1. Introduction

Both in environmental site characterization and in the design of geotechnical engineering projects, one of the most important soil properties of interest to the soils engineers is permeability. To some degree, permeability will play a role in the migration of contaminant and on the design of almost any structure. For example, compacted clay liners are often used to minimize potentially contaminated seepage from the base of tailings impoundments. In designs that make use of earthen materials (soils and rock, etc.) the permeability of the materials will usually be of great importance.

To illustrate the importance of permeability in environmental site characterization and geotechnical design, consider the following applications where knowledge of permeability is required:

- The rate of flow to wells from an aquifer is dependent on permeability.
- The migration of contaminant seepage through a saturated or unsaturated soil profile is dependent on permeability.
- The design of earth dams is very much based upon the permeability of the soils used.
- Permeability influences the rate of settlement of a saturated soil under load.
- The performance of landfill or tailings impoundment liners is based upon their permeability.
- The stability of slopes and retaining structures can be greatly affected by the permeability of the soils involved.
- Filters to prevent piping and erosion are designed based upon their permeability.

Soils are permeable (i.e. water may flow through them) because they consist not only of solid particles, but a network of interconnected pores. The degree to which soils are permeable depends on a number of factors, such as soil type, grain size distribution, water content, degree of compaction and soil history. This degree of permeability is characterized by the coefficient of permeability (or hydraulic conductivity).

The coefficient of permeability, k, is a product of Darcy's Law (1856), which establishes the following empirical relationship for flow through saturated porous media:

$Q = K\,i\,A$, where: (1)

Q = flow rate (L^3/T)

i = hydraulic gradient (unitless)

A = cross-sectional area of flow (L^3)

K = coefficient of permeability (L/T)

It should be noted that while geotechnical engineers mostly use the term coefficient of permeability, this same parameter is usually referred to as hydraulic conductivity by hydrogeologists and environmental scientists.

Flow in the unsaturated zone differs significantly from flow in the saturated zone. In the unsaturated zone, the pressure head is negative, meaning that it is less than atmospheric pressure. The water content is less than the porosity because some of the void space is filled with soil gas (air). In saturated porous media, water can move through the entire cross-sectional area of the pore space. However, as water is replaced by air, it can only move through the reduced cross-sectional area occupied by the remaining water. This has the effect of lowering the hydraulic conductivity.

The result is that water content of the porous medium is variable and a function of the pressure head. The more negative the pressure head (or inversely the higher the suction) the lower the moisture content. The exact relationship depends on the soil type. Similarly, the hydraulic conductivity decreases with decreasing water content and increasing suction. The result is that Darcy's law (presented above) becomes non linear. Special testing methods therefore had to be developed and adapted to measure the hydraulic conductivity of a soil in the unsaturated zone in order to predict the movement of water.

2. Overview of permeability testing methods

Saturated hydraulic conductivity (K_{sat}) is a critically important soil property. As a result, many methods have been developed over time for field and laboratory measurement of K_{sat} (e.g. Klute, 1986). Unfortunately, these methods often yield substantially dissimilar results, as K_{sat} is extremely sensitive to sample size, flow geometry, and soil physical and hydraulic characteristics. In addition, most K_{sat} measurement methods are neither appropriate for all applications nor accurate for all soil types and conditions (Bouma, 1983). The literature (e.g. Dun and Philips (1991) shows that regardless of the land practices a small portion of the soil volumes transports a large portion of the water flow, indicating that the spatial hydraulic characteristics of soils are highly variable. K_{sat} measurements should therefore be evaluated carefully to ensure that the K_{sat} values obtained are both accurate and appropriate for the intended use.

Because of its dependence on water content, a complete description of hydraulic conductivity in unsaturated materials is very difficult. However, there are many applications in which only the saturated hydraulic conductivity is required. These include estimates of infiltration capacity for input in hydrogeologic models, evaluation of the suitability of soils for constructing liners, evaluation of the potential leakage from tailings impoundments, and several others.

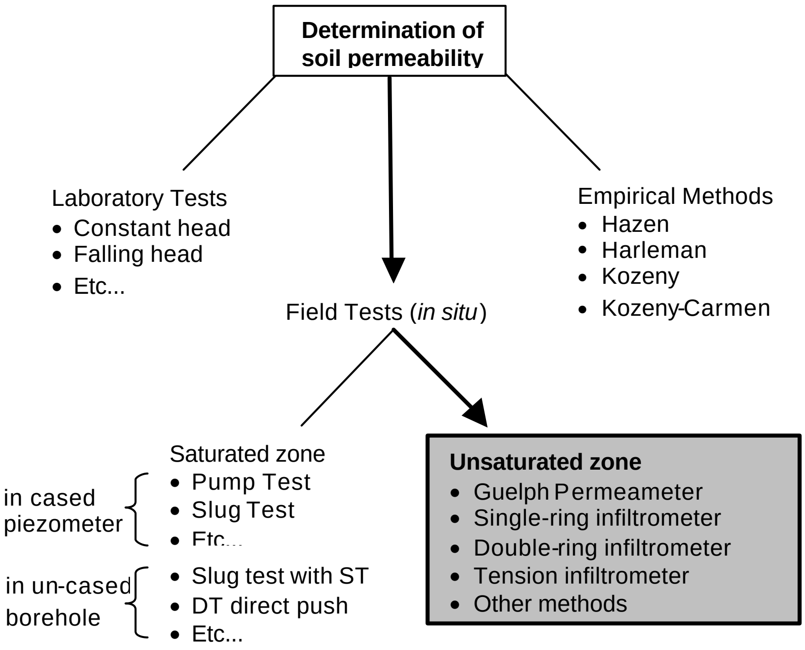
Several methods are available to the engineer/scientist for determining soil permeability (Figure 1). These methods vary in complexity and applicability and exist both for saturated and unsaturated conditions. In general, the hydraulic conductivity of a soil can be determined by the following approaches:

1. Laboratory tests;
2. Empirical methods (based on grain diameter and grain size distribution);
3. Field *(in situ)* tests.

Field tests are by far the most reliable for they permit the testing of larger volumes of soils. In the field, the permeability of a soil can either be measured in existing monitoring wells (for the saturated zone) or from the surface, sub-surface using infiltration methods (for the vadose zone). Pump and/or slug tests (slug in or slug out) are typically used for determining the permeability of the soil aquifer material by recording the drawdown (or change in well head) over time. Special methods such as the dual tube (DT) direct push in methods can also be used to determine the permeability of a soil without the need of a piezometer.

The discussion that follows briefly presents laboratory and empirical methods, while the focus is kept on *in situ* field methods for determination of saturated permeability (hydraulic conductivity) in the unsaturated (vadose) zone of a soil profile. Figure 1 summarizes the testing methods available for determining the permeability of a soil.

Figure 1. Summary of testing methods for determining soil permeability.



2.1 Laboratory methods

In the laboratory, permeameters (Figure 2) are generally used to perform two common tests to determine this soil property: the falling head permeability test and the constant head permeability test. Which test is used depends upon the type of soil tested. For soils of high permeability (e.g. sands and gravel) a constant head test is normally used. For soils of intermediate to low permeability, a falling head test is normally used.

In the constant head test, a constant total head difference is applied to the soil specimen, and the resulting quantity of seepage can then be measured. In this test, a valve (Figure 2a) at the base of the sample is opened and the water starts to flow. After a sufficient volume of water is collected over the time of the test, the volumetric flow rate Q is calculated. Hydraulic conductivity is then determined with Darcy's Law of the form:

$$K = \frac{QL}{Ah} \tag{2}$$

where L is the length of the sample, A is the cross-sectional area of the sample, and h is the constant head shown on Figure 2. This method works well for coarse-grained soils, but with clays and silts, the quantity of seepage is much too small to be accurately measured.

The falling head test is different in that it does not fix the total head difference across the soil specimen. Instead, a standpipe is connected to the inflow, and the water level in this standpipe is then allowed to drop as water flows through the specimen. In this test, the head is measured in the standpipe of Figure 2b, along with the time of measurement. For a sample of length L and a cross-sectional area A, the conductivity is determined by

$$K = 2.3\frac{aL}{A(t_1 - t_0)}\log_{10}\frac{h_0}{h_1} \tag{3}$$

where a is the cross-sectional area of the standpipe and (t_1-t_0) is the elapsed time required for the head to fall from h_0 to h_1. This method will not work well for coarse-grained soils, because they are so permeable that the head drops too rapidly to be accurately measured.

For compressive soils (e.g. silts and clays) the hydraulic conductivity is significantly influenced by the degree of compaction (in-situ density) of the sample. It is therefore important to record the in-situ density of the soil to be tested and to recreate the similar conditions in the permeameter. For this reason, fine-grained soils are best tested in a triaxial testing apparatus, where the confining stress can be controlled accurately.

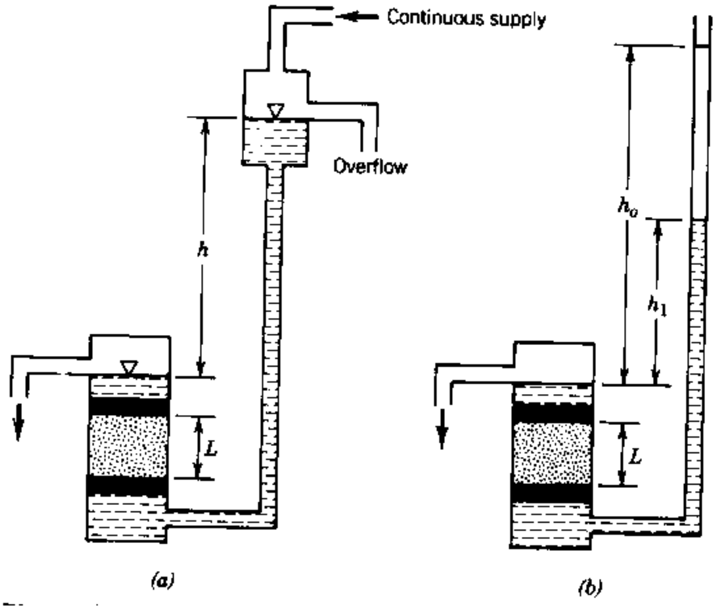


Figure 2: Laboratory set up for constant head test (a) and falling head test (b) (adapted from Dominico & Schwartz, 1990).

2.2 Empirical methods

As hydraulic conductivity can be readily measured in the laboratory, there have been numerous attempts to relate the measured values to various properties of a porous medium. One commonly accepted relationship has been proposed by Hazen (1991):

$$K = Cd_{10}^{2} \tag{4}$$

where K is the hydraulic conductivity in cm/s, d_{10} is the effective grain size in cm , and C varies from 100 to 150 $(\text{cm.sec})^{-1}$ for loose sands. The effective grain size is defined as the value where 10% of the particles are finer and 90% are coarser.

Another formula of the form given by Equation 4 has been proposed by Harleman and others (1963), stated as

$$k = (6.54 \times 10^{-4})\ d_{10}^{2} \tag{5}$$

where k is the permeability in cm^2 and d_{10} is again the effective grain size in cm. Several other empirical methods such as Masch and Denny (1966) and Krumbein and Monk (1943) were proposed to correlate permeability with grain size but are not discussed any further in this document.

In addition to these empirical approaches, there are other, more hydraulically-based attempts to relate permeability to porous medium properties. Kozeny (1927) considered the porous medium to be a bundle of capillary tubes and demonstrated that permeability must have the form

$$k = Cn^3/S^{*2} \qquad (6)$$

where C is a dimensinless constant that takes on values of 0.5 for circular capillaries, 0.562 for square capillaries, and 0.597 for equilateral triangles; k is permeabilty in L^2; n is porosity; and S^* is the specific surface, defined as the interstitial surface area of the pores per unit bulk volume of porous material.

One of the better known hydraulically-based models is the Kozeny-Carmen equation, stated as

$$K = \left(\frac{r_w g}{m}\right)\frac{n^3}{(1-n)^2}\left(\frac{d_m^2}{180}\right) \qquad (7)$$

where K is hydraulic conductivity, ρw is fluid density, μ is fluid viscosity, g is the gravitational constant, and d_m is a representative grain size (Bear, 1972).

Other hydraulic models exist, such as Collins (1961) that considers tortuosity and pore size distribution, but are not discussed any further in this document.

2.3 Field Methods for the Saturated Zone

In virtually all groundwater investigations, the focus of the study is to provide an estimate of the transmissive nature of the subsurface material. A large number of experimental techniques have been developed over the years to provide estimates of the hydraulic conductivity of aquifer material. These techniques range from laboratory-based permeameter or grain-size analyses to large-scale multiwell pumping tests. In the last two decades, a field technique for the estimation of hydraulic conductivity insitu know as the slug test has become increasingly popular (mainly for its rapidity, simplicity and low cost), especially among scientists and engineers working at sites of suspected groundwater contamination.

The following section briefly presents some of the most widely used field-testing methods for determining hydraulic conductivity in the saturated zone. The discussed methods are:

- Slug tests
- Pump tests
- Tracer dilution tests

2.3.1 Slug Test

The slug test essentially consists of measuring the recovery of head in a well after a near-instantaneous change in head at that well (a nearby observation well can also be used in certain situations). In the standard configuration, a slug test begins with a sudden change in water level in a well. This can be done, for example, by rapidly introducing a solid object (“slug”) or equivalent volume of water into the well (or removing the same), causing an abrupt increase (or decrease) in water level. Following this sudden change, the water level in the well returns to static conditions as waster moves out of the well or into it (when change was a decrease in water level) in response to the gradient imposed by the sudden change in head. These head changes through time, which are termed the response data, can be used to estimate the hydraulic conductivity of the aquifer formation through comparison with theoretical models of test responses. The design, performance, and analysis of slug tests is described in details by Butler (1998).

2.3.2 Pump Test

Pumping tests may be conducted to determine (i) the performance characteristics of a well and (ii) the hydraulic parameters of the aquifer. This later purpose is to provide data from which the principal factors of aquifer performance (transmissivity, hydraulic conductivity and storage coefficient) can be calculated. A pumping test consists of pumping a well at a certain rate and recording the drawdown in the pumping well and in nearby observation wells at specific times. There are two primary types of aquifer tests: constant-rate tests and step-drawdown tests. In the constant-rate test, the well is pumped for a significant length of time at one rate, whereas in a step-drawdown test the well is pumped at successively greater discharges for relatively short periods. Data from both types of aquifer pumping tests can be analyzed to determine important hydraulic characteristics of an aquifer and/or of a well.

Measurements required for pump tests include the static water level just before the test is started, time since the pump started, pumping rate, pumping levels or dynamic water levels at various intervals during the pumping period, time of any change in discharge rate, and time the pump stopped. It is important to also monitor water level recovery following cessation of the pump for a period approximately as long as for the pumping phase. The recovery measurements are extremely valuable in verifying the aquifer coefficients calculated during the pumping phase of the test.

A number of analytical techniques have been developed over the years to interpret pump test data to extract the desired hydraulic parameters. These methods are presented in details in various textbooks such as Freeze and Cherry 1979), Driscoll (1986), Domenico and Schwartz (1990) and other manuals.

2.3.3 Tracer Dilution/pumpback Test

In the tracer dilution test, changes in concentration with time of an initially uniform column of tracer (e.g. a salt, bromide or other solutions) in a borehole are used to determine seepage velocities and hence indicate the ambient groundwater flow velocity about the borehole. To determine the direction of the flow and hence the groundwater velocity the method needs to be extended to measuring tracer breakthrough curves from observation boreholes of differing orientations. This can be expensive and time consuming to set up the required observation boreholes and often less than ideal sets of orientations have to be used.

If, however, the tracer used with a single borehole dilution is conductive, such as for a saline tracer, the movement of the material into the surrounding media along the ambient groundwater gradient will produce anisotropy in the conductivity about the borehole.

In the tracer-dilution test, a solution of known concentration is circulated/mixed within the well-screen section. The decline of tracer concentration (i.e., "dilution") with time within the well screen can be monitored directly using a vertical array of bromide specific-ion electrode probes located at known depth intervals. Based on the dilution characteristics observed, the vertical distribution (i.e., heterogeneity) of hydraulic properties and/or flow velocity can be estimated for the formation within the well screen section. The presence of vertical flow within the well screen can also be identified from the probe/depth dilution response pattern. A description of the performance and analysis of tracer-dilution test characterization investigations is provided in Halevy et al. (1966), Hall et al. (1991), and Hall (1993).

Alternaively, the tracer pumpback is a constant-rate pumping test that is initiated after the average tracer concentration has decreased (i.e., diluted) to a sufficient level within the well screen (usually a 1 to 2 order of magnitude reduction from the original tracer concentration). The objective of the pumpback test is to "capture" the tracer that has moved from the well to the surrounding aquifer. Tracer recovery is monitored by measuring the tracer concentration in water pumped from the well. The time required to recover the centroid of tracer mass/concentration provides information of the aquifer effective porosity, ne. Effective porosity is a primary hydrologic parameter controlling contaminant transport. The pumpback method will not be discussed in any further details, given the considerable level of complexity associated with this technique.

2.4 Field methods for the Vadose Zone

For field methods, a distinction must be made between “saturated” (K_s) and “field-saturated” (K_{fs}) hydraulic conductivity. True saturated conditions seldom occur in the vadose zone except where impermeable layers result in the presence of perched water tables. During infiltration events or in the event of a leak from a lined pond, a "field-saturated" condition develops. True saturation does not occur due to entrapped air. The entrapped air prevents water from moving in air-filled pores that, in turn, may reduce the hydraulic conductivity measured in the field by as much as a factor of two compared to conditions when trapped air is not present (i.e. full saturation). Field tests methods generally simulate the "field-saturated" condition. The hydraulic conductivity measured in the unsaturated (vadose) zone is thus referred to as the "field-saturated" hydraulic conductivity (K_{fs}) (Reynolds et al, 1983).

Several test methods are available for determining the field saturated hydraulic conductivity (K_{fs}) of unsaturated materials above the water table. Most of the methods involve measurement of the infiltration rate of water into the soils from an infiltrometer or permeameter device. Infiltrometers typically measure conductivity at the soil surface, whereas permeameters may be used to determine conductivity at different depths within the soil profile. A representative list of the most commonly used equipment includes the following:

- infiltrometers (ponded, tension, single and double ring infiltrometers),
- double-tube method;
- air-entry permeameter,
- borehole permeameter methods (constant and multiple head methods).

Field tests used for determining the value of K_{fs} can either be constant head tests and falling head tests. Most field methods that do not use piezometers correspond to constant head tests. These various methods are briefly presented in the following paragraphs. The interested reader is encouraged to consult the list of references provided below, which summarizes the principle, operation mode and interpretation of each of these field methods. Table 1 compares different field test methods commonly used for measuring hydraulic conductivity in the vadose zone of soils.

The remainder of this document describes the various field methods for determining field-saturated permeability in the unsaturated (vadose) zone.

Table 1: Review and comparison of test methods for measuring hydraulic conductivity in the vadose zone (adapted from ASTM D5126).

Characteristics	*Single Ring Infiltrometer*	*Double Ring Infiltrometer*	*Double-Tube Test Method*	*Air-Entry Permeameter*	*Borerehole Permeameter*	
					Free Surface	*2 Head Solution*
Relative Accuracy	Low	Fair	Fair	Good	Poor	Variable
Relative Cost	Low	Low-Moderate	Moderate	Moderate	Low-Moderate	
Time required (at Kfs=10-5cm/s)	<4h	<4h	4h to 1d	<4h	<4h	
Depth of Testing possible	Surface	Surface	0 to 1ft	0 to 1ft	Any	
Range of Kfs (cm/s) for which the test is suited	10^{-2} to 10^{-6}	10^{-2} to 10^{-6} 10^{-6} to10^{-8} (with flexible bag for inner reservoir)	$<10^{-6}$	$<10^{-8}$	$<10^{-6}$	
Advantages	Simple apparatus, can estimate Kfs from infiltration data, can increase diameter to reduce scale efects and edge effect	Similar to single ring		Measures vertical Kfs only, accounts for capillary effects	Simple numerical solution, good approximation for sands	Simple solution, accounts for capillarity
Disadvantages	Lateral flow affects accuracy, measures infiltration not Kfs, surface crust reduces infiltration, measures on surface of soil only	Similar to single ring	Cumbersome apparatus, time-consuming numerical solution	Sometimes difficult to drive tube, difficult to identify wetting front in wet soil	Does not account for capillary effects, high error for medium to fine unstructured soils	Occasionally gives negative values

2.4 References for Permeability Testing

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3. Borehole Permeameter Tests

Borehole permeameter test methods encompass a wide range of test designs, which differ to varying degrees in theory, procedure, apparatus and methods of solution. The common feature among the different types of borehole permeameter tests is that the rate of water infiltration into a borehole is used to determine field-saturated hydraulic conductivity in the unsaturated zone.

One of the most popular borehole infiltration tests is the constant-head borehole infiltration test. The method involves the measurement of the steady-state infiltration rate required to maintain a steady column of water in an un-cased, cylindrical hole that terminates above the water table (Reynolds, 1993). Other factors such as borehole geometry, borehole radius (r), and depth of ponding (h), and along with certain capillary parameters are typically used in the solution. Hence, by accounting for capillary effects, borehole test methods attempt to measure field-saturated hydraulic conductivity rather than infiltration rates. Another variation of this test consists of conducting multiple constant head borehole infiltration tests within the same borehole. Different water levels are established within the borehole for each individual test. Results from one or more tests at different ponded heights are solved simultaneously to find K_{fs} and capillarity.

Borehole permeameter tests are the only currently available test, which can measure field-saturated hydraulic conductivity at depth within the unsaturated zone. Borehole tests may be conducted at great depth within the unsaturated zone, and are frequently used to measure the variability of conductivity with depth by conducting tests at selected horizons within an advancing borehole.

3.1 Guelph Permeameter

3.1.1 Overview

The Guelph Permeameter (GP) is perhaps the most well known/used borehole permeameter technique and is described in detail below. It is sometimes referred to as the Guelph Pressure Infiltrometer (PI) to describe the same instrument.

The GP is an easy to use instrument for quickly and accurately measuring *in situ* hydraulic conductivity. Accurate evaluation of soil hydraulic conductivity, soil sorptivity, and matrix flux potential can be made in all types of soils. The equipment can be transported, assembled and operated easily by one person. Measurements can normally be made in ½ to 2 hours, depending on soil type, and require only about 2.5L of water. The Guelph permeameter is manufactured by Soil Moisture Equipment Corp. (www.soilmoisture.com), based in California, USA. Reynolds et al. (1985) provide a complete description of the GP apparatus, which is essentially an in-hole Mariotte bottle constructed of concentric, transparent plastic tubes.

The GP comes as a complete kit and extension tubes can be used for determining hydraulic conductivity to a maximum operating depth of approximately 3m. Ring attachments allow ring infiltrometer measurements from 10-cm and 20-cm diameter rings. A tension adaptor can also convert the GP to a tension infiltrometer (see section 4.1 below), which allows measurements to be made under tensional and very low head conditions.

3.1.2 Principles

Guelph permeameter measurements are carried out in the vadose zone above the water table, where the soil is unsaturated (Figure 3, Photo 1). Steady flow produces a small inner saturated zone adjacent to the well, encased within a larger outer wetted, but unsaturated volume. As a consequence, combined saturated-unsaturated flow occurs as shown in Figure 4.

The GP method measures the steady-state rate Q (m^3/s) necessary to maintain a constant depth of water H (m) in an uncased cylindrical well of radius a (m), above the water table. Then the field saturated hydraulic conductivity K_{fs} (m/s), and the matric flux ϕ_m (m^2/s) are calculated from Q, H, and using the following approximate analytical solution (Reynolds et al., 1985).

$$Q = \left(\frac{2pH^{2}}{C} + pa^{2}\right)K_{fs} + \frac{2pH}{C} f_{m} \qquad (8)$$

$$= Ak_{fs} + B\phi_{m}$$

where C is a dimensionless shape factor primarily dependent on the H/a ratio and soil type (see Soil Moisture operating manual, Fig. 45 on p. 25).

Photo 1: Guelph Permeameter testing in mine tailings, note dessication cracks at the surface (a courtesy of Robertson GeoConsultants Inc., 2003)

The field saturated hydraulic conductivity (K_{fs}) is a measure of the capacity of porous medium to conduct a wetting liquid, and the matric flux potential (ϕ_m) is a measure of the capacity of porous medium to absorb a wetting liquid. Hence the first term of equation (2) represents the pushing action, and the second term represents the capillary pulling action.

Although the GP is normally used under constant head conditions, the test tube can also be used for falling head testing in special applications such as investigating the change in permeability associated with tailings settlement (Photo 2).

The GP can be used anywhere a hole can be augered in the soil. Soils typically possess a three dimensional heterogeneity, while the GP method essentially provides a “point” measurement. Therefore, the number of measurements to adequately represent field variability will depend on factors such as soil type, type of application, project objectives, etc...A description of the soil profile (by sampling or from soil survey reports) will greatly complement the value and understanding of data obtained with the Guelph Permeameter.

A complete description of the operating mode of the GP is provided with the Operating Instructions manual from Soil Moisture Equipment Corp., which can be found on the manufacturer website (www.soilmoisture.com). For the sake of brevity, the standard procedures are only summarized below:

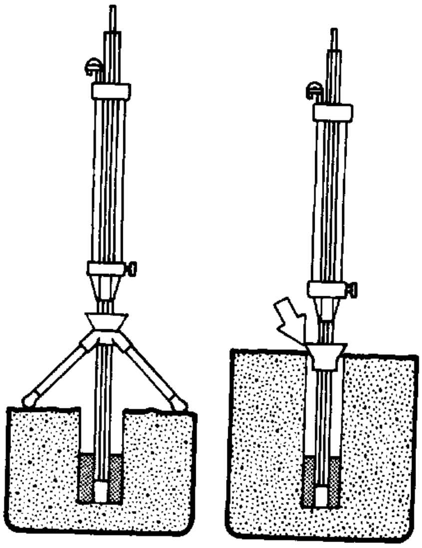

Figure 3: Guelph permeability testing in the unsaturated zone (adapted from Soil Moisture Equipment Corp., 1986).

Photo 2: Set up for falling head testing in using the GP test tube in freshly deposited mine tailings (courtesy of Robertson GeoConsultants Inc., 2003).

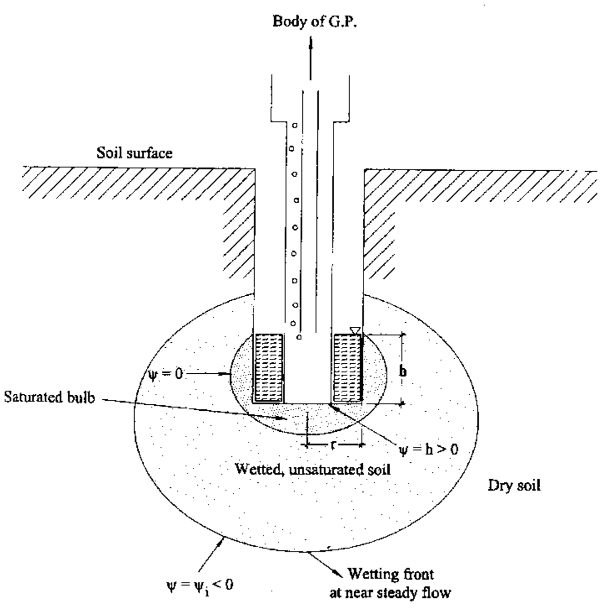

Figure 4: GP saturated bulb and wettting front surrounding the auger hole (ψ is the pressure head, h is the height of ponded water, ψi is the initial pressure head in the soil, from Giakoumakis and Tsakiris, 1999).

3.1.3 Summary of field procedures

i. Excavate a cylindrical well to the desired depth in the material to be tested;
ii. Fill the permeameter with liquid and inserting it in the well;
iii. Start the permeameter by raising the air-inlet tube out of the outlet port;
iv. Set the desired H level by adjusting the height of the air-inlet tube;
v. Monitoring the rate of fall, r, of the liquid surface in the reservoir until a steady rate, r, is attained;
vi. Calculate Q using r and the cross-sectional area of the reservoir;
vii. Calculate K_{fs}, S and α using the solution equations (see Appendix A).

A convenient field data sheet for GP testing is provided in Appendix A of this document, which permits calculations to be performed in the field (adapted from Reynolds et al., 1985).

Baragello (1997), Bagarello and Provenzano (1995), Reynolds and Elrick (1985) and Meiers (2002) provide useful recommendations with respect to the augering of a borehole and comment on the importance of avoiding smearing (which should be removed by brushing and puncturing the side walls of a hole) in fine-grained materials and cleaning the base of the hole free of debris. Negligence at following these recommendations could result in a substantial underestimation of K_{fs}.

Subsidence of the GP tip into the base of the hole during a measurement can produce an underestimate of K_{fs} because the actual ponded water in the well becomes lower then assumed and the outflow from the GP is reduced by a smaller effective infiltration area (Bagarello, 1997). Meiers (2002) recommends additional care may be needed to support the weight of the GP when filled with water so that the water outlet tip does not sink into the base of the well during measurements. In soft and/or unstable materials (sands), it is advisable to use a screen insert to minimize collapse of the well during measurements, since this may block flow from the tip

(Meiers, 2002). Alternatively, a platform (e.g. made of wood) can be used for supporting the weight of the GP water reservoir and prevent subsidence in soft materials (Photo 3).

Photo 3: Set up for GP Testing in soft tailings using a wooden support platform (courtesy of Robertson GeoConsultants Inc., 2003)

3.1.4 Analysis of field data

The values of K_{fs} that can be measured with the GP range from 10^{-4} to 10^{-8} m/sec. Beyond these limits there is a reduction in accuracy and precision. In soils with $K_{fs}<10^{-8}$ m/s the rate of infiltration is too low to me monitored accurately.

Currently, two methods are widely accepted for solving equation (2) and hence determining the field saturated hydraulic conductivity; one consist of a single (or one ponded) height analysis (Elrick et al., 1989) while the other consists of using a dual (or two ponded) height analysis (Simulataneous Equation, or Richards analysis) (Reynolds et al., 1985). The Richards analysis accounts for all three forces that contribute to the 3D flow of water into soils. The gravitational pull of liquid out through the bottom of the well and the capillary pull of water out of the well into the surrounding soils). If used in combination in a complementary fashion the results will be more representative of the field conditions (Meiers, 2002).

Equation 9 below summarizes the calculation involved in determining K_{fs} when using the GP unit from Soil Moisture Equipment Corp. with the dual height analysis method.

(9)

For the 1st Set of readings R1' (cm/sec) = (__________)/60
R1

For the 2nd Set of readings R2' (cm/sec) = (__________)/60
R2

Kfs (cm/sec) = [(0.0041)(___________)(__________)] - [(0.0054)(___________)(__________)]
field saturated hydraulic conductivity — reservoir constant — R2' - steady state rate of flow — reservoir constant — R1' - steady sta rate of flow

Where R1' and R2' are the steady-state drop in water level during the first and second test, respectively.

Note that the value of the K_{fs} is dependent not only on the calculation procedure but it is also strongly influenced by the accurate detection of the steady-state flow rate (Q). Bagarello et al. (1999) provide a discussion and comparison of the various procedures to determine when steady-flow rate is achieved. The Three Quasi-Equal Readings and the Three Equal Readings (TR) are the most widely used criteria for evaluating steadiness in the flow rate.

The heterogeneous nature of soils can potentially lead to inaccuracies in flow measurements. Bagarello and Provenzano (1996) provide recommendations as to how the data should be treated to minimize the effects of heterogeneities on the calculation of K_{fs}. Reynolds et al. (2002) conducted field experiment comparing the tension infiltrometer (TI), the Guelph pressure infiltrometer (PI) and the intact soil core (SC) methods for measuring K_{sat} for various soil types and land management practices and concluded the PI method gave the most consistent and most reliable results of the three methods studied.

3.1.5 References for Guelph Permeameter Theory and Testing

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3.2 Air-Entry Permeameter

3.2.1 Overview

The air-entry permeameter (AEP) is similar to a single-ring infiltrometer in design and operation in that the volumetric flux of water into the soil with a single permeameter ring is used to calculate the vertical field-saturated hydraulic conductivity (K_{fs}) of geologic materials in the unsaturated zone. The primary differences between the two test methods are that the AEP typically penetrates deeper into the soil profile and measures the air-entry pressure of the soil. Air-entry pressure is used as an approximation of the wetting front pressure head for determination of the hydraulic gradient, and consequently field-saturated hydraulic conductivity.

Bouwer (1966) first proposed the AEP, and more recently Stephens et al. (1988) and Havlena and Stephens (1991) have shown that the AEP produces good results in low permeability clays and engineered clay liners. A short discussion of the applications of AEP's is included in ASTM standard D 5126. Useful information regarding the AEP is summarized in Table 2 below.

3.2.2 Principles

The AEP consists of a single permeameter ring, typically 30 cm in diameter and 25 cm-deep, sealed at the top by an air-tight cover, that is driven into the soil approximately 15 to 25 cm. Water is introduced into the permeameter through a standpipe, to the top of which is attached a graduated water supply reservoir and a pressure gauge (Figure 5). Daniel B. Stephens & Associates, Inc. has recently developed an automated AEP, which includes a pressure transducer and datalogger that automatically controls the water valve and records the water level data.

Water is allowed to infiltrate into the soil within the permeameter ring, and the flow rate is measured by observing the decline of the water level within the reservoir. After a predetermined amount of water has infiltrated (based upon the estimated available storage of the soil interval contained within the ring), and the flow rate is relatively stable, infiltration is terminated and the wetted profile is allowed to drain. The air-entry value is the minimum pressure measured over the standing water inside of the permeameter ring attained during drainage. Once the mimimum pressure is achieved, the permeameter is removed, and the depth to the wetting front is determined.

3.2.3 Summary of field procedures

Measurements of K_{fs} using the AEP in low-permeability soils can be performed in 1-8 hours. The testing is conducted in two stages. The first stage is the infiltration stage, in which water is allowed to enter the ring and infiltrate into the soil. The rate of decline in the water level within the water supply reservoir is measured during this stage.

The second stage is the drainage stage, during which the water supply is shut off and the water within the soil is allowed to redistribute. As the water redistributes, the tension within the water inside the ring increases until the point where the air-entry pressure (or bubbling pressure) is reached and bubbles migrate upward through the soil into the ring. The pressure during the drainage stage must be recorded at regular intervals. The minimum pressure (tension) achieved during this portion of the test is used to calculate the air-entry pressure of the soil, which is used in the determination of saturated hydraulic conductivity.

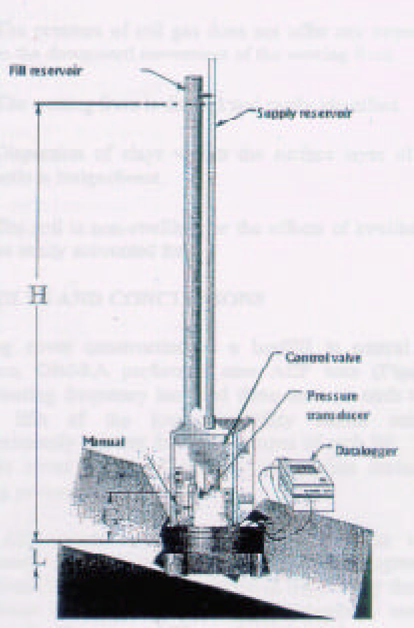

Figure 5: Automated air-entry permeameter (after Pegram et al., Daniel B. Stephens & Associates)

3.2.4 Analysis of field data

As soon as minimum pressure is reached, air begins to bubble up through the wetting front. Field-saturated K_{fs} can be calculated from the critical "air-entry" value or minimum pressure. Field-saturated K_{fs} is approximately equal to ½ of K_s in most soils or ¼ of Ks in fine-texture (clayey) soils (ASTM 5126).

Determination of vertical field-saturated hydraulic conductivity (K_{fs}) is based on measurement of the infiltration rate during Stage 1 and the air-entry pressure from Stage 2, along with the various values for the permeameter geometry , using the following Darcy-type equation, first proposed by Bouwer (1966).

$$K_{fs} = \frac{2\frac{dH}{dT}L(\frac{R_{ws}}{R_{sr}})^2}{H_f + L - (0.5P_a)} \qquad (10)$$

where:

K_{fs} = field-saturated hydraulic conductivity;
DH/dT = rate of fall in head just before water supply was shut off (cm/s);
R_{ws}/R_{sr} = radius of water supply reservoir over radius of AEP soil ring;
H_f = final head reading (cm);
P_a = Pmin + G + L (cm);
P_{min} = gauge pressure at air-entry (negative value) (cm);
G = height of gauge above the soil surface.

This equation for vertical saturated hydraulic conductivity using an AEP assumes that the air-entry pressure of the material within the permeameter ring can be used to approximate the critical pressure along the wetting front, thus allowing the AEP to measure the actual hydraulic gradient applied to the soil. Such direct measurement offers a distinct advantage over other ring type permeameter tests, for which the wetting-front pressure head and resulting hydraulic gradient must be estimated. This solution also assumes that the critical pressure during drainage is approximately twice that for imbibition, and therefore, ½ the value for air-entry is used in the solution equation.

Several assumptions are included within the use of the AEP, which are the same for all ring-type permeameters and infiltrometers presented in the section 4.2.2 below.

Table 2: Technical summary of air-entry permeameter, AEP (from Daniel B. Stephens & Associates)

Parameters Measured:	• Vertical, saturated hydraulic conductivity • Air-entry (bubbling) pressure
Range of use:	K_{fs} of 10^{-9} to 10^{-4} cm/s
Time to complete Test:	20 min for 10^{-4} cm/s materials to 1 day or more for 10^{-9} cm/s materials (longer term tests require uniform temperature and shielding)
Depth of test:	Up to 30 cm
Suitable soils:	Dry to moist sand, silt and clay
Advantages:	• Equipment is relatively inexpensive, portable, easily operated, and requires minimal use of water. • Rapid determination of vertically oriented saturated hydraulic conductivity under a variety of conditions. • Instrument can be automated with pressure transducers and data logger to increase measurement accuracy.
Disadvantages:	• Not suitable for soils with an abundance of rootlets, worm burrows or macropores. • Difficult to emplace permeameter ring in consolidated and stony materials.

3.2.5 References for Air-Entry Permeameter

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4. Infiltrometer test methods

Measurement of field saturated hydraulic conductivities is often done by borehole permeameters, which measures K_{fs} at depth (e.g. Elrick and Reynolds, 1992). In many cases, however, measurement of the soil surface K_{fs} is essential, especially in infiltration-related applications (e.g. irrigation management or tailings discharge). Ring infiltrometers are often used for measuring the water intake rate at the soil surface. The following sections present and discuss some of the infiltration test methods that are commonly used for field permeability testing in the vadose zone.

Infiltrometer test methods measure the rate of infiltration at the soil surface, that is influence both both by saturated hydraulic conductivity as well as capillarity effects of soil. Capillary effect refers to the ability of dry soil to pull or wick water away from a zone of saturation faster than would occur if soil were uniformly saturated. The magnitude of the capillary effect is determined by initial moisture content at the time of testing, the pore size, soil physical characteristics (texture, structure), and a number of other factors. Capillary effects are minimized by waiting until steady-state infiltration is reached.

4.1 Tension Infiltrometer

4.1.1 Overview

Tension (disk) infiltrometers (TI) have been widely used for *in situ* determination of the field-saturated hydraulic conductivity (K_{fs}) of soils under near-saturated conditions in the vadose zone (Meiers, 2002). Near-saturated conditions refer to measurements made over the negative pressure range, -20cm to 0.0cm, where water contents are nearly as high as those for saturation (Reynolds and Elrick, 1991). Tension infiltrometers determine the steady-state infiltration rate into the soil through a porous plate on which a constant negative water pressure (tension) is applied.

As for the GP, the tension infiltrometer is also capable of giving estimates of several useful parameters for characterizing soil structure. Soil Moisture Equipment Corp. manufactures the Guelph Tension Infiltrometer Adapter (Model 2825KI), which is designed to attach directly to the GP reservoir (Figure 6).

Tension infiltrometers are especially useful for quantifying the effects of macropores and preferential flow paths on infiltration in the field. A new method has recently been established for determining the K_{fs} from the TI method (Reynolds and Elrick, 1991, Ankeny et al. 1991). In this method, sequences of steady infiltration measurements made at several tensions on a single infiltration surface are used for calculating the K_{fs}.

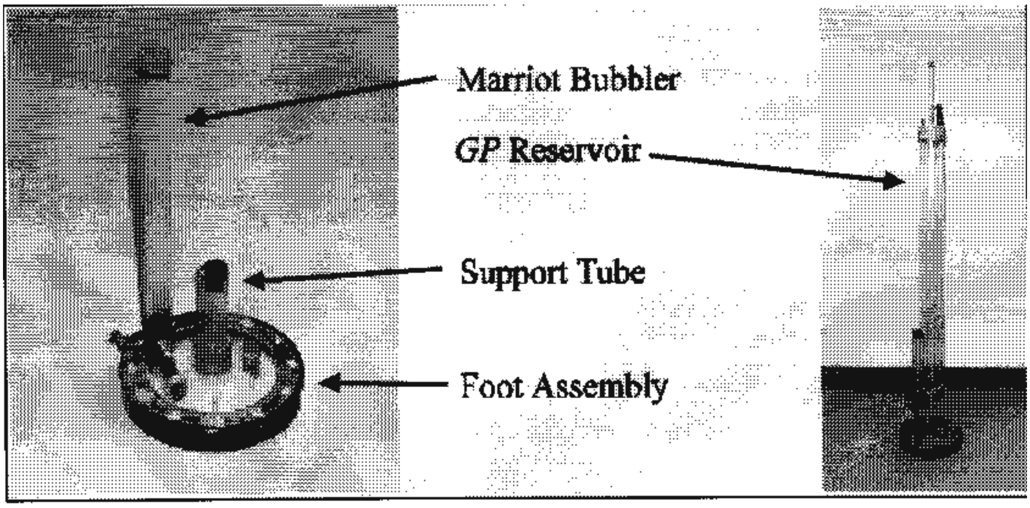

Figure 6: Components of the tension infiltrometer (left) and the GP reservoir connected to the TI (adapted from Meiers, 2002).

4.1.2 Principles

White et al. (1992) reviewed the use of tension infiltrometers and presented alternative methods of measurement and analysis of the resulting data. Figure 7 presents a schematic description of the tension infiltrometer and its components. All Mariotte-type instruments operate on the same physical principles. The major components of a TI are (i) the bubbling tower, which contains the air-entry tubes that control tension at the soil surface, (ii) the water reservoir, in which the water level falls as water flows into the soil, (iii) the porous base plate, which establishes hydraulic continuity with soil, and (iv) data logger and pressure transducers (optional). Tension (negative pressure) in the air pocket at the top of the water reservoir is linearly related to the height of water in the column (Ankeny, 1992), e.g. a 1mm drop in water level corresponds to a 1mm decrease in tension in the air pocket. Thus, cumulative infiltration can be monitored by recording tension changes over time.

Measurements conducted with the TI require that the negative water pressure in the soil be transferred to the TI through the porous membrane, which requires a good hydraulic contact. A flat surface, free of debris or coarse particles, is favorable for making "good" contact with the TI membrane. In order to achieve these conditions, the soil surface is prepared by removing any occasional fragments (>2 mm) from the site or by locating a 20 cm diameter area free of coarse fragments (Meiers, 2002).

A support ring slightly larger than the TI foot assembly is placed on the ground. The bottom edge of the support ring is inserted about 0.5cm into the ground; this acts to support the contact material so a uniform thickness can be established (Reynolds, 1993). Once the foot assembly is positioned in the testing area and in good contact with the soil, the sides of the foot assembly must be sealed to the ground using a saturated soil paste made onsite with saturated material. When determining K_{fs} from a tension infiltrometer, the radius of the support ring is used for calculating the area of the infiltrating surface. The applied tension must be corrected when using a contact material. The following correction factor is given by Meiers (2002):

$\psi_s = \psi_b - T$ where, (11)

ψ_s is the negative pressure head at the soil surface [L];

ψ_b is the negative pressure head reading on the marriot bubbler [L]; and

T is the thickness of the contact material [L].

The use of a contact material is required in situations where the surface is irregular or not level. Contact materials should only be used where necessary since they can cause a difference between the pressure head set on the tension infiltrometer membrane and the pressure head at the soil surface (Azevedo et al., 1998). Contact materials shall have an air-entry value greater than the greatest suction being applied and a hydraulic conductivity greater than the material being tested (i.e. contact material is hydraulically "transparent"). Contact material, usually a silicate sand, must have a grain size distribution in the range of 53µm to 105µm (Meiers, 2002).

Poor contact results in poor data. The sand should have a conductivity greater than that of the soil being measured to avoid impeding flow. If too fine a sand is used, conductivities may be underestimated because of the impedance of the contact layer. Coarse sand, however, may not wet fully, which could also lead to underestimation of infiltration rates. If contact material conductivity is greater than soil conductivity, the maximum error in water potential at the contact-soil interface is the thickness of the contact layer itself. Therefore, the thickness of the contact layer should be kept to a practical minimum (Arkeny, 1992).

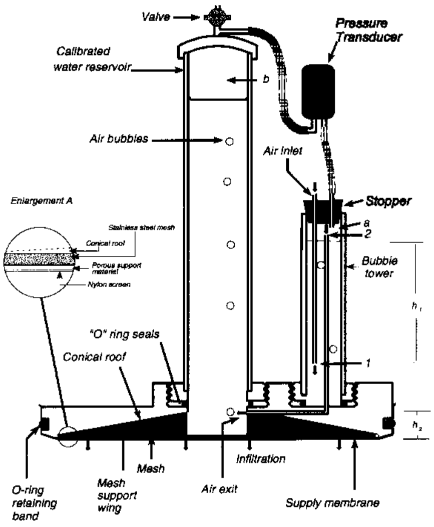

Figure 7: Example of a single reservoir tension infiltrometer with conical roof in base (after Evet et al., 1999).

4.1.3 Summary of field procedures

The typical approach to infiltration measurements using the tension infiltrometer is briefly summarized here (adapted from Ankeny, 1992):

1. Soil surface preparation: The surface crust or top 10 or 20m is carefully removed unless the crust itself is being tested.
2. Ring insertion: A sharpened ring is pushed a short distance (~1cm) into the soil to define the area of the infiltration surface and prevent lateral surface flow of ponded water, and cheesecloth is placed in the ring.
3. Filling with contact material: If necessary, contact sand is added to fill the inside of the inserted ring and leveled.
4. TI installation: A Tension infiltrometer is centered over the sand-filled ring, and the legs of the device are pushed into the soil until contact is made with the sand. This step can be adapted when using the TI-kit to adapt to the Guelph permeameter.
5. Measurements: Measurements are typically made from low to high tension.

Note: The air contained in the porous base plate must be removed by submerging the apparatus in water for a period of 24 hours.

A complete step-wise description of the field set up and operating procedures is provided by Ankeny (1992).

The time required to reach steady-state in unconfined infiltration measurements depends on initial soil water content and on hydraulic properties of a given soil. In general, drier soil and lower hydraulic conductivity result in the need for a longer infiltration period in order to reach steady-state infiltration. The change in rate over time should therefore be monitored to confirm that steady rates are reached. Not reaching steady-state results in an overestimate of hydraulic conductivity (K_{fs}). Typically, Arkeny et al. (1990) suggest collecting data for 1000 seconds at each tension measuring from low to high tension under most condition. This methodology is normally adequate except for very dry and/or high bulk density porous media. In very porous or sandy soil, steady-state rates are reached much faster and times can be shorter. As a practical field guide, if a third of the 25,4 mm-diameter water reservoir has emptied, most likely more than enough water has been added to the soil wetting bulb for the infiltration rate to approach steady-state.

4.1.4 Analysis of field data

Wooding's equation for steady-state unconfined (three dimensional) infiltration rates is used in calculation hydraulic conductivities. The K_{fs} can be calculated by the following equation (Meiers, 2002):

$$K_{fs} = \frac{G_d a * Q_1}{r(1+G_d a * pr)\left(Q_1 \big/ Q_2\right)^P} \qquad (12)$$

where,

$$P = \frac{y_1}{(y_1 - y_2)} \qquad (13)$$

$$\alpha^* = \frac{\ln(Q_1/Q_2)}{y_1 - y_2} \qquad (14)$$

and,

K_{fs} (L/T) is the field saturated hydraulic conductivity;

Q (L^3/T) is the steady-state flow rate for the two water tensions;

r (L) is the radius of the infiltrating surface;

G_d is a dimensionless shape factor;

ψ (L) is the pore water pressure head for two different tensions (typically –5 and –10cm);

α^* (L^{-1}) is the texture/structure parameter.

Note: When using equation 12 (i.e. steady-state flow from two applied negative heads), the recommended procedure indicates that the lowest negative head (or lowest tension, e.g. –5cm) is applied first, followed by the greater negative head (e.g. –10cm) (Arkeny, 1992).

4.1.5 References for Tension Infiltrometers

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4.2 Single-ring Infiltrometer:

4.2.1 Overview

This category includes ponded infiltrometer, drum (cylinder) test and other types of single-ring apparatus.

The single ring apparatus typically consist of a cylindrical ring 30cm or larger in diameter that is driven about 5cm into the soil (Bouwer, 1986). Water is ponded within the ring above the soil surface (Photo 4). The upper surface of the ring is often covered to prevent evaporation. The volumetric rate of water added to the ring sufficient to maintain a constant head within the ring is measured. Alternatively, if the head of water within the ring is relatively large, a falling head type test may be used wherein the flow rate, as measured by the rate of decline of the water level within the ring, and the head for the later portion of the test are used in the calculations. Infiltration is terminated after the flow rate has approximately stabilized. The infiltrometer is removed immediately after termination of infiltration, and the depth to the wetting front is determined either visually, with a penetrometer-type probe, or by moisture content determination for soil samples.

A special type of single-ring infiltrometer called the pounded infiltration basin is presented in ASTM method D5126 but is not discussed here because it is seldom used.

It is good practice to establish the soil strata to be tested from the soil profile determined by the description of soil samples from an adjacent auger hole. The test site should be nearly level, or a level surface should be prepared. The test may be set up in a pit if infiltration rates are desired at depth rather than at the surface. In low permeability materials where test duration are expected to be considerable, provisions should be made to protect the test apparatus from direct sunlight, which could promote water evaporation from the rings and/or water level fluctuation in the Mariotte reservoir.

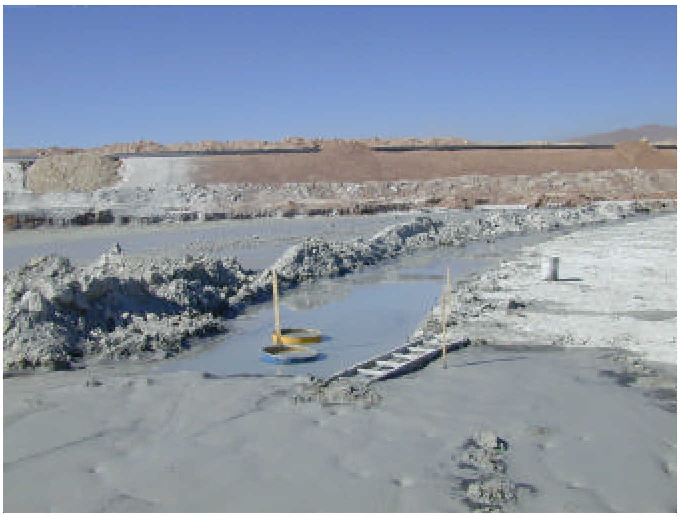

Photo 4: Single-ring infiltrometers installed in "freshly deposited mine tailings (courtesy of Robertson GeoConsultants Inc., 2003).

4.2.2 Principles

Ring infiltrometers are often used for measuring the water intake rate at the soil surface. Water flow from a single-ring infiltrometer into soil is a 3D problem (Reynolds and Elrick, 1990). The total flow rate into the soil from a single-ring infiltrometer is a combination of both vertical and horizontal flow.

Most infiltrometers generally employ the use of a metal cylinder placed at shallow depths into the soil (Photo 5a, 4b), and include the single ring infiltrometer, the double-ring infiltrometer and the infiltration gradient method. Various adaptations to the design and implementation of these methods have been employed to determine the field-saturated hydraulic conductivity of material within the unsaturated zone. The principles of operation of these methods are similar in that the steady volumetric flux of water infiltration into the soils within the infiltrometer ring is measured. Saturated hydraulic conductivity is derived directly from solution of Darcy's Equation for saturated flow. Primary assumptions are that the volume of soil being tested is field-saturated and that the saturated hydraulic conductivity is a function of the flow rate and the applied hydraulic gradient across the soil volume. Additional assumptions common to infiltrometer tests are as follows:

- The movement of water into the soil profile is 1-D downward;
- Equipment compliance effects are minimal and may be disregarded or easily accounted for;
- The pressure of soil gas does not offer any impedance to the downward movement of the wetting front;
- The wetting front is distinct and easily determined;
- Dispersion of clays in the surface layer of finer soils is insignificant;
- The soil is non-swelling, or the effects of swelling can easily be accounted for.

Photo 5a,b: Set up for single-ring infiltrometer test in mine tailings with “clean hole” base (a) and gravel-geofabric base (b) to simulate underdrain bottom condition (courtesy of Robsertson GeoConsultants Inc., 2003).

4.2.3 Analysis of field data

A method to calculate the K_s from data obtained from a pressure or ring-infiltrometer for both early-time and steady-state infiltration was developed by Reynolds and Elrick (1990), Elrick and Reynolds (192) and Elrick et al. (1995). Their steady-state method uses a shape factor based on Garder’s (1958) relationship between hydraulic conductivity and matric pressure head.

Wu et al. (1999) developed new single-ring infiltrometer methods that use a generalized solution to measure the field saturated hydraulic conductivity (K_{fs}). The K_{fs} values can either be calculated from the whole cumulative infiltration curve (Method 1) or from the steady-state portion of the cumulative infiltration curve by using a correction factor (Method 2).

The generalized equation (Wu and Pan, 1997) is

$$i/i_c = a + b(t/T_c)^{-0.5} \qquad (15)$$

where

$$i_c = f\,K_s \qquad (16)$$

$$f = \frac{H + f_m^{'}/K_s}{G^*} + 1 \qquad (17)$$

$$G^* = d + r/2 \qquad (18)$$

$$f_m^{'} = \int_{h_i}^{0} K'(h)dh \qquad (19)$$

$$T_c = \frac{\Delta q\, l s}{K_s - K_i} \approx \frac{\Delta q}{K_s^2} f_m^{'} \text{ since } K_i << K_s \text{ undermost filed soil moisture conditions.} \qquad (20)$$

where $\Delta\theta = \theta_0 - \theta_i$.

In equations 15 through 20, a and b are dimensionless constants (a=0.9084, b=0.1682) from the generalized equation, H is the ponded depth in the ring, d is the ring insertion depth, r is the radius of the ring infiltrometer, K_s and K_i are the hydraulic conductivity at saturated water content (θ_0) and at initial water content (θ_i), h and h_i are matric and initial matric pressure heads, and K’(h) is the modified van Genuchten hydraulic conductivity pressure head function (Wu and Pan, 1997).

There are two ways to calculate Ks by applying the generalized infiltration equation to the measured infiltration curves from a single-ring infiltrometer. Method 1 is based on the cumulative infiltration equation. By integrating equation 15 from t=0 to t=t, we have:

$$I = a\ f\ K_s\ t + 2\ b\ f\ K_s (tT_c)^{0.5} \tag{21}$$

or $$I = A\ t + B\ t^{0.5}$$

Solving for Ks yields:

$$Ks = \frac{\Delta q\, l_s}{T_c} \tag{22}$$

Method 2 is based on the assumption that the last part of the infiltration has reached steady-state, which leads to the following relation:

$$I = a\ t + c = a\ f\ K_s + c \tag{23}$$

Solving for K_s yields:

$$K_s = A\ /a\ f \tag{24}$$

where A is the slope and c is the intercept from the linear regression, and f, which is defined in equation 17 can be estimated by

$$f \approx \frac{H + 1/a}{G*} + 1 \tag{25}$$

since $a = K_s / f_m \approx K_s / f'_m$, where ϕ_m is the matrix flux potential.

The advantage of utilizing the generalized equation to calculate K_s is that one can use the whole infiltration curve without segregating early-time and steady-state infiltration components. As well, the method does not require the assumption of a fixed $-\alpha = K_s/\phi_m$ for a particular soil. Wu and Pan (1997) further showed that α values suggested by Elrick and Reynolds (1992) for soils of different textures and structures can be used in calculating K_s from cumulative infiltration curves.

By applying scaling theory, Wu and Pan (1997) developed a generalized solution for single-ring infiltrometers, and showed that infiltration rate from a single ring infiltrometer is approximately *f* times greater than the 1D infiltration rate for the same soil, where *f* is a correction factor that depends on soil initial and boundary conditions and ring geometry. For a relatively small ponded head, the 1D infiltration rate of a soil is approximately equal to the field saturated hydraulic conductivity (K_{fs}).

For a fine soil, constant and falling head methods produce very similar infiltration rates (Wu and Pan, 1997) for a time period practical for field measurements (e.g., a few hours) because the head drop in the ring is small. However, for a coarse textured soil, the head drop is fast. Thus, the falling head method measures substantially lower infiltration rates if the measurement is taken when the ponded head is small. Measurements taken immediately after refilling the infiltrometer will be close to the infiltration rate by the constant head method.

The effect of layering on infiltration measurement is time and position dependent. For a limited period of measurement, the layering effect is more profound when the underlying soil is closer to the surface. The time required for the wetting front to reach the interface of texture discontinuity can be estimated from the correction factor, *f*, and the cumulative infiltration.

4.3 Ponded Infiltrometer

4.3.1 Overview

The Ponded Infiltrometer is a variant of a single ring infiltrometer. Figure 8 presents a schematic diagram of the ponded infiltrometer. Bouwer (1986) stated that cylinder infiltrometers are typically 0.30m in diameter but that infiltrometers of 1m in diameter or greater should be used to obtain meaningful results. However, driving large cylinders into most soils may disrupt soil macropores and other structural features affecting infiltration. Soil variability necessitates infiltration measurements at many locations to characterize infiltration accurately on a field scale. Because of size and set-up time required for existing cylinder infiltrometers, infiltration measurements at multiple sites are difficult to obtain in a reasonable length of time.

The ponded infiltrometer is a self-regulating, single-ring infiltrometer which is simple in design and operation and can be used with a variety of water containment rings. The device allows (i) accurate control of ponded water height, (ii) precise measurement of water flow, (iii) direct delivery of water into the containment ring, and (iv) rapid setup and transport.

Obtaining data from these permeameters is a lot easier than with other single-ring devices or with the double ring infiltrometer, although it is a lot more complicated to analyze due to the flow being in three dimensions. When analyzing this data, absorption and capillary forces, which act in all directions, and the geometry of the water source have to be considered (White et al, 1992). When using this device, a good intimate contact between the disc and the soil surface needs to be established, e.g. fine sand. A drawback of using such a material is that it will interfere with the measurements especially in the early stages of infiltration giving inaccurate sorptivity values. Another disadvantage when using the ponded infiltrometer is that if there is a large macropore in the site the water tower may not be able to supply water quick enough, also causing inaccurate results.

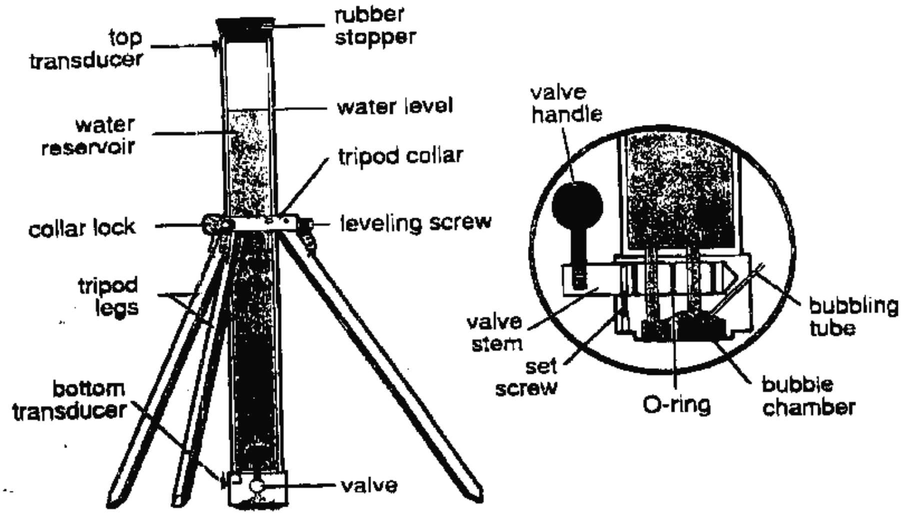

Figure 8: Schematic diagram of a ponded infiltrometer (after Ankeny, M.D., 1992)

4.3.2 Principles

The major components of the ponded infiltrometer are a Mariotte reservoir, a valved base, a containment ring and a tripod (see Figure 8, adapted after Prieksat et al., 1992). Optionally, a datalogger connected to two pressure transducers at the top and base of the water column can be used for automating the water flow measurements. Prieksat et al., (1992) describe the design for an automated, self-regulating ponded (single-ring) infiltrometer. Commonly, the water reservoir and the base are constructed of plastic polycarbonate. A rubber stopper is used to seal the top of the reservoir after filling. Pressure, created by pushing the stopper into the reservoir, starts water flow out of the base when the base valve is opened.

The base consists of a bubble chamber, and bubbling tube, a high-flow air-impermeable nylon membrane, two ports and a two-port valve. The bubble tube regulates the height of water ponded on the soil to +/-1mm. The bubble tube is adjusted up or down within the bubble chamber to raise or lower, respectively, the height of the ponded water in the ring from 0.5 to 1.0cm. This means that the water level in the containment ring can be adjusted without having to raise or lower the entire Mariotte reservoir as is required by previous designs. Because water flow from the device is partly determined by the ponded water height, water heights of <1.0cm will minimize the size of the water reservoir required to make infiltration measurements.

Two ports connect the Mariotte reservoir to the bubble chamber. The two-port valve is opened during measurement and is closed for movement between sites. The bubble chamber is design to funnel air bubbles up through only one of the two ports. Thus, only water flows through the other port and air bubbles do not limit water flow. Having a second port with unrestricted water flow reduces water-height fluctuations in the containment ring and thus increases measurement precision.

A low-impedence nylon filter covers the bottom of the base, which helps to disperse water flowing from the Mariotte reservoir and limit disturbance of the soil surface. The nylon filter also prevents air from entering the device, except through the bubble tube. The membrane provides a direct link between water ponded in the containment ring and water contained in the Mariotte reservoir without allowing air to enter the system.

Much discussion has occurred about the size of the containment ring that is required to obtain accurate infiltration data. This question remains unanswered, but scaling the dimensions of the device to fit the desired conditions of specific studies will allow the device to be used with a variety of containment ring sizes.

A unit change in water height in the Mariotte reservoir causes a unit change in air pressure above the water (Constantz and Murphy, 1987). Thus, water flow from the reservoir can be calculated from the change in air pressure in the reservoir with time.

4.3.3 Symmary of field procedures

The typical approach to infiltration measurements using the ponded infiltrometer is briefly summarized here (after Ankeny, 1992):

- Soil surface preparation: The soil surface crust or top 10 or 20mm is carefully removed ub a 150mm diameter area, unless the crust itself is being tested.
- Ring insertion: A sharpened ring is pushed a short distance (~1cm) into the soil to define the area of the infiltration surface and prevent lateral surface flow of ponded water, and prevent disruption of the soil structure. Cheesecloth (or geotextile) is normally placed in the ring to act as a separation medium.
- Measurements: The ponded infiltrometer is set over the ring and ponded measurements are made.

Infiltration can be measured with or without removal of any soil crust. A pointing trowel works well to prepare the surface. If the soil is too wet to avoid smearing, the measurement should wait. The skirted ring is gently pressed into the prepared surface up to the stop ring (the larger diameter outer ring). Next, layers of cheesecloth are placed on the soil surface in the ring to reduce soil slaking into macropores. Initially, the infiltrometer is centered and leveled above the containment ring by adjusting the angle of each tripod legs with the leveling screws. The pointed tripod legs can also be pushed into the ground to stabilize the device. After leveling and centering, the water reservoir and the base are lowered until the base makes contact with the containment ring.

The water reservoir tube and base are then locked in place with the collar lock so that the weight of the infiltrometer is supported by the tripod and not by the containment ring. Using this procedure allows the base and the bubbling tube to be placed at the same relative height above the soil surface each time the device is set up.The water valve can then be open, and water level adjusted prior to starting infiltration test proper.

4.3.4 Analysis of field data

During infiltration events, the water enters the soil in response to potential gradients of water potential and gravitational potential. The water potential term is governed by the dryness of the soil and the pore structure of the soil. These two factors combine to form a sorptivity factor which is made up of the combined influences of capillary action and adhesive forces to soil solid surfaces. The sorptivity of the soil is often expressed as "S". The gravity term is a constant for different soils and is due to the impact the pore size, continuity and distribution on the rate of water flow through soil under the influence of gravity. This term is known as "A".

Infiltrometer tests are useful for measuring the rate of infiltration but do not provide a direct measure of field-saturated hydraulic conductivity. Since entrapped air exists within the wetting front, true saturated conditions do not form during infiltration tests. Experience indicates that field saturated K_{fs} is approximately 50-75% less than K_s (Reynolds and Elrick, 1986).

The initial water infiltration rate is largely governed by the sorptive forces of the dry soil, this is then replaced once the soil wets up by the gravitational term. Equations describing infiltration include the Green et al. empirical model (described by Bouwer, 1986):

$$I = S\,t^{0.5} + A\,t \qquad (26)$$

where:

I = cumulative infiltration (cm of H_2O);
S_i = sorptivity of soil (determined from plot of cumulative infiltration against $t_{1/2}$);
T = time increment in seconds; and
A = approximates ½ K_{fs}.

4.3.5 References for Ponded Infiltrometers

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Wu, L. and L. Pan (1997). A Generalized Solution to Infiltration from Single-Ring Infiltrometers by Scaling. Published in: Soil Sc. Soc. Am. J. 61:1318-1322, 1997.

4.4 Double-ring Infiltrometer

4.4.1 Overview

This test method describes a procedure for field measurement of the infiltration rate of water into soils. A detailed description of the double-ring infiltrometer test method is provided in ASTM standard D3385-94. This test method is particularly applicable to relatively uniform fine-grained soils, with an absence of very plastic (fat) clays and gravel-size particles and with moderate to low resistance to ring penetration. This test method may be conducted at the ground surface or at given depths in pits, and on bare soil or with vegetation in place, depending on the conditions for which infiltration rates are desired. However, this test method cannot be conducted where the test surface is below the groundwater table or perched water table.

The double ring infiltrometer is a way of measuring saturated hydraulic conductivity of the surface layer, and consists of an inner and outer ring inserted into the ground. Each ring is supplied with a constant head of water from a Mariotte bottle. Hydraulic conductivity can be estimated for the topsoil when the water flow rate in the inner ring is constant.

Having the two rings eliminates the problem of overestimating the hydraulic conductivity in the field due to 3D flow. The outer ring supplies water, which contributes to lateral flow so that the inner ring is contributing only to downward flow.

Water moves from the Mariotte bottles into the rings via a tap at the base of the vessells until the height equals that of the base of the bubble tube. When water moves into the soil, reducing the height of ponded water to below that of the bubble tube, more water is fed into the ring.

Some draw-backs of the double ring are that it is very time consuming, requiring trial and error when adjusting the bubble tubes to get the water levels in each ring equal. The practicality of the instrument is reduced by the fact that the rings are extremely heavy to move. It also requires a flat undisturbed surface, which sometimes is not available. During the experiment it is sometimes necessary to refill the Mariotte bottles. To do this, the tap has to be turned off and this disrupts the experiment.

This test method is difficult to use or the resultant data may be unreliable in pervious or impervious soils (i.e. soils with hydraulic conductivity $>10^{-2}$cm/s or $<10^{-6}$cm/s) or in dry or stiff soils that most likely will fracture when the rings are installed.

4.4.2 Principles

The double-ring infiltrometer method consists of driving two open cylinders, one inside the other, into the ground, partially filling the rings with water, and maintaining the liquid at constant level. The volume of water added to the inner ring, to maintain the water level constant is the measure of the volume of water that infiltrates the soil. The volume infiltrated during timed intervals is converted to an incremental infiltration velocity, usually cm/hour and plotted versus elapsed time. The maximum steady-state or average incremental infiltration velocity, depending on the purpose/application of the test is equivalent to the infiltration rate.

The underlying principles and method of operation of the double ring infiltrometer are similar to the single ring infiltrometer, with the exception that an outer ring is included to ensure that one-dimensional downward flow exists within the tested horizon of the inner ring. Water that infiltrates through the outer ring acts as barrier to lateral movement of water from the inner ring. Double-ring infiltrometers may be either open to the atmosphere, or most commonly, the inner ring may be covered to prevent evaporation. For open double ring infiltrometers the flow rate is measured directly from the rate of decline of the water level within the inner ring for falling head tests, or from the rate of water input necessary to maintain a stable head within the inner ring for the constant head case. For sealed double ring infiltrometers (see below), the flow rate is measured by weighing a sealed flexible bag that is used as the supple reservoir for the inner ring.

ASTM method D 5093 – 90 describes an alternative double-ring infiltrometer method using a sealed inner ring for field measurement of infiltration rate through soils. Briefly, the infiltrometer consists of an open outer and a sealed inner ring. The rings are embedded and sealed in trenches excavated in the soil. Both rings are filled with water such that the inner ring is submerged.

The rate of flow is measured by connecting a flexible bag filled with a known weight of water to a port on the inner ring. As water infiltrates into the ground from the inner ring, an equal amount of water flows into the inner ring from the flexible bag. After a known interval of time, the flexible bag is removed and weighed. The weight loss, converted to volume, is equal to the amount of water that has infiltrated into the ground. An infiltration rate is then determined from this volume of water, the area of the inner ring, and the interval of time. This process is repeated and a plot of infiltration rate versus time is constructed. The test is continued until the infiltration rate becomes steady or until it becomes equal to or less than a specified value.

The following discussion focuses on standard double-ring infiltrometer method (i.e. sealed-inner ring is not covered).

4.4.3 Summary of field procedure

After a test site has been selected and the soil surface has been prepared, the outer ring is driven into the soil using a driving cap on top of which a wood block can be used to absorb the blow from a sledge hammer. The outer ring is inserted by moving the wood block around the edge of the driving cap. The ring is inserted to a depth that will (a) prevent the test water from leaking to the ground surface surrounding the ring, and (b) be deeper than the depth to which the inner ring will be driven. A depth of about 15cm is usually adequate.

Once the outer ring is in place, the inner ring can be centered inside the large ring and driven to a depth that will prevent leakage of water to the ground surface surrounding the ring. A depth of about 5-10 cm is usually adequate. Both the outer and the inner rings should be level. The soil surrounding the wall of the ring (s) should be exempt of excessive disturbance. In case extensive cracking or heave are observed, the ring (s) should be reset to a different location using a technique that will minimize such disturbance.

There are three ways to maintain a constant head (water level) within the inner ring and annular space between the two rings: manually controlling the flow of liquid, the use of constant-level float valves, or the use of a Mariotte tube. The latter option is the preferred one since it auto-regulates water flow to the ring. A pair or water bottles is used to fill both rings with water to the same desired depth in each ring. The water flow from the Mariotte tube can then be initiated. As soon as the fluid level becomes constant, the water level in the inner ring and in the annular space is measured (and recorded) to the nearest 2 mm using a ruler or a tape measure. The water level is maintained at a selected head (level) in both the inner ring and annular space between rings throughout the test to prevent flow of water from one ring to the other.

The volume of water that is added to maintain a constant head in the inner ring and annular space during each timing interval is determined by measuring the change in elevation of the water level in the appropriate graduated Mariotte tube. For average soils, the volume of water used to maintain the head is recorded at every 15 min intervals for the first hour, 30 min for the second hour, and 60 min during the remainder of a period of at least 6 hours, or until a relatively constant infiltration rate is achieved. The appropriate reading frequency may be determined only through experience and may be more frequent for high-K materials.

4.4.4 Analysis of field data

As with the single ring infiltrometers the wetting front is allowed to advance below the bottom of the ring, but it is assumed that infiltration through the outer ring functions as an effective barrier to lateral flow beneath the ring. However, the accuracy of this assumption may be limited.

The volume of water used during each measured time interval is converted into an incremental infiltration velocity for both the inner ring and annular space using the following equations:

For the inner ring, calculate as follows:

$$V_{IR} = \Delta V_{IR} / (A_{IR}.\Delta t) \qquad (27)$$

where:

V_{IR} = inner ring incremental infiltration velocity (cm/hr);
ΔV_{IR} = volume of water used during time interval to maintain constant head in the inner ring (mL);
A_{IR} = internal area of inner ring (cm^2), and
Δt = time interval (hour)

For the annular space between rings, calculate as follows:

$$V_A = \Delta V_A / (A_A.\Delta t) \qquad (28)$$

V_{IR} = annular space incremental infiltration velocity (cm/hr);
ΔV_{IR} = volume of water used during time interval to maintain constant head in the annular space between the rings (mL);
A_{IR} = area of annular space (cm^2), and
Δt = time interval (hour)

The infiltration rate calculated with the inner ring should be the value used for results if the rates for the inner ring and annular space differ. The difference in rates is due to divergent flow.

4.4.5 References for Double-Ring Infiltrometers

ASTM, 1998. Standard method D 3385 – 94 – Standard Test Method for Infiltration Rate of Soils in Field using Double-Ring Infiltrometer. Annual Book of ASTM Standards 2001, Section 4: Construction, Vol. 04.08 Soil and Rock (I): D 420 – D 5779, pp.368-374.

ASTM, 1998. Standard method D 5093 – 90 – Standard Test Method for Field Measurement of Infiltration Rate of Soils using Double-Ring Infiltrometer with a Sealed-Inner Ring. Annual Book of ASTM Standards 2001, Section 4: Construction, Vol. 04.08 Soil and Rock (I): D 420 – D 5779, pp.1041-1054.

4.5 Other Methods

Other methods for determining the field-saturated hydraulic conductivity of soils include, for example, the double tube (DT) test method. This method is described by McCall et al. (2002), and summarized in ASTM Standard D5126. This test method will not be discussed any further in this document because it requires considerably more equipment and time, and is less popular than the other methods presented above. Briefly, the DT method utilizes two coaxial cylinders positioned in an auger hole. The difference between the rate of flow in the inner cylinder and the simultaneous rate of combined flow from the inner an outer cylinder is used to calculate K_{fs}.

The ex-situ core permeametry technique is another field-based method that is convenient for determining K_{fs} of a discrete soil stratum. The method is presented by Chappel and Ternan (1997) and is not discussed further in this summary. The interested reader is encouraged to consult references provided below for more details on principles and procedures regarding these and other methods.

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McCall, W., Butler, J.J., Healy, J.M., Lanier, A.A., Sellwood, S.M., Garnett, E.J. (2002). A Dual-Tube Direct-Push Method for Vertical Profiling of Hydraulic Conductivity in Unconsolidated Formations. Environmental & Engineering GeoSciences, Vol. VIII, No.2, May 2002, pp. 75-84.

Appendix A. Field sheet for Guelph permeameter testing.

Guelph Permeameter Field Sheet-Standardized Procedure For Permeameter Readings & Calculations

PROJECT NAME:	TEST LOCATION:
PROJECT NUMBER:	BORE/HOLE ID:

Reservoir Constants: (See label on Permeameter)

Depth of Well Hole:______________

Combined Reservoirs	**X**	**cm^2**	☐	Check Used
Inner Reservoir	**Y**	**cm^2**	☐	Reservoir

Note: In standarized procedures the radius of the well hole is always 3.0 cm

1st set of readings, height of water in well (H1) set at 5cm

Reading Number	Time (hh:mm:ss)	Time Interval (min)	Water Level In Reservoir (cm)	Water Level Change (cm)	R. Water Level Change, R1 (cm/min)

2nd set of readings, height of water in well (H1) set at 10cm

Reading Number	Time (hh:mm:ss)	Time Interval (min)	Water Level In Reservoir (cm)	Water Level Change (cm)	R., Water Level Change, R2 (cm/min)

CALCULATIONS

R', the steady state rate of flow, is achieved when R is the same in three concecutive time intervals.

For the 1st Set of readings **R1'** = (R1)/60 = cm/sec

For the 2nd Set of readings **R2'** = (R2)/60 = cm/sec

K$_{fs}$ = [(0.0041)(_______________)(_______________)] - [(0.0054)(_______________)(____________)] = ____________ cm/sec
field saturated hydraulic conductivity — reservoir constant — R2' - steady state rate of flow — reservoir constant — R1' - steady state rate of flow

m = [(0.0572)( )( )] - [(0.0237)( )( )] = cm/sec
matric flux potential — reservoir constant — R1' - steady state rate of flow — reservoir constant — R2' - steady state rate of flow

a = (Kfs)/(m) = cm^{-1}
alpha par.

?? = (_______________) - (_______________) = ____________cm^3/cm^3
delta theta — field saturated water content of soil, in cm/cm — ambient water content of soil, in cm/cm

Moisture content

Estimated		CHECK
Measured		ONE

S = sqrt[2(Del)(m)] = cm.sec$^{-1/2}$
sorptivity

Robertson GeoConsultants Inc. Mining, Geotechnical and Environmental Engineers	TESTED BY:	DATE:
	REVIEWED BY:	PAGE OF