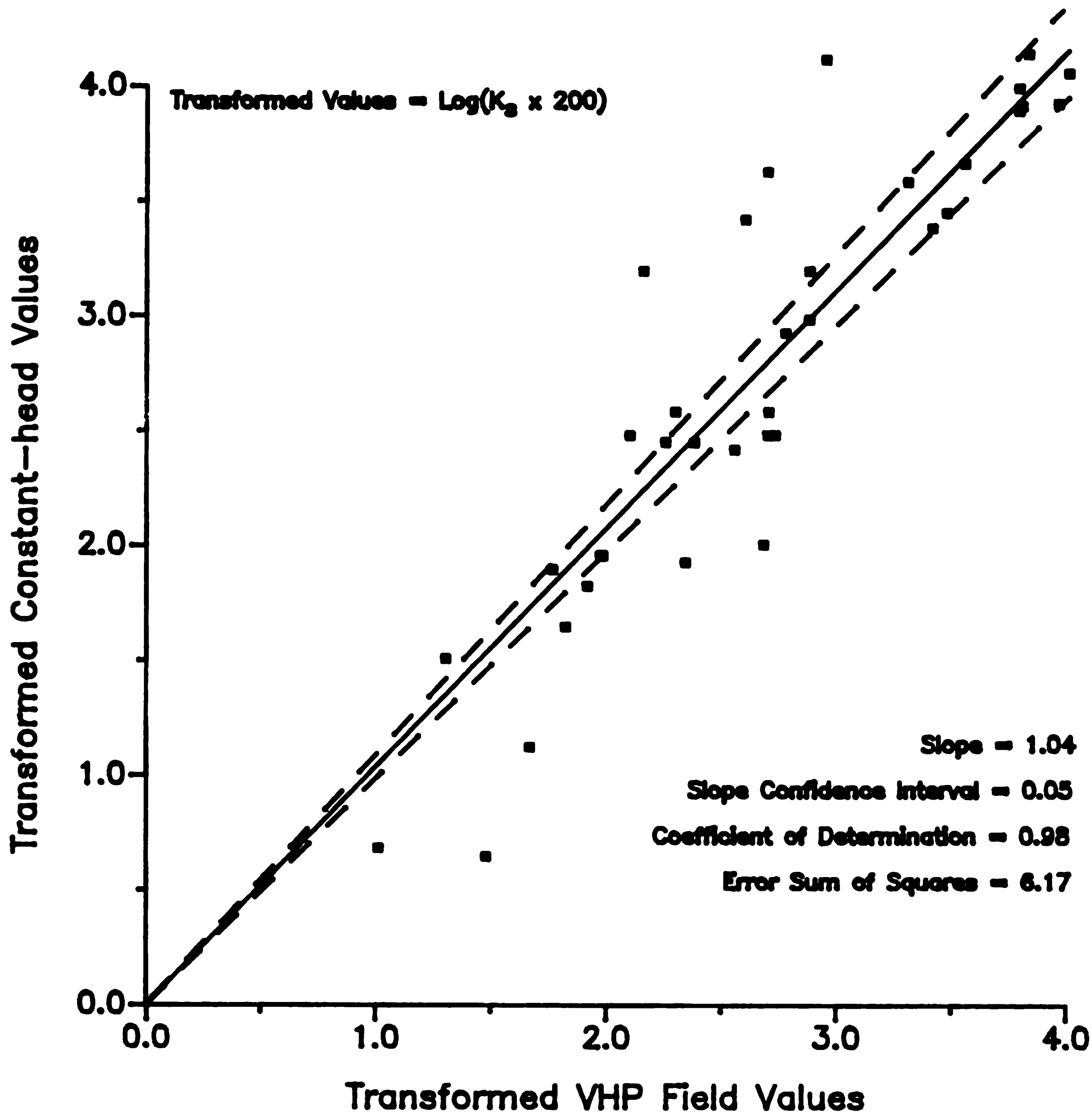


Figure 8

Linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter in situ versus, the same value for undisturbed cores obtained with the standard constant-head outflow laboratory procedure, for the range of 0.05 to 51 cm/hr.

TABLE 4

Analysis of the variance for linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter in situ versus, transformed values obtained by the constant-head laboratory procedure for the same site.

ANALYSIS OF VARIANCE				
Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Regression	1	297	297	1686
Residual	35	6.167	0.176	
Total	36	303	Sig. of F Value: < 0.00005	

The results show that at the 95% confidence limit the slope (1.04) is not significantly different from 1. Therefore, the VHP in situ would be expected to accurately predict the constant-head method value 95% of the time given the residual standard deviation of 0.42 about the predicted value in the range of 0.05 to 51 cm/hr. Furthermore, the coefficient of determination was 0.98 falling within the limits (above 0.80) described by Bender, Douglass, and Kramer (1982), to accept the correlation of the two methods.

An additional comparison was conducted between the field values and laboratory measured VHP values to determine the variance involved between field and laboratory. These laboratory measurements were again taken on the same cores as the constant-head method. The scatter diagram for this comparison is found in figure 9. Table 5 gives the analysis of the variance for the comparison. The standard deviation is

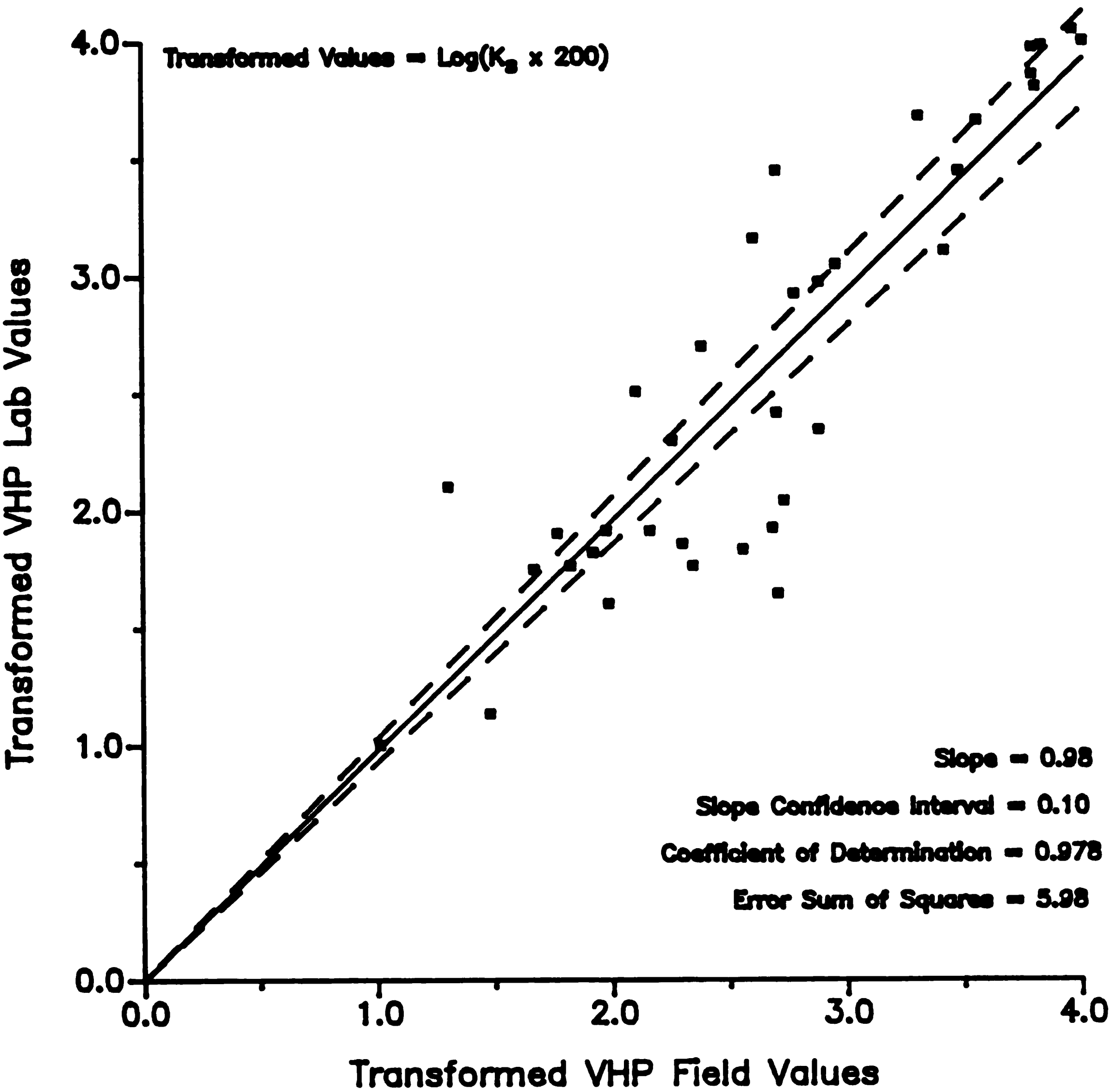


Figure 9

Linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter in situ versus, the same value for undisturbed cores obtained with the velocity-head permeameter in the laboratory, for the range of 0.05 to 51 cm/hr.

0.42 for the data. This compares closely to the value of 0.42 for the standard deviation between the two different methods. Finally, the 0.42 standard deviation compares favorably to the 0.32, 0.36, and 0.42 standard deviations reported by Mason, Lutz, and Petersen (1957).

TABLE 5

Analysis of the variance for linear regression of transformed saturated hydraulic conductivity values obtained with the velocity-head permeameter in situ versus, transformed values obtained with the velocity-head permeameter in the laboratory for the same site.

ANALYSIS OF VARIANCE				
Source of Variation	Degrees of Freedom	Sum of Squares	Mean Square	F Value
Regression	1	265	265	1551
Residual	35	5.98	0.171	
Total	36	271	Sig. of F Value: < 0.00005	

This comparison between field and laboratory again demonstrates the value of the VHP as tool for determining the field saturated hydraulic conductivity. Although the coefficient of determination for this comparison is lower than the laboratory comparison of VHP with the constant-head method, the lack of fit test and residual analysis reveals that the data are evenly distributed about the regression line. The F value for the lack of fit test was below 1.0 (0.74) which means, according to Draper and Smith (1981), the data contain no bias. The comparison of the VHP in the field to the VHP in the laboratory demonstrated a lower correlation (coefficient of determination 0.81). Here again, the lack-of-fit test suggests that the data are evenly distributed about the regressed line since the F value equals 0.67. The fact that both comparisons yield the same residual standard deviation suggests that the variance cannot be attributed to a difference between the VHP and constant-head method. On the other hand, the variance does compare very closely with that of Mason, Lutz, and Petersen (1957) when differences in both the soil and laboratory testing are taken into account. Based on these comparisons one should accept the hypothesis that the two methods are correlated and that the VHP in the field will predict similar results to that of the constant-head method. As the comparison shows, 83% of the difference falls within one order of magnitude. Since the saturated hydraulic conductivity varies so greatly it is probably adequate to talk about values for soils in orders of magnitude, therefore, the results obtained by the VHP are considered sufficiently accurate.

E. Use of the VHP For Site Inspection

Figure 10a and 10b shows the results of the field tests as they compare to the acceptance or rejection of sites by the sanitarians. The horizontal line on both graphs represents the base or minimum value of K_s (2.54 cm/hr or 60 min/in) required to provide an adequate system using standard design techniques.

Figure 10a shows all sites that were rejected by the sanitarians, also were rejected by the results of the VHP. Of these rejected sites numbers 3 and 11 would be the only sites that could be considered borderline cases, where a specially designed septic system may have

Fig. 10a

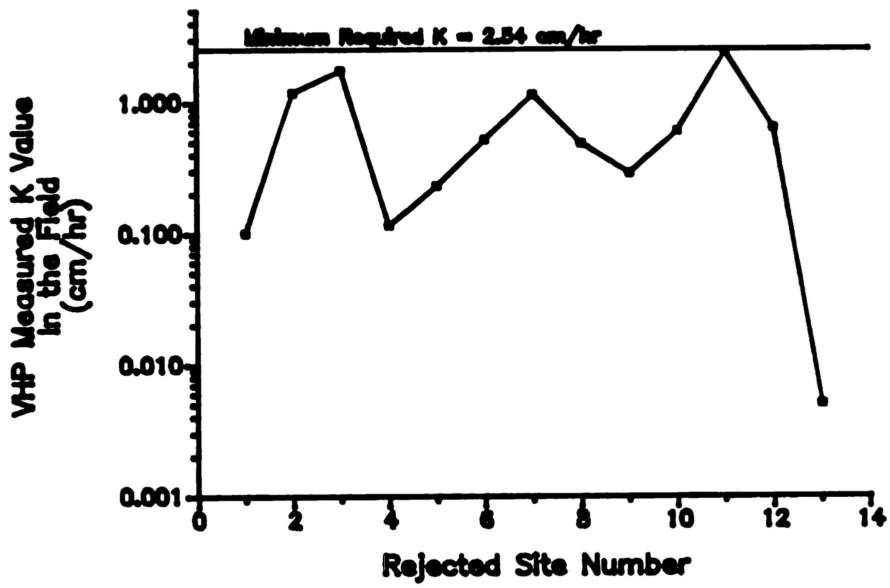


Fig. 10b

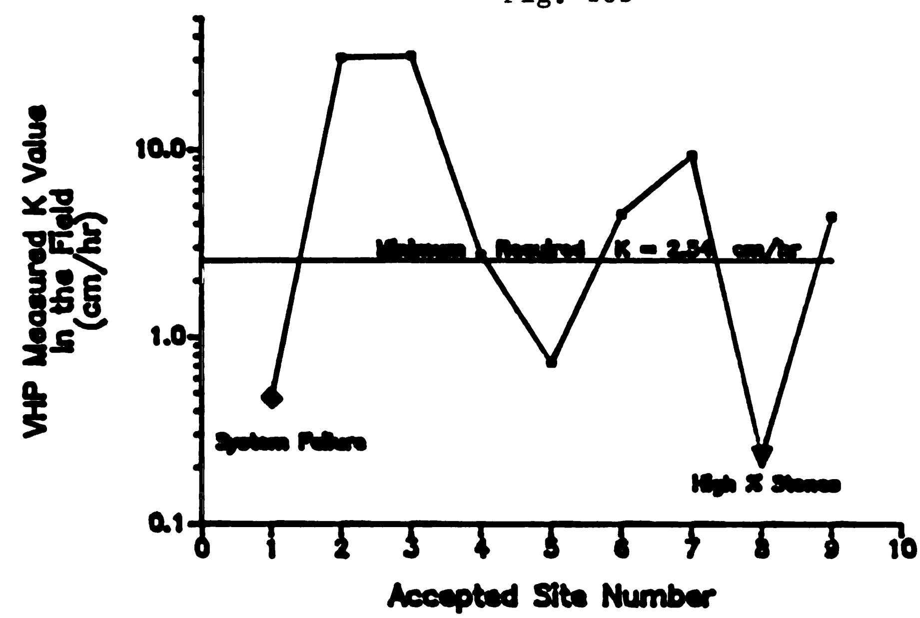


Figure 10

Figures 10a and 10b represent a comparison of rejected and accepted sites by county sanitarians to the results obtained by the VHP in situ. compensated for the slightly lower K_s. It must be noted that the results of the VHP used for site rejection did not include other site information that may have contributed further to the rejection of these sites.

Figure 10b shows three sites that were accepted by sanitarians, to fall below the minimum accepted K_s. This does not mean that the sanitarian made a mistake by accepting a site that should have been rejected. The measured values were obtained at a depth of 71 to 76 cm and do not reflect possible coarser material at a greater depth. From these findings it can be concluded that the sanitarians made an accurate decision for all field sites tested.

The first site to fall below the accepted minimum value of 2.54 cm/hr in figure 10b was at the location of a failed system. This failed system was estimated to be from 30 to 40 years old. The K_s measured with the VHP for this site was 0.47 cm/hr. Given this situation acceptance of the site is essential but, design of a system should also be modified to take into consideration the lower K_s value. In this case it was reported by the sanitarian that the designed system for this situation made some allowance for the low hydraulic conductivity. A second site included in figure 10b was difficult to measure with the VHP due a large amount of stones being present in the soil. The value of K_s obtained with the VHP for this site was 0.23 cm/hr. This site would also require special design considerations due to the findings of the VHP. Finally the third value falling below the minimum accepted value was again measured at a depth of 76 cm. At this site a coarser material was found below 76 cm with an estimated higher hydraulic conductivity. The acceptance of the site was conditional and based upon requiring absorption system installation in the coarse material. All three points represent a need for special design considerations but do not necessitate rejection of the given sites.

The results of the VHP in the field as compared to the evaluation by county sanitarians are inconclusive. Although the results compare favorably (figure 10a and 10b) they do not demonstrate the overall necessity of VHP use for every situation. The instrument cannot replace the trained sanitarian, but can be a great aid to site evaluation. A trained sanitarian can easily determine if a soil would be expected to have a saturated hydraulic conductivity of greater than 2.54 cm/hr and meet the acceptable standards. In such instances it would be needless to incorporate increased costs associated with evaluating the saturated hydraulic conductivity. The second aspect has to do with the presence of limiting layers in an otherwise acceptable site. It is difficult to assess the significance of a limiting layer found in certain soil strata though the soil saturated hydraulic conductivity above and below that layer is quite high. In the case of a continuous limiting layer causing reduced flow rates, the trained sanitarian may make allowances for such problems in design and construction of septic waste disposal systems. One application of the VHP is that the instrument is capable of determining the reduction in the conductivity through such a limiting layer. With the magnitude of the problem known, a more applicable site specific design can be made resulting in decreased costs in some cases and better protection against failure in others. A second application relates to sites having existing questionable or failed soil absorption systems. It is at these locations that the sanitarian or consultant must be able to identify the expected saturated hydraulic conductivity for purposes of design and implementation.

By demonstrating the reliability of the VHP as used in the field from this study it is suggested that this method be adopted as an acceptable method of parameter evaluation for sites with possible limitations. Once the magnitude of the conductivity is determined an efficient design can be submitted and approved for implementation. In order to maintain honesty and reliability either a reputable consultant or the county sanitarians would have to perform such testing for the landowner. The use of the percolation test in the past took as many as several days to evaluate the saturated hydraulic conductivity and then was stated as producing questionable results. With the simplicity of the VHP it is much easier to obtain a reliable reading and reduce errors in evaluating the K_s of the soil. This coupled with the short time required for testing ensures that the VHP method is an accurate yet inexpensive method to evaluate problem soils for potential home waste disposal sites.

Limitations associated with the VHP yielding a point rather than a field measurement can be eliminated by taking multiple measurements at various locations in the soil profile. Once field K_s estimates are made, then point evaluation with the VHP can assist in special design considerations. This does not change the fact that some soils are not suitable for septic waste disposal systems. The final result is that well trained sanitarians are essential and that their decisions can be aided by the use of the VHP.

F. Associated Results

Figure 11 is a plot of the saturated hydraulic conductivity versus the bulk density for all available values. The conclusion is that the bulk density for the cores sampled in this study show no direct correlation to the corresponding saturated hydraulic conductivity values. Finally, a comparison of the vertical versus horizontal saturated hydraulic conductivity is found in Figure 12. The line in this figure is not a regression line, but a line with slope equal to one. Figure 12 demonstrates that, although a trend exists for the horizontal value to mimic the vertical value, significant variations exist. The result, although limited by the small amount of data collected, would suggest that conductivity in both the vertical and horizontal direction needs to be measured and compared for design purposes.

G. Special Testing With the VHP

1. Existing System

The use of the VHP as an aid in testing an existing system of known history proved to be interesting. The system as described in the procedures section had been literally untouched since its installation. Several research studies have looked at and evaluated existing systems based on ponding and loading rates but, fewer have looked more closely at the effects of the clogging mat in the extended operation of a system. What appeared to happen in the system studied was a change in the clogging mat based on location and loading at specific points.

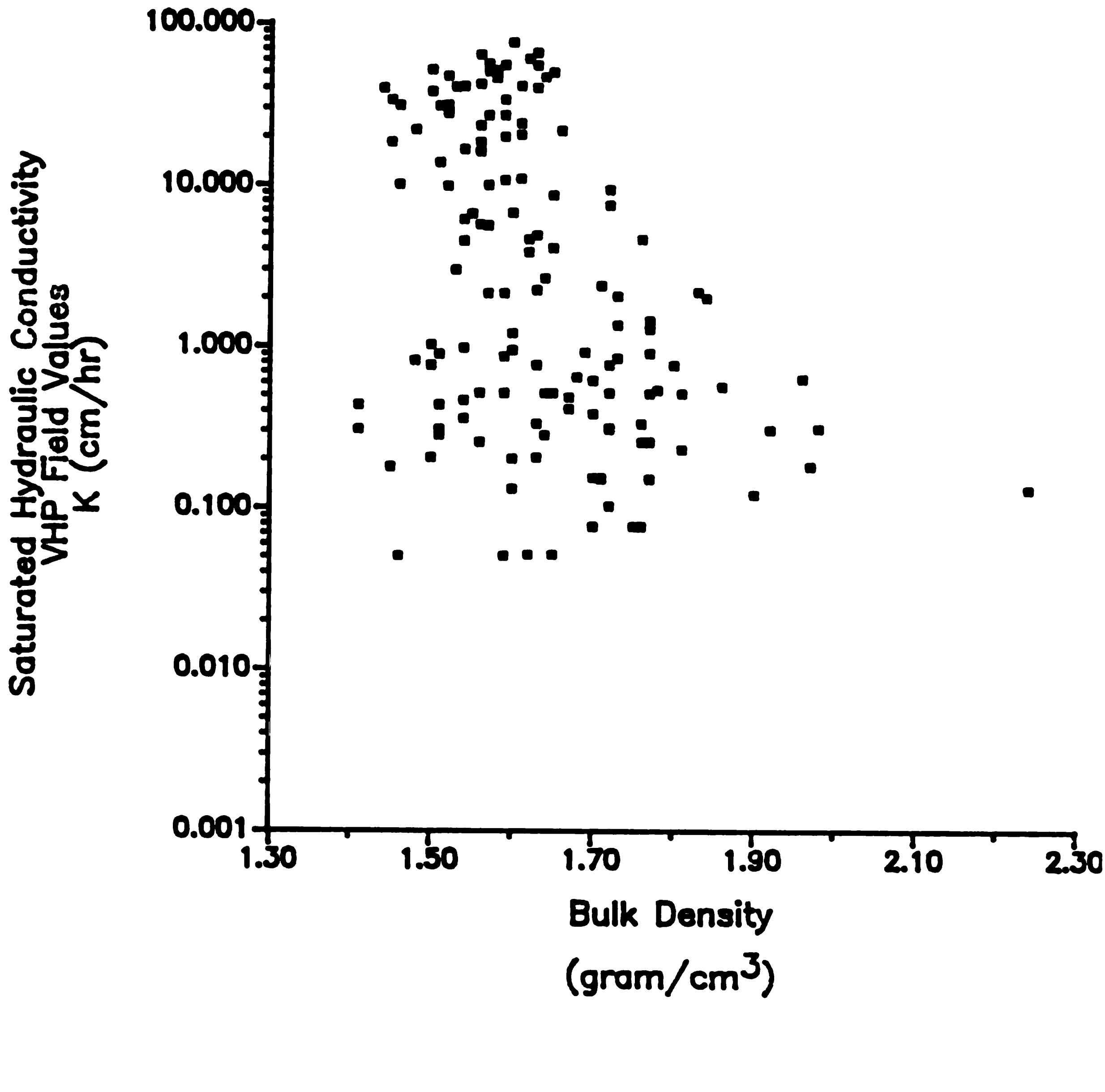


Figure 11

Plot of bulk density versus K_s obtained with the velocity-head permeameter from undisturbed cores in the laboratory.

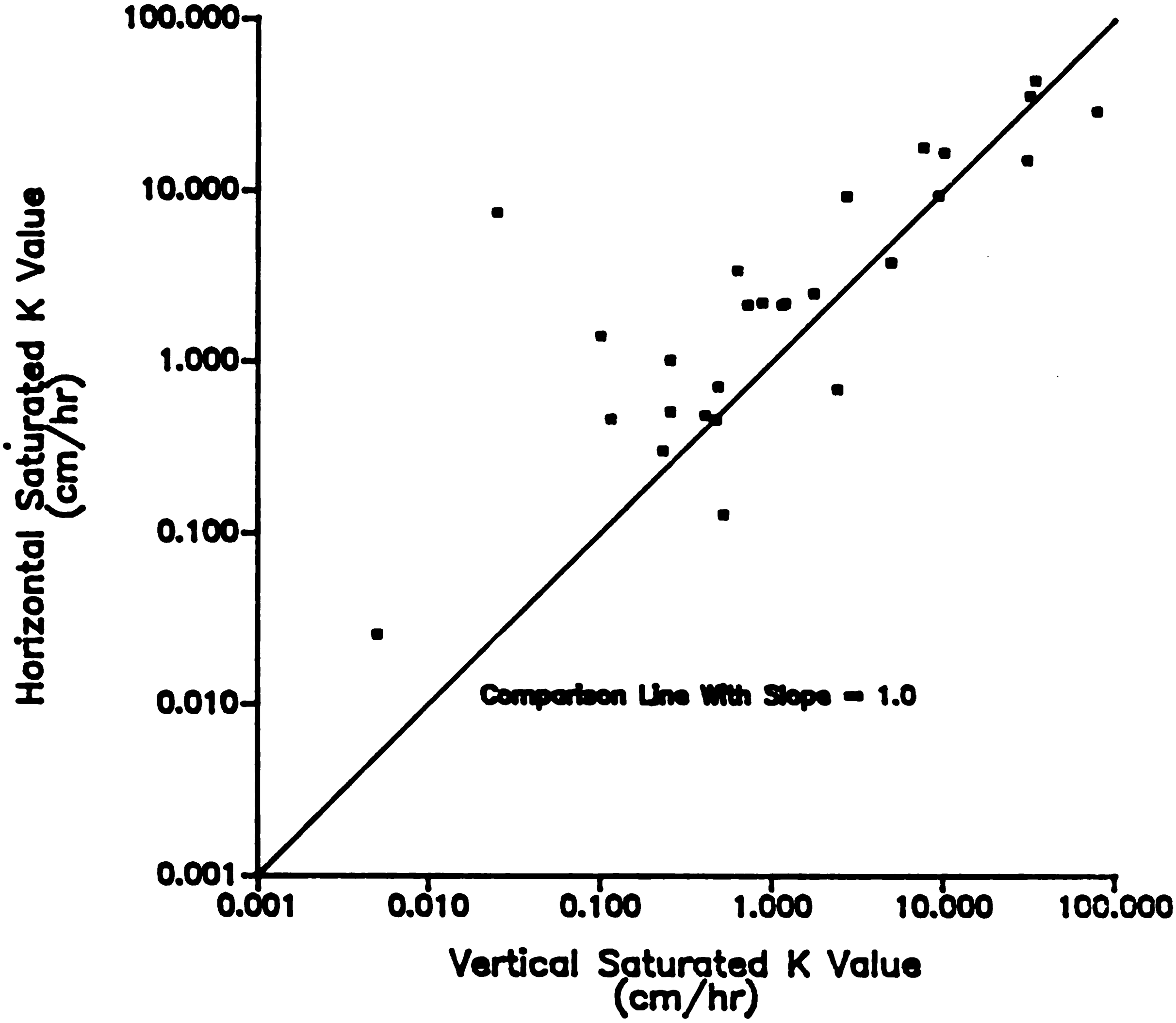


Figure 12

Plot of vertical K_s versus horizontal K_s obtained in situ with the velocity-head permeameter. A line having a slope of 1 is also plotted to allow comparison.

Although the septic tank had never been pumped the system appeared to be functioning quite well. This is attributed to the parent material being a homogenous, medium to fine sand with a good K_s evaluated at 8 cm/hr. Several conclusions can be drawn after looking at both the proximal and distal end of the system. The first is that the distal end of the system has been resting since a reduction in the family size has taken place, consequently, the limiting affect of the clogging mat on K_s, has been reduced. A second conclusion is that the side wall of the trench is probably more important in a functioning system than the trench bottom. This second point is supported here by the point measurements taken at the stone soil interface with the VHP. The results demonstrated that horizontal K_s significantly increased with decreased depth in the trench. This gradient was not so obvious at the resting, distal end of the system. This underscores the importance of the VHP for site evaluations since no other instrument so easily measures both the horizontal and vertical saturated hydraulic conductivities. A final conclusion is that through resting, the system's increased worm and microbial action reduce the clogging mat and its negative effects on effluent movement. This was supported by the addition of 100 gallons of water to the system without subsequent ponding in the distal end. These results help show the benefit of designing systems that incorporate rest periods during operation.

2. Special Site Evaluation.

A difficult site to evaluate was one where a mound of pond spoils had been placed on top of grass/weed vegetation several years earlier. In the original design, the system was to be installed such that the spoil would serve as the mound for the system. The major concern was the condition of the interface and its possible limitations due to the decayed vegetative matter. Through testing with the VHP, a limiting layer at this interface was identified and the original design was modified. It was proposed to move the location of the system and to build the mound from imported sandy loam. However, testing of the surface excavation revealed a low K_s in the top 20 cm of soil, presumably due to high organic matter and wetting in the clayey surface. The VHP findings thereby enabled a more accurate design which was site specific and will be expected to have a long life given proper maintenance of the system.

3. Subirrigation Site Testing

Special testing of Michigan State University's campus subirrigation demonstration site on the main campus farms suggested an interesting phenomenon in the transport of water from drain tubes into the soil. The system was tested in two locations, one that was working and one that demonstrated some problems. The only difference between the two areas was a layer of sandy loam about 7.6 cm thick found near the surface in the location that demonstrated no problems. The theory then developed from these findings was that the clay to clay loam soil had significant biopores and cracks which allowed water to flow easily in the vertical directions. Piezometers located over the drain tubes verified this. The lateral movement of water, however, was limited to the sandy loam layer near the surface which was missing in one location. Without the presence of this layer the movement of water in the horizontal direction was significantly reduced to the point that crop yields were effected.

4. Muskegon Waste-Water Facility

The water ponding problem of this facility was very unique due to management practices. Coarse sand was the predominant soil in the location tested and would be expected to transmit large amounts of water. After leveling the area, a water ponding problem developed. With only one day of VHP testing it was found that the problem was surface sealing and probably caused by colloidal suspension. The same area when it had been maintained with cover crops did not exhibit such a problem. It appears that the cover crop prevented ponding by maintaining openings through the surface layer which enabled infiltration. The cover crop probably also reduced the impact energy of the applied waste water and kept colloids from being dislodged from the organic matter at the surface. Once the VHP fully exposed this problem a design was developed to help effect a solution. Due to the legalities involved, another consulting firm was called in to evaluate the problem. After several days of field work and evaluations, the firm submitted a solution similar to that resulting from VHP research. It is obvious from this example of a 75% reduction in field time that use of the VHP can result in tremendous cost savings.

V. CONCLUSIONS

The VHP and the constant-head laboratory method have been successfully compared and demonstrate a close correlation between the two methods in the range of 0.01 to 76 cm/hr. The results prove that the VHP will reasonably predict K_s obtained from the constant-head laboratory method. Use of the VHP for site inspection of potential waste water soil absorption systems proved advantageous in situations where it was difficult to estimate the actual K_s of certain soils. K_s values obtained with the VHP on these difficult soils proved very useful for design purposes of matching a system to the soil conditions.

A. Validity of the VHP

1. Laboratory

A total of 155 cores taken from soils in Michigan were tested in the laboratory. Of the 155 cores 16 were determined to be outliers for various reasons, leaving 139 cores used for the comparison. To help eliminate error in the results attributed to soil variations, a new method was used to provide the data for comparison. The application of this new method was simply to use two different tests on the same core rather than taking averages of cores from the same site. This would be expected to reduce error attributed to soil variation, while at the same time causing an increase in error due to extended flow at saturation and necessary movement of cores to enable this approach. A least squares linear regression was performed on the data to determine if correlation existed. The VHP when used in the laboratory and compared to the constant-head laboratory method on the same core, yielded a good correlation between the two methods (R^2=0.98) with a slope of 1.02, a standard deviation of 0.43, and a F value of 6,248. The bias that automatically exists in the data due to a large range of values was decreased through the use of a log-log transformation, enabling a relative comparison for all values of K_s. Inspection of this comparison reveals that the variation in the data increases as the saturated hydraulic conductivity decreases. This is not surprising since it is known that the constant-head method tends to have limited accuracy at lower values (Klute, 1965). By limiting the range of values to 0.6 to 76 cm/hr the data produce an even better correlation (R^2=0.99) and the same slope of 1.02, a new standard deviation of 0.28, and a F value of 13,100. The error involved with the data is not additive based on residuals found in the analysis. A plot of the residuals exhibiting a funnel shape suggests a certain bias in the data for larger values of K_s (Draper and Smith, 1981). Based on variations reported for comparisons of cores using the same method by Mason, Lutz, and Petersen (1957) the variation found in the data from this study is within the same limits, yielding evidence that supports the verdict of the existence of a correlation between the two methods.

2. Field

Comparison of the values obtained in the field using the VHP to the cores tested in the laboratory with the constant-head method demonstrated a good correlation (R^2=0.98) with a slope of 1.04, a standard deviation of 0.42, and a F value of 1,686. The variation found in this comparison was evenly distributed throughout the range investigated. The comparison was based on site averages and would thereby be expected to incorporate an additive type error as a result of the soil variations. The variance in the results was determined to fall within acceptable limits of those pointed out by the authors above. This variation was shown to be acceptable by a comparison of field values to laboratory values measured with the same instrument. Since variation in the data was greater for the comparison of the same instrument from field to laboratory, it can be concluded that the variance for the different methods included no special source of error introduced through the comparison of these two methods.

B. Application of the VHP to Site Evaluation

The VHP is a useful tool for evaluating sites with soils that are expected to have limitations for application of septic effluent due to low K_s. The instrument would not be necessary for all site inspections since the county sanitarian is quite capable of estimating the K_s for many soils. The application of the VHP is most suited for those soils that are described as limiting or questionable by the sanitarian. The results obtained by the VHP can be used to support the sanitarian's decision should a site be brought before a board of appeal. Should a special design be required for a poorly suited site, the VHP can yield values that would be critical for proper design.

The analysis of the bulk density for most of the 139 cores revealed the expected trend of increasing K_s with decreasing bulk density. Linear regression of the data, though, revealed a lack of linear correlation between these two soil parameters. This suggests that bulk density is not a good parameter to use to estimate K_s. Finally, a comparison of the field horizontal and vertical saturated hydraulic conductivities demonstrated that the two values tend to be similar for each soil, yet may vary by a factor of 10 to 100. For this reason it is imperative that the horizontal K_s be considered along with the vertical measurement, especially when considering design of wastewater treatment systems.

VI. Recommendations

1. The variation in the data obtained at K_s values below 1 cm/hr suggests that the VHP should be compared to another method at these values. The falling-head method would be considered to be acceptable based on the acceptance of its values for lower values of K_s (Klute 1965). Such a comparison may enable a stronger confidence in the VHP at lower values and help to describe problems encountered with the constant-head method at these same values.

2. Testing of more cores for each site will also assist in describing the differences between methods in the lower range. This research used only 3 to 5 cores for each site and made analyzing the variance for a specific site difficult. If more cores are tested it will be much easier to make comparisons related to variance in the data.

3. A comparison between the VHP and other field methods used above a water table would be helpful. Since the VHP can predict results obtained by the constant-head method a further comparison to field methods would be of interest.

4. A finite element approach to solve for flow within the core head would be helpful to better understand the significance of the wetted front upon leaving the core. Such an investigation was beyond the scope of this study, but is now underway by others could help to prove the hypothesis that a small increase in the K_s occurs as the wetted front leaves the core. It may also provide understanding as to the exceptionally low R^2 values found during field testing in some soils.

5. The VHP could be used to evaluate the dependence of the saturated hydraulic conductivity on the initial pressure for clay soils. Although the mechanism by which this phenomenon occurs is unclear, the results of measuring K_s on soils of varying clay contents would prove helpful in understanding the relationship of these two parameters.

6. One existing soil absorption system was evaluated in this research with the VHP. In the future an analysis of several existing systems would be of interest since the VHP could enable careful study of horizontal and vertical flow in existing systems. Further use could also help in describing the significance of clogging mats in different soils and the benefits of rest periods on these systems.

7. Since the VHP is a point measurement, more field testing should be done to determine the necessary number of point tests required to adequately describe a complete soil profile. This could be accomplished by comparing the VHP to a below water table method such as the drain tube method which, is considered to give the most accurate field value for the hydraulic conductivity.

8. Testing of the VHP should continue for various soil types to better understand limitations of the instrument due to soil type if any. This should continue both in the United States as well as in other countries such as China, since the VHP is such a versatile instrument requiring no supportive equipment other than the hand held calculator.

APPENDIX A

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Revision Level: E00XM

Interface: Parallel/ CENT

Line Ending: CR,LF

Data Encoding: ASCII/7

Fonts Available:				Job Status:	
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APPENDIX B

B. Laboratory Data

Site No.	Constant-head	VHP	LOCATION	SOIL SERIES
1	1.00	.90	Ingham Co.	Aubbeenaubbee
	3.30	1.20	Section 30	
	1.10	.90	Meridian Twp.	
	.10	.10	R1.28W,T3.46N	
2	.30	.60	Ingham Co.	Capac Loam
	1.50	1.40	Section 26	
	.20	.20	Meridian Twp.	
	.04	.10	R0.06W,T3.47N	
	.10	.80		
3	.50	.51	Ingham Co.	Marlette Fine
	.40	.31	Section 12	Sandy Loam
	.40	.40	Williamstown Twp.	
	.20	.20	R1.29E,T4.13N	
4	3.40	.30	Ingham Co.	Marlette Fine
	.40	.10	Section 10	Sandy Loam
	2.70	2.50	Williamstown Twp.	
	3.50	.10	R1.06E,T4.17N	
5	.80	.90	Ingham Co.	Spinks Loamy
	.50	.30	Section 27	Sand
	.40	1.10	Wheatfield Twp.	
			R1.06E,T2.41N	
6	13.90	3.00	Clinton Co.	Gilford Sandy
	15.90	6.10	Section 30	Loam
	7.40	10.00	Bingham Twp.	(pit 1)
			R2.24W,T6.32N	hor. clay layer
7	6.10	4.00	Clinton Co.	Wasepi Sandy
	8.60	5.60	Section 30	Loam
	8.70	4.50	Bingham Twp.	(pit 2)
			R2.24W,T6.32N	hor. clay layer
8	29.00	31.00	Clinton Co.	Wasepi Sandy
	39.00	38.00	Section 30	Loam
	40.00	40.00	Bingham Twp.	(pit 2)
			R2.24W,T6.32N	vert. sand layer
9	29.00	33.00	Clinton Co.	Wasepi Sandy
	14.00	14.00	Section 30	Loam
	28.00	22.00	Bingham Twp.	(pit 2)
			R2.24W,T6.32N	hor. sand layer

10	8.60	1.00	Clinton Co.	Gilford Sandy
	12.00	11.00	Section 30	Loam
	19.00	10.00	Bingham Twp.	(pit 3)
			R2.24W,T6.32N	hor. 28 inch.
11	50.00	60.00	Clinton Co.	Gilford Sandy
	21.00	41.00	Section 30	Loam
	54.00	66.00	Bingham Twp.	(pit 3)
			R2.24W,T6.32N	hor. 41 inch.
12	1.30	.51	Clinton Co.	Wasepi Sandy
	6.30	.76	Section 30	Loam
	6.90	2.10	Bingham Twp.	(pit 4)
			R2.24W,T6.32N	hor. 20 inch.
13	36.00	27.00	Clinton Co.	Wasepi Sandy
	16.00	9.30	Section 30	Loam
	11.00	4.60	Bingham Twp.	(pit 4)
			R2.24W,T6.32N	hor. 33 inch.
14	35.00	18.00	Clinton Co.	Wasepi Sandy
	57.00	55.00	Section 30	Loam
	78.00	76.00	Bingham Twp.	(pit 4)
			R2.24W,T6.32N	vert. 48 inch
15	34.00	31.00	Clinton Co.	Gilford Sandy
	55.00	27.00	Section 30	Loam
	59.00	50.00	Bingham Twp.	(pit 5)
			R2.24W,T6.32N	hor. 36 inch.
16	1.20	2.10	Clinton Co.	Gilford Sandy
	1.50	.89	Section 30	Loam
	1.90	.81	Bingham Twp.	(pit 5)
			R2.24W,T6.32N	hor. 46 inch.
17	13.00	10.00	Clinton Co.	Gilford Sandy
	15.00	18.00	Section 30	Loam
			Bingham Twp.	(pit 5)
			R2.24W,T6.32N	vert. 48 inch.
18	36.00	47.00	Ingham Co.	Metea Loamy
	40.00	42.00	Section 26	Sand
	47.00	52.00	Williamstown Twp.	
			R1E, T3.46N	
19	43.00	40.00	Ingham Co.	Riddles Hillsdale
	52.00	31.00	Section 12	Sandy Loam
	27.00	28.00	Williamstown Twp.	28 inch.
	40.00	28.00	R1.26E,T4.16N	
20	.50	.20	Ingham Co.	Riddles Hillsdale
	1.70	.20	Section 12	Sandy Loam
	1.60	.50	Williamstown Twp.	8 inch.
	3.60	.60	R1.26E,T4.16N	

21	1.40	.50	Ingham Co.	Metea Loamy
	2.10	.50	Section 7	Sand
	2.70	.80	Locke Twp.	site 1
	.80	.50	R1.33E,T4.18N	
	.70	.50		
22	.01	.05	Ingham Co.	Metea Loamy
	.01	.05	Section 7	Sand
			Locke Twp.	site 2
			R1.33E,T4.18N	
23	.03	.08	Ingham Co.	Colwood-Brookston
	.01	.05	Section 7	Loams
	.03	.08	Locke Twp.	site 3
			R1.33E,T4.18N	
24	.07	.20	Ingham Co.	Riddles Hillsdale
	1.00	.61	Section 7	Sandy Loam
	.06	.28	Onadaga Twp.	
	.05	.05	R2.25W,T1.17N	
25	16.00	3.80	Ingham Co.	Spinks Loamy
	5.30	2.00	Section 27	Sand
	2.00	.41	Wheatfield Twp.	2nd time
			R1.06E,T2.41N	
26	.83	.25	Ingham Co.	Boyer Spinks
	.68	.33	Section 21	Loamy Sand
	.20	.15	Onadaga Twp.	site 1
	.10	.08	R2.02W,T1N	
27	5.20	8.60	Ingham Co.	Boyer Spinks
	18.00	20.00	Section 21	Loamy Sand
	24.00	24.00	Onadaga Twp.	site 4
	29.00	41.00	R2.02W,T1N	
28	.08	.31	Ingham Co.	Oshtemo Spinks
	.05	.25	Section 21	Loamy Sand
			Onadaga Twp.	site 6
			R2.02W,T1N	
29	.42	.23	Ingham Co.	Oshtemo Spinks
	.49	.31	Section 16	Loamy Sand
	.38	.51	Onadaga Twp.	
	.28	.56	R2.09W,T1.09N	
30	87.00	64.00	Ingham Co.	Aubbeenaubbee
	65.00	41.00	Section 6	Capac Sandy Loam
	49.00	34.00	Alaiedon Twp.	9 feet
	73.00	55.00	R1.23W,T3.29N	

31	.10 .01 .13 .24	.25 .18 .20 .43	Ingham Co. Section 4 Williamstown Twp. R0.2E,T4.25N	Gilford Sandy Loam (pond spoils) 35 inch.
32	.56 .73 .20 .36 .30	.53 .31 .43 .36 .31	Ingham Co. Section 4 Williamstown Twp. R0.2E,T4.25N	Gilford Sandy Loam (pond spoils) 47 inch.
33	.25 2.16 .60 2.90	.91 2.16 1.30 2.00	Ingham Co. Section 23 Leroy Twp. R2.18E,T3N	Marlette Fine Sandy Loam
34	22.40 21.70	21.60 23.30	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand (below stones distal end)
35	4.40 8.80 2.00 1.90	4.90 7.50 2.40 2.60	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand (below gray sand)
36	16.30 12.50 7.30 5.10	16.60 10.70 5.60 6.70	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand (trench bottom proximal end)
37	.42 .49 .32	.74 .76 .41	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand (6 inch. below trench bottom)
38	61.00 50.00 36.00 48.00 24.00	46.00 50.00 32.00 47.00 26.00	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand (parent material)
39	36.00 36.00 60.00 59.00 81.00	36.00 40.00 56.00 51.00 56.00	Monroe Co. Section 7 Erie Twp. R8E, T8S	Oakville Fine Sand

NOTE: There are six section divisions between Town and Range divisions and each of these six divisions is subdivided into 10 partitions for more accuracy in locating a site. This yielded the two decimal places in most of the site locations as available.

APPENDIX C

C. Field Data

VHP Field	Constant-head	Site No.
.90	1.40	1
.10	.16	2
.47	.45	3
.33	.22	3
2.54	1.90	4
1.20	1.40	4
1.80	1.30	5
31.00	39.00	18
32.00	41.00	19
.99	1.90	20
2.70	1.50	21
.05	.02	24
.15	.02	22
1.10	.42	25
.72	7.80	26
4.50	65.00	26 (site 2)
10.16	19.00	27
.23	.07	28
.48	.45	27 (site 3)
.29	.39	29
34.00	69.00	30
2.40	.50	31
.41	.33	32
.63	1.50	33
3.00	4.20	35
13.00	12.00	6
3.80	7.80	7
18.00	23.00	9
2.00	13.00	10
46.00	42.00	11
51.00	57.00	14
2.50	21.00	13
3.80	4.80	12
31.00	49.00	15
2.50	1.50	16
15.00	14.00	17

NOTE: For location of site numbers see appendix B. Laboratory Data

APPENDIX D

D. Vertical versus Horizontal Data

Vertical	Horizontal
5.000000E-03	2.540000E-02
2.500000E-02	7.366000
1.000000E-01	1.400000
1.143000E-01	4.600000E-01
2.300000E-01	3.000000E-01
2.540000E-01	1.016000
2.540000E-01	5.080000E-01
4.064000E-01	4.826000E-01
4.700000E-01	4.572000E-01
4.826000E-01	7.112000E-01
5.200000E-01	1.270000E-01
6.270000E-01	3.378200
7.190000E-01	2.133600
8.750000E-01	2.200000
1.140000	2.133600
1.190000	2.184400
1.750000	2.489200
2.413000	6.858000E-01
2.730500	9.220000
4.953000	3.760000
7.620000	17.780000
9.323000	9.320000
10.033000	16.600000
30.480000	14.990000
31.620000	35.737800
34.050000	43.940000
78.280000	28.960000

APPENDIX E

E. Bulk Density versus K_s Values

Bulk Density	K_s
1.410000	3.048000E-01
1.410000	4.318000E-01
1.440000	39.573200
1.450000	18.290000
1.450000	1.778000E-01
1.450000	33.430000
1.460000	10.000000
1.460000	5.000000E-02
1.460000	30.988000
1.480000	21.870000
1.480000	8.100000E-01
1.500000	51.540000
1.500000	1.016000
1.500000	37.800000
1.500000	2.032000E-01
1.500000	7.600000E-01
1.510000	30.760000
1.510000	2.794000E-01
1.510000	8.900000E-01
1.510000	3.048000E-01
1.510000	13.740000
1.510000	4.318000E-01
1.520000	31.170000
1.520000	27.660000
1.520000	47.190000
1.520000	28.090000
1.520000	9.780000
1.530000	40.390000
1.530000	2.950000
1.540000	4.600000E-01
1.540000	6.100000
1.540000	9.700000E-01
1.540000	4.470000
1.540000	40.767000
1.540000	3.556000E-01
1.540000	16.560800
1.550000	6.604000
1.560000	16.030000
1.560000	18.310000
1.560000	5.638800
1.560000	42.040000
1.560000	2.540000E-01
1.560000	23.291800
1.560000	64.033400
1.560000	5.100000E-01
1.570000	5.580000
1.570000	26.920000

1.570000	9.910000
1.570000	2.110000
1.570000	56.235600
1.570000	50.240000
1.570000	55.626000
1.580000	45.923200
1.580000	51.054000
1.590000	5.000000E-02
1.590000	55.194200
1.590000	26.850000
1.590000	2.110000
1.590000	33.629600
1.590000	5.100000E-01
1.590000	8.600000E-01
1.590000	10.668000
1.590000	19.812000
1.600000	76.200000
1.600000	9.400000E-01
1.600000	1.300000E-01
1.600000	2.000000E-01
1.600000	1.200000
1.600000	6.705600
1.610000	10.900000
1.610000	24.053800
1.610000	40.950000
1.610000	20.345400
1.620000	60.400000
1.620000	5.080000E-02
1.620000	4.572000
1.620000	3.810000
1.630000	65.790000
1.630000	7.600000E-01
1.630000	4.851400
1.630000	39.801800
1.630000	3.300000E-01
1.630000	54.860000
1.630000	2.209800
1.630000	2.032000E-01
1.640000	46.482000
1.640000	2.616200
1.640000	2.800000E-01
1.640000	5.100000E-01
1.650000	49.707800
1.650000	4.040000
1.650000	5.100000E-01
1.650000	5.080000E-02
1.650000	8.610600
1.660000	21.640800
1.670000	4.064000E-01
1.670000	4.800000E-01
1.680000	6.400000E-01
1.690000	9.144000E-01
1.700000	6.096000E-01
1.700000	1.524000E-01

1.700000	7.620000E-02
1.700000	7.620000E-02
1.700000	3.800000E-01
1.710000	2.362200
1.710000	1.524000E-01
1.710000	1.500000E-01
1.720000	3.048000E-01
1.720000	9.300000
1.720000	5.100000E-01
1.720000	7.620000E-01
1.720000	1.016000E-01
1.720000	7.467600
1.720000	3.100000E-01
1.730000	2.032000
1.730000	8.382000E-01
1.730000	1.350000
1.750000	7.620000E-02
1.760000	3.302000E-01
1.760000	4.570000
1.760000	2.540000E-01
1.760000	7.620000E-02
1.770000	1.500000E-01
1.770000	5.080000E-01
1.770000	2.540000E-01
1.770000	1.440000
1.770000	1.270000
1.770000	9.000000E-01
1.780000	5.334000E-01
1.800000	7.600000E-01
1.810000	5.080000E-01
1.810000	2.286000E-01
1.830000	2.159000
1.840000	1.981200
1.860000	5.588000E-01
1.900000	1.200000E-01
1.920000	3.048000E-01
1.960000	6.300000E-01
1.970000	1.800000E-01
1.980000	3.100000E-01
2.240000	1.300000E-01

APPENDIX F

F. Plot of Transformed Data With Outliers

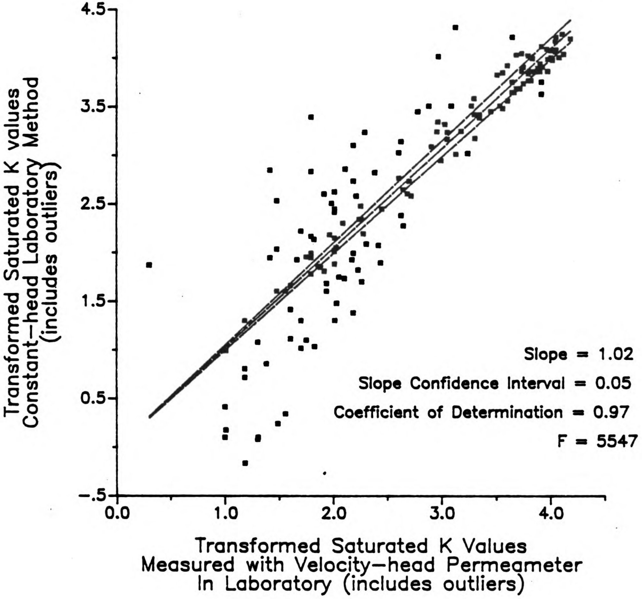

Figure 13

Linear regression of transformed saturated hydraulic conductivities obtained with both the velocity-head permeameter and the standard constant-head outflow procedure, in the laboratory on the same undisturbed cores. Before removal of outliers from the data set.