DEVELOPMENT OF AN IN SITU HYDRAULIC CONDUCTIVITY PROBE

AND

MEASUREMENT PROTOCOL

BY

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A thesis submitted

to the Faculty of Graduate Studies

in partial fulfillment of the requirements for the Degree of

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ABSTRACT

Manitoba's regulations require that the effective hydraulic conductivity for compacted earthen manure storages (EMS) may not be greater than 1×10^{-9} ms^{-1}. However, evidence by various researchers and interviews with members of the industry and government regulators suggest that seepage and damage to the clay liners may occur when EMS are not managed properly. Currently, the Livestock Manure and Mortalities Management Regulation M.R.42/98 under the Environmental Act does not have a protocol in place for measuring hydraulic conductivity in situ after the EMS are in operation. This study describes a method and a measurement protocol that can be utilized to determine hydraulic conductivity in situ and ensure compliance with Manitoba's regulations. An in situ hydraulic conductivity probe (K-Probe) has been developed to measure the hydraulic conductivity of the clay liner based on pore pressure dissipation rates. The pore pressure is induced by electrokinetic methods. Using modified Hvorslev's analysis, a probe constant has been determined based on K measurements on undisturbed soil cores that were obtained from the field. The hydraulic conductivity results obtained from the K-Probe method did not vary significantly from the laboratory (K_L) using ASTM testing methods ($\alpha = 0.05$). However, K_F did vary as a function of depth and location at the $\alpha = 0.05$ significance level. The K-probe measurement protocol determined that the K-Probe requires a minimum of 10 testing locations for a statistically valid assessment for compliance with Manitoba's regulations.

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I dedicate this thesis to my dear wife Michèle.

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1.0 INTRODUCTION

1.1 Background and problem

There is growing public concern over the long-term availability of clean water as a sustainable resource. There is increasing public pressure to develop new environmental standards and sustainable development practices. In Canada, there is an increasing demand for instrumentation and quantitative methods to ensure compliance with government environmental regulations. One area of agriculture that has greatly increased in economic growth and environmental scrutiny is Manitoba's pork industry. For example, Manitoba's pork industry is the largest agricultural industry in Manitoba. Manitoba is the largest hog-exporting province in Canada, and exports have increased more than five fold from 1981 to 2001 (Manitoba Agriculture and Food, 2002). All the manure and wastewater generated from livestock production operations is stored in various containment systems. The most common and economical type of containment systems are the clay lined earthen manure storages (EMS). Many new hog operations are being constructed in areas south of Winnipeg on "marginal soils" (low clay content soils), which require high clay content soils to be transported in to overlay coarser textured soils to meet the government guidelines on seepage limits. However, if there is a breach in the clay liners, then underlying aquifers can be compromised. Furthermore, EMS pre-dating the government regulations that came into effect within the last decade may be exempt from the current construction standards. Public concerns are exacerbated by the lack of methods for quantifying the potential contamination, which is costly and partially subjective. It is important to note that the potential economic burden incurred to clean up an aquifer is much greater than the cost of prevention.

Swine manure produces hydrogen sulphide, which is a toxic gas that has a very strong odour, but is highly soluble in water. Hydrogen sulphide is reduced by diluting manure with water; hence, increasing wastewater consumption and wastewater output. Manure also contains high levels of inorganic nitrogen. The main contributor to groundwater pollution is the highly soluble nitrate-nitrogen (NO_3^-), which is biologically oxidized from ammonium (NH_4^+) found in manure slurries. Ham and Desutter (2000) report concentrations of up to 2530 mg L^{-1} of total nitrogen in swine anaerobic lagoons. Levels this high in total nitrogen could produce an environmental hazard if the EMS clay liners became compromised in any way. Therefore, the Livestock Manure and Mortalities Management Regulation M.R.42/98 under the Environmental Act set the EMS clay liner hydraulic conductivity (K) limit to $1x10^{-9}$ ms^{-1} or less (The Environment Act, 1998). The hydraulic conductivity is a coefficient of proportionality between the rate of water flow through a permeable medium and the hydraulic gradient inducing the flow (Fetter, 1994). However, these guidelines are only as effective as the ability to effectively monitor in situ conditions and enforce them. Therefore, accuracy for in situ measuring devices is of utmost importance to ensure consistent, non-subjective, and legally defensible results.

A measurement technology and a protocol for effectively monitoring EMS clay liners is the focus of this study. A new type of in situ hydraulic conductivity probe has been developed for saturated clay soils. The probe has been tested in the field using the methods of electrokinetics and Hvorslev's analysis.

1.2 Thesis scope

As Manitoba's increasing demand for pork continues, there will be a need for manual inspections and in situ monitoring systems for anaerobic earthen manure storages. This thesis provides a potential in situ technology that measures hydraulic conductivity for clay liner management, integrity monitoring, and regulation compliance assessment after the EMS has been in use.

1.3 Objective

1.3.1 Primary objective

The primary objective of this thesis is to, construct, calibrate and test an in situ probe to measure hydraulic conductivity. This will help monitor the integrity of earthen manure storages (EMS) clay liners over time in order to meet the current government regulation.

1.3.2 Secondary objective

The second objective is to develop a protocol for using the in situ hydraulic conductivity probe to establish a consistent method of testing EMS clay liners. This protocol is based on methods described by Richards and Thompson (1989); a study for a permeability protocol developed for compacted clay liner construction quality control measures. This protocol utilizes log-normal sample distributions and statistical confidence intervals to effectively estimate the clay liner seepage rate from a pre-determined finite number of samples.

2.0 EARTHEN MANURE STORAGES IN MANITOBA

2.1 Introduction

Groundwater quality is of great concern to rural communities facing new and expanding livestock operations. Many rural residents obtain potable water from groundwater sources. Water for livestock and irrigation needs is also collected from surface and shallow groundwater aquifers. Potential contamination of these water resources is a sensitive issue for livestock operations and contamination prevention is of utmost importance.

In most livestock operations, nutrient rich manure slurries are stored in open earthen manure storages. This technology is the most common method to store livestock manures in agricultural operations. Earthen manure storages usually store a large volume of liquid manure and treat manures as their solid's content decreases from 80 to 85 % by means of biological digestion. The attractiveness of the common EMS is the ability to store typically 8000 to 16 000 m^3 of manure in a relatively economical holding system.

Earthen manure storages in the past have been designed on the basis of empirical assumptions, rather than sound scientific research (Chang *et al.*, 1974). The EMS consist of a clay liner at the base of the storage (3 -5 m deep) that is compacted to form a layer 0.3 m to 1 m thick depending on provincial guidelines. The issuing of permits in Manitoba for design and construction is regulated by Livestock Manure and Mortalities Management Regulation M.R.42/98 (Section 6) under the Environmental Act. This regulation states the EMS clay liner shall be compacted to an effective hydraulic conductivity of less than 1 x 10^{-9} ms^{-1}. However, this regulation does not explicitly outline the clay liner thickness and is therefore at the discretion of the Manitoba Conservation representative.

Current methods of determining hydraulic conductivities include laboratory testing methods as well as in situ methods. Laboratory methods include remoulded soil samples acquired from auger stems during drilling for site investigations, or cores taken from Shelby Tubes during hollow stem drilling. Usually, particle analysis and Atterberg limits are conducted on these samples and hydraulic conductivities are estimated from this data. The hydraulic conductivities determined by laboratory methods represent the clay liner's permeability at the time of construction. These investigations are cumbersome and expensive and may not accurately represent in situ conditions. However, piezometer slug tests are performed on site, but are usually only done prior to EMS construction for local hydrogeological background information. After commissioning, seepage (in situ hydraulic conductivity) monitoring is not done.

2.2 Groundwater quality concerns with seepage

Nitrate pollution is a serious problem if the seepage reaches the groundwater aquifers. Many rural communities depend on ground water for potable water supplies. The government uses environmental standards to set appropriate guidelines to ensure the health and safety of rural communities with intensive agricultural demographics. For instance, in Canada, the guidelines for drinking water quality state that the maximum allowable concentration of nitrate nitrogen is to be 10 mgL^{-1} (Health Canada, 2002).

main heath effects related to nitrogen compounds are given by Domenico and Schwartz (1998) as:

1) methemoglobinemia, a type of blood disorder in which oxygen transport in young babies or unborn fetuses is impaired; or
2) the possibility of forming cancer-causing compounds after drinking contaminated water.

2.3 Manitoba's EMS construction regulations

Manitoba's manure storage facilities siting and construction regulations (Livestock Manure and Mortalities Management Regulation 42/98, Schedule A, Sections 5 and 6, March 30, 1998) is summarized below. Figure 2.1 illustrates the cross sectional view of a typical EMS in Manitoba. Regulation 42/98, applies to livestock agricultural operations that are greater than 400 animal units. By definition, one animal unit is equivalent to the number of livestock that excrete 73 kg of total nitrogen in one year. Therefore, this regulation does not apply to agricultural operations that produce less than 29 000 kg of total nitrogen per year. Moreover, if an agricultural operation has less than 400 units in each category of livestock (eg. 300 poultry units, 300 hog units, 300 cattle units) this regulation also does not apply. This regulation only applies to the construction, modification, or expansion of a manure storage facility after the law was passed.

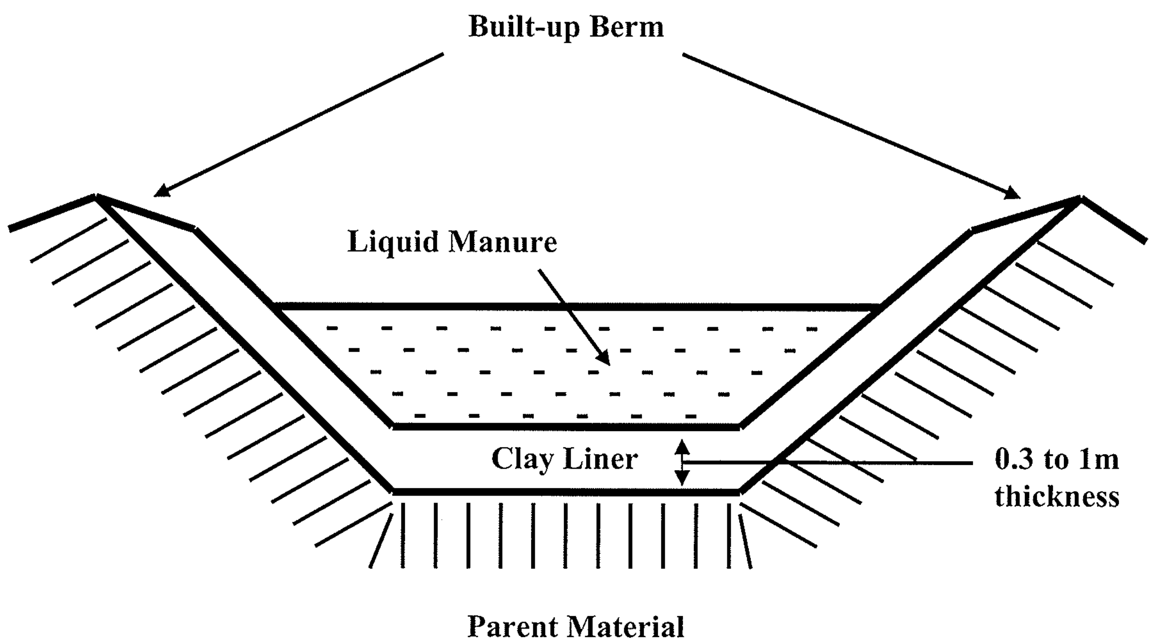

Figure 1.1 A cross sectional diagram of a typical EMS in Manitoba. The storage liner consists of a compacted clay liner.

2.3.1 Siting requirements

The construction of a manure storage facility under "Regulation 42/98" requires:

1. the site to be 100 m from surface water;
2. the site to be 100 m from a sinkhole, spring or well;
3. the site to be 100 m from the boundaries of the agricultural operation.

2.3.2 General construction requirements

The construction of a manure storage facility under "Regulation 42/98" states that:

1. Earthen manure storage capacity shall be sufficient to entirely encompass manures produced by the agricultural operation until the time of removal.
2. Topsoil is to be stripped where dyke is to be constructed before excavation and compaction
3. All excavated material shall be constructed using 0.15 m lifts and compacted;
4. Where a synthetic liner is not required, the sides and bottom of EMS shall be disced to a minimum depth of 0.20 m and compacted with a fully ballasted sheepsfoot packer to within 95% of maximum Standard Proctor dry density at a moisture content 0.9 and 1.2 optimum;
5. Construction shall be completed at temperatures greater than freezing;
6. Where the proposed site is located in the unsaturated portion of an aquifer, or where there is at least a 2 m of overburden materials having a hydraulic conductivity of 1×10^{-9} ms^{-1} or less between the bottom of the constructed

facility and the uppermost part of an underlying aquifer; a synthetic liner shall be installed;

7 The bottom of the EMS will be at least 1m above the elevation of the underlying water table;

8 Dyke protection shall be installed in the form of rip-rap or concrete, at the access ramp where agitation equipment of pump is rolled down, at the point of discharge of the inlet pipe where manure is pumped in, and at the overflow channel for a two or more cell facility.

9 All dykes need to be seeded to grass within one year of construction.

Regulation 42/98 is written in a general format which allows for subjectivity from the Director of Livestock Manure and Mortalities Management. Regulation 42/98 does not specify the overall clay liner thickness upon construction. However, Benson and Daniel (1994b), recommends minimum thickness of compacted clay liners to be 0.6 to 0.9 m. Furthermore, Regulation 42/98 does not provide recommendations or criteria for maintenance, measurement of in situ hydraulic conductivity, and long term groundwater quality management. In the Province of Ontario, the Ministry of Agriculture and Food has provided a more explicit draft protocol (Province of Ontario Ministry of Agriculture and Food, 2002) for construction and siting nutrient storage structures of various types. In this protocol, a minimum compacted clay liner thickness of 1.0 m is specified.

2.4 EMS seepage investigations

There have been various investigations into seepage losses from EMS using different methods and systems such as laboratory methods, in situ permeability instruments, groundwater tracer methods (contaminant transport), small scale pilot test, and whole mass- balance methods. These methods are listed in order of increasing complexity but are not defined as methods of increasing accuracy. However, the cost of implementation is proportional to complexity. Seepage studies have been well researched and documented by many investigators (Benson, *et al.* 1999; Benson and Daniel, 1994a; Benson and Daniel, 1994b; Culley and Phillips, 1989; Ham, 2002; Ham and DeSutter, 1999; Hegg *et al.*, 1979; McMillan and Woodbury, 2000; Parker *et al.*, 1999a; Parker *et al.*, 1999b; Robinson, 1973; and Westerman *et al.*, 1995).

In 1979, a water quality study was carried out on manure storages by the Water Resources Research Institute (WRRI) at Clemson University in South Carolina. The objective of this study was to obtain water samples from numerous wells installed around three swine manure lagoons to determine if water quality degradation had occurred over a two-year period. All the manure storages were newly constructed but only two were used to their design capacity. In their conclusions, two of the lagoons that operated normally, had one or two of the observation wells exhibit elevated pollutant concentrations approaching levels found in the manure itself and was most likely caused by heterogeneous soils. They also concluded based on a water budget, a seepage rate of 665 litres per day (0.665 m^3 day^{-1}) and contaminants was detected in wells two meters from the lagoon. This type of contamination spill could be devastating in a soil formation containing sand veins below or beside the in situ clay liner. High water table could contribute to rapid spread of the contaminated plume. Robinson (1973) concluded that after cattle waste was added to a 1.2 x 5.8 x 23.5 m digestion pond, the seepage rate decreased from 0.11 md^{-1} to 0.006 md^{-1} in 3 months. After 6 months the seepage decreased to 0.003 md^{-1}. This study suggests that livestock manure can decrease the effective bulk hydraulic conductivity through a clay liner.

2.5 Self sealing mechanisms of clay liners

When manure is first applied to clay liners, there is a dramatic decrease in the effective seepage rate from the EMS. Researchers Barrington *et al.* (1987a), Barrington *et al.* (1987b), Chang *et al.* (1974), and Davis *et al.* (1973), all investigated the self sealing mechanisms caused by pore blockage in the surface of the clay liner. Barrington *et al.* (1987a) and Fonstad (2000) summarized numerous researchers conducting studies on the sealing of soil by manures. Chang *et al.* (1974), conducted important research investigating the change in hydraulic conductivity of soils as a function of wastewater application over time and the chemical processes at the bottom of wastewater ponds. Four soil types were packed into acrylic columns, which were later retrieved and analysed. The researchers concluded that the hydraulic conductivity decreased after different periods up to sixty-nine days of submergence. The authors hypothesized that the surface layers of the submerged cores became physically clogged by entrapment of suspended and colloidal material. They also found an increase in the polysaccharide content in the surface layers at the final round of sampling, which correlated well with a further decrease in hydraulic conductivity. However, the quantity of pollutants which permeated the core prior to the seal formation could be a source of groundwater pollution in areas with shallow ground water tables in marginal soils. The common agreement among these researchers is that EMS seepage rates decrease substantially over the first few months of operation.

Most manure waste storages are subjected to annual or semi-annual pump-outs, using methods of agitation and mixing with high volume pumps, which remove the manure and potentially clay liner materials. However, these studies did not investigate the EMS operation and maintenance aspects in order to determine the dynamic effects of mixing and agitation on the clay liner sealing mechanisms.

2.6 Evidence of potential EMS seepage

Preliminary results from experiments conducted on several lagoons by University of Manitoba researchers have found increases in hydraulic conductivities in the bottom of EMS as compared to side retraining walls (Wiebe, 1998). This finding supports the idea that the bottom of the lagoons may have degraded from scouring during the pumping stages of waste removal. Large auger pumps are usually placed on the bottom of the lagoons to stir up the solids settled at the bottom. Newer facilities have rip-rap (concrete mixed in with large stones) or one or two concrete pads poured at the location of pumping. However, older facilities may not. This study also suggests that field hydraulic conductivity, K_F, may vary as a function of depth. If the upper clay layers become less compacted and the Atterberg limits change, the K_F could increase near the liner's surface and further promote the scouring processes. Benson *et al.* (1999), completed a data-base consisting of 85 full scale compacted clay liners to evaluate K_F, that were intended to achieve a K_F less than 1×10^{-9} ms^{-1}. Surprisingly, only 74 %

achieved this criterion. This evidence suggests that there may be a large number of EMS in Manitoba and throughout Canada that could have a K_F greater than $1x10^{-9}$ ms^{-1}.

2.7 Industry perspectives for EMS in Manitoba

Earthen manure storages have been designed and improved over the years due to engineering experience and government regulations. Information in this sub-section has been gathered from interviews with members of the industry and government regulators.

2.7.1 EMS construction methods and costs

There are two types of EMS Cells:

One type of EMS is referred to as a "Cut and Fill":

This type is constructed in areas where the parent material has a sufficiently low K; less than $1x10^{-9}$ ms^{-1}. The earth is dug out 2 to 3 m deep and berms are built up on the sides. The average depth of an EMS is 4 m and has a storage capacity ranging from 8000 to 16 000 m^3 depending on the cell type. The primary cells are usually the smallest because they are used to collect the majority of the solids. The secondary and tertiary cells have a greater volume as they contain mostly liquids.

Another type of EMS is referred to as a "Compacted Clay Liner":

The construction process is similar to the above facility but this EMS involves utilizing the parent material on site or clayey material located off site with the sufficient soil texture or a combination of both. Consulting firms designing the clay liners, usually use a lining material that is minimally 30% clay and has a liquid limit of 30 as a general design requirement. After the material has been removed, the liner is created by compacting the disked clay in 0.15 to 0.20 m lifts (layers) up to a 1 m thickness establishing 95 % Standard Procter Moisture Density (ASTM D 698-78) using a sheep's foot packer.

In some areas, the contamination risk is high due to coarse textured soils and high water tables. The previously mentioned EMS are combined with a synthetic liner that is virtually impermeable. As the level of sophistication increases, so does the cost. Table 2.1 provides the cost estimates for both types of EMS and other type of manure storages.

Table 2.1 Approximate 2003 cost estimates for EMS compared to other storage methods

Storage Type	Estimated Cost/m^3
Cut and fill	\$7.50
Cut and fill with synthetic liner	\$17.50
Compacted liner	\$12.50
Concrete Structure	\$20.00
Steel structure	\$32.50

It is evident from Table 2.1, EMS consisting of clay liners alone are significantly less expensive than other types of manure storage systems. Therefore, livestock operations using simple clay liners may become more profitable with lower initial start-up costs. However, long term maintenance and monitoring costs should be factored into the initial costs for comparison.

2.7.2 Cost of site investigations

During site assessments, samples are acquired typically by using 3, 10 m boreholes where soil logs and descriptions are documented. Soils obtained from bore hole investigations are bagged and sent for soil analysis. The samples are tested for Atterberg limits and two particle size tests. One particle size test contributes the sand, silt, and clay percentage and the other is a descriptive particle distribution curve for numerous particle sizes. From the results, which usually take more than two weeks, hydraulic conductivities are empirically estimated from these tests. At most EMS construction sites in Manitoba, in situ K tests are not conducted, and therefore, comparison with in situ K can not be ascertained. Laboratory hydraulic conductivity samples are not usually conducted unless underlying soils are of "marginal quality". Manitoba Conservation can request K tests to be done if the consulting company finds the soils too coarse textured. For example, in La Broquerie, south of Winnipeg, the area consists of sandy lenses, which require careful investigation and more auger tailings or Shelby Tubes to be assessed. Current practice for acquiring soil samples in the field for laboratory testing include disturbed knife samples which are taken from the auger stem, bagged, labelled, stored, and sent for analysis where they are remoulded for testing using ASTM

5048 – 90 constant head K methods. However, hollow-stem augers (used to retrieve intact cores) and Shelby Tubes (direct push core sampling) are not common practice for providing samples for lab analysis although they provide specimens more representative of in situ K. After preliminary results confirm marginal soils, Shelby Tube samples may be required from the site and are collected under Manitoba Conservation supervision. Typically, 3 to 6 cores are taken at specific locations determined by the site representative of Manitoba Conservation. The sampling locations are chosen arbitrarily at the government regulator's discretion. A core sample can be obtained using a Shelby Tube apparatus after construction of the clay liner. However, this practice is not undertaken. Should the results of the laboratory analysis for pre-construction site investigations comply with the soil-hydraulic parameters specified by Manitoba's guidelines, then the assumption is that the effective seepage rate through the constructed clay liner, meets the same hydraulic parameters as the laboratory analysis. This assumption is based on research carried out by Benson *et al.* (1994) and Benson *et al.* (1999), that provided recommended soil properties for EMS liner construction (Table 2.2).

Table 2.2 Soil properties used to achieve hydraulic conductivities of 1×10^{-9} ms^{-1} from Benson *et al.* (1994).

Soil Property	Minimum Requirements
%Clay content	≥15
Liquid Limit	≥20
Plasticity Index	≥7
% fines < 075mm	≥30

However, Benson *et al.* (1999) suggest these criteria act as a source of guidance and conclude that:

> "...in no circumstances should a soil be considered acceptable based solely on empirical predictions. Such predictions are not an acceptable surrogate for laboratory hydraulic conductivity testing."

A drill rig capable of drilling to a 10 m depth costs approximately $800 per hour and a field technician's charge-out rate ranges from $60 to $80 per hour. Table 2.3 illustrates the pricing in 2003 for various soil analyses by two environmental consulting firms in Winnipeg, Manitoba. The hydraulic conductivity analysis costs are high and contribute significantly to the site investigation process. For this reason, hydraulic conductivity analysis is minimized or not conducted at all.

Table 2.3 Approximate 2003 pricing for soil analysis by two soil testing companies in Winnipeg, MB.

Soil Analysis	Cost/Sample
Particle Size Hydrometer method	$25
Particle Size Pipette method	$40-$90
Atterberg Limits	$50-$68
Hydraulic Conductivity Shelby Tube	$500
Hydraulic Conductivity Remoulded Sample	$650
Porosity	$35

2.8 In situ hydraulic conductivity (K) methods

Most in situ hydraulic conductivity measurements are made from a series of instruments that are combined with data acquisition instrumentation and data processing software. Daniel (1989) describes the methods, assumptions and field data, and compares the relative advantages and disadvantages of 9 different in situ hydraulic conductivity methods for compacted clayey soils. ASTM 5126 – 90, compares field methods for determining hydraulic conductivity in the vadose zone, or unsaturated zone. Several of these in situ methods are described in the next sections. For simplification, the saturated hydraulic conductivity and field hydraulic conductivity will be defined as K_s and K_F, respectively for the rest of this thesis. Saturated hydraulic conductivity, usually tested in the laboratory, refers to water flowing through a soil in which the water filled voids are at a pressure greater than atmospheric pressure. Field hydraulic conductivity refers to water flowing through a soil in which the water filled voids are at a pressure greater than or less than atmospheric pressure depending on the in situ conditions. For example, the observed field saturated hydraulic conductivity will be greater under the watertable where the soil voids are exposed to pressures greater than atmospheric pressure. However, measurements taken above the watertable, where conditions are unsaturated, the observed field hydraulic conductivity will decrease.

2.8.1 Borehole methods

Borehole or augerhole methods are one of the most popular site investigative methods for estimating hydraulic conductivities for relatively shallow water tables. Hvorslev's Method or slug test method uses boreholes to insert piezometers into them. Piezometers are usually made of steel or PVC tube that is perforated near the bottom. When the static water level (H) is measured, a unit volume of water or metal slug is introduced or removed from the well. When a slug is introduced, then the water level will rise to the initial falling head H_0. As the water head decreases, time is recorded until the water level returns to the static piezometric water level H. Water levels can be measured accurately inside the piezometers with pressure transducers that measure the change in head pressure. This method is advantageous for site investigations to gather information on local hydrogeology, but is not very effective for clay liner investigation that range from 0.3 to 1 m in thickness.

2.8.2 Infiltration methods

The infiltration methods used to find in situ hydraulic conductivities are very useful for estimating surface infiltration rates or clay liner infiltration (seepage) rates. Popular single ring or double infiltrometers, which can either be open or sealed, are commonly used for clay liner testing (Benson *et al.*, 1999 and Daniel, 1989). These methods encompass large volume test pads that can effectively measure seepage prior to EMS commissioning. However, after an EMS contains any amount of waste, infiltrometer cannot be used.

2.8.3 Porous probe methods

Porous probes consist of a porous element at the end of a tube, which allows passage of water as inflow or outflow. The tube and porous tip are inserted into the soil and K_F tests are administered with falling or constant head type of arrangements. Porous probes are similar to borehole methods in that Hvorslev's falling and constant head equations with shape factors apply for saturated soils. The assumptions governing the Hvorslev equations apply to porous probe methods as Daniel 1989, describes:

> "It is assumed that the soil is homogeneous, isotropic, uniformly soaked, and incompressible; boundaries are at infinity; soil is not smeared across the porous element; effects of soil suction are negligible...."

Porous probes are generally used for saturated soil; however, a device known as the BAT^{TM} permeameter can also be used for unsaturated compacted clays. Out flow tests are conducted in this case where water is introduced into the system and dissipating pore pressures are measured over time (Torstensson, 1986). A major advantage of the BAT^{TM} permeameter is its delivery of quick responses of low K_F. BAT Geosystems in Sweden sells the BAT^{TM} permeameter and claims that typically testing times for soil K_F's are equal to:

K_F = $10E^{-7}$ ms^{-1} test time is 120 s

K_F = $10E^{-8}$ ms^{-1} test time is 1200 s

K_F = $10E^{-10}$ ms^{-1} test time is 18 000 s

The costs for the BAT^{TM} permeameter system is approximately $6500. This is an expensive hydraulic conductivity instrument, which samples a small soil volume. Daniel (1989), describes a number of advantages and disadvantages for the BAT^{TM} permeameter. A major disadvantage is the introduction of a fluid and resulting long equilibration times, which are unacceptable for use in EMS.

2.8.4 Water balance methods

Ham and DeSutter (1999) describe seepage losses from swine waste lagoons using a water balance method to measure clay liner K_F. The method is a mass balance method, which requires numerous parameter calculations and data collection. The water balance method can be described by the following equation:

$$\mathbf{Q_{in} + P = Q_{out} + E + S + St_f - St_i} \qquad \mathbf{(3.1)}$$

Where:

Q_{in}	=	waste input flow, md^{-1},
P	=	Precipitation, md^{-1},
Q_{out}	=	waste output flow, md^{-1},
E	=	Evaporation, md^{-1},
S	=	Seepage rate, md^{-1},
St_f	=	Final storage, md^{-1},
St_i	=	Initial storage, md^{-1}.

If all the variables in this equation are accounted for accurately, then the K_F may be easily determined. Unfortunately, this is not the case. Parker *et al.* (1999a) and Cumba and Hamilton (2002), describe numerous problems with under or over estimation of evaporation rates in water balance methods.

Borehole methods, infiltrometer methods, and porous probes, are common methods for in situ measurement of K_F, however, these methods are labour intensive, expensive, highly intrusive, and destructive. Borehole tests require a drill rig to auger through the soil and insert a piezometer or porous probe. For clay liner EMS monitoring, numerous large holes would be required for seepage measurements. This action itself would compromise the integrity of the liner if proper sealing precautions (post drilling) were not taken. A new type of measuring instrument that could minimize measuring costs and prevent damage to the compacted clay liners, while still obtaining accurate measurements of seepage, would be an asset for the livestock industry and government regulators.

2.9 Industry EMS monitoring and inspection

Some liners in Manitoba have been found scoured in some areas of the EMS, causing large holes in the liner where agitation and pumping have occurred. Initially, after EMS pump-out, Manitoba Conservation conducts routine checks on lagoons and mandate liner repairs only if visually apparent. However, the lagoons are usually never pumped out dry and therefore the visual method is inadequate for observing the bottom of the lagoon for problems caused by scouring. Secondly the when the EMS are drained then the sides of the berms are exposed to sun drying and cracking. This process can degrade the permeability qualities of the clay liner from preferential flow through macro-pores and channels.

3.0 PROBE DESIGN AND DEVELOPMENT

3.1 Introduction

A hydraulic conductivity probe (K-Probe) has been developed to measure the hydraulic conductivity of the clay liner based on pore pressure dissipation rates. The pore pressure is induced by electrokinetic methods. After field testing, using Darcy's Law and modified Hvorslev's analysis, a probe constant has been determined to predict the field saturated hydraulic conductivity (K_F) based on K measurements on undisturbed soil cores that were obtained from the field.

3.2 Probe development background theory

3.2.1 Darcy's Law

Darcy determined that for a cylindrical column filled with soil, the volume discharge rate, Q, is directly proportional to the head drop $(h_2 - h_1)$ and to the cross sectional area, A, but is inversely proportional to soil length (L). The head in this case refers to the sum of the water pressure head and elevation head above a pre selected datum at a particular instant in time. Darcy's law is formed by introducing a proportionality constant, K, known as the hydraulic conductivity to form the relationship:

$$Q = -KA\frac{h_2 - h_1}{L} \tag{3.2}$$

The negative sign indicates the flow occurs from higher head to lower head or in other words, in the direction of fluid head loss. If we divide both sides of the equation by the cross sectional area and assume a homogeneous isotropic porous medium, then the steady state flux in all directions x, y, and z is given by:

$$q_l = -K \left. \frac{\partial h}{\partial l} \right|_{l = x, y, z} \qquad (3.3)$$

where q is in units of velocity (or flux) and is also referred to as the Darcy velocity. Moreover, the average linear flux or pore velocity is a function of porosity. Therefore, for a given porosity n, the average liner pore velocity is q/n. From the above equation, the flux vector q has the components q_x, q_y, q_z, and the gradient vector is composed of $\partial h/\partial x$, $\partial h/\partial y$, $\partial h/\partial z$, and the above equation can also be written in the form:

$$q = -K\nabla h \qquad (3.4)$$

Where the $-\nabla h$ term represents the gradient vector. In an isotropic medium, both the flux and gradient vectors point in the same direction.

3.2.2 Hvorslev's theory of Basic Time Lag

Hydrostatic time lag is based on the validity of the falling head permeameter, the assumption that Darcy's Law is valid, and that the soil and water are incompressible. Hvorslev (1951) conducted studies measuring hydrostatic pressure differences and hydraulic permeabilities in boreholes, wells, and piezometers for a number of in situ configurations. Figure 3.1 demonstrates the basic premise of the basic hydrostatic lag time used to calculate in situ hydraulic characteristics.

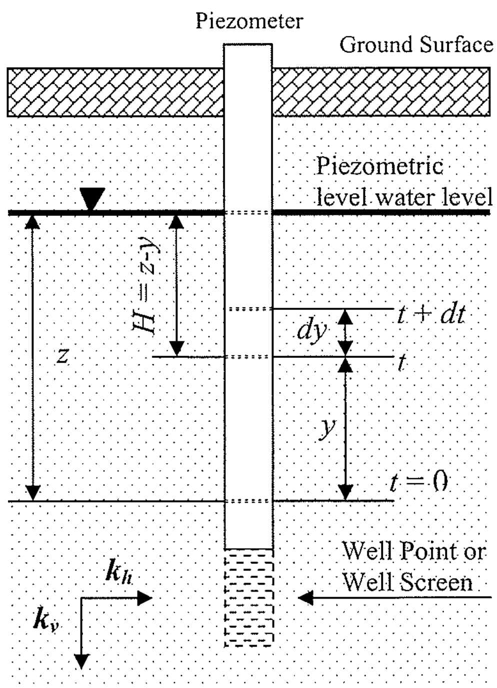

Figure 3.1 Piezometer installed below the watertable for determining the in situ hydraulic conductivity (Adapted from Hvorslev (1951), U.S. Corps of Engineers, W.E.S.)

From Figure 3.1, Hvorslev (1951) developed the differential equations for flow into the piezometer. Hvorslev (1951) expressed the relationship for flow into the piezometer from the relation:

$$Q = FKH = FK(z - y) \tag{3.5}$$

where:

Q	=	Volumetric flow rate, (m^3s^{-1}),
F	=	Shape factor, which is a function of the well point intake geometry, (m),
K	=	Hydraulic conductivity coefficient, (ms^{-1}),
H	=	Elevation head where $H = z - y$ (m).

Hvorslev assumes that friction losses at the inlet are negligible during low flow rates. For a standpipe or piezometer as seen in Figure 3.1, the general equation for volumetric flow for a period, *dt,* is:

$$Q\,dt = A\,dy \tag{3.6}$$

where:

A	=	Inside cross-sectional area of the pipe, (m^2),
dy	=	Change in head from time t to time $t + dt$, (m),
dt	=	Period, (s).

by combining the two previous equations and assuming constant water table, the pressure head changes in the standpipe and the volumetric flow in the period *dt* are represented by:

$$\frac{dy}{z-y} = \frac{FK}{A} dt \tag{3.7}$$

A certain period is required for a given volume of flow, V, to fill or equalize the pressure from an instantaneous head difference initiated at t = 0. Hvorslev (1951) describes this equalization process the *Basic Time Lag*, T_L, required for a zero pressure difference given the initial volumetric flow rate (at time t = 0) is maintained. In other words, $Q = FKH$ during the entire equalization process, and is consequently:

$$T_L = \frac{V}{Q} = \frac{A}{FK} \tag{3.8}$$

To simplify equation 3.5, the differential equation for basic hydrostatic time lag determination is:

$$\frac{dy}{H} = \frac{dt}{T_L} \tag{3.9}$$

To determine T_L in the field, a relationship is plotted with the pressure head ratio on the log scale and time on the linear scale, and is given by the resulting relationship in Figure 3.2. For constant groundwater pressure, the T_L is the time at which the head ratio H/H_0 is equal to 0.37.

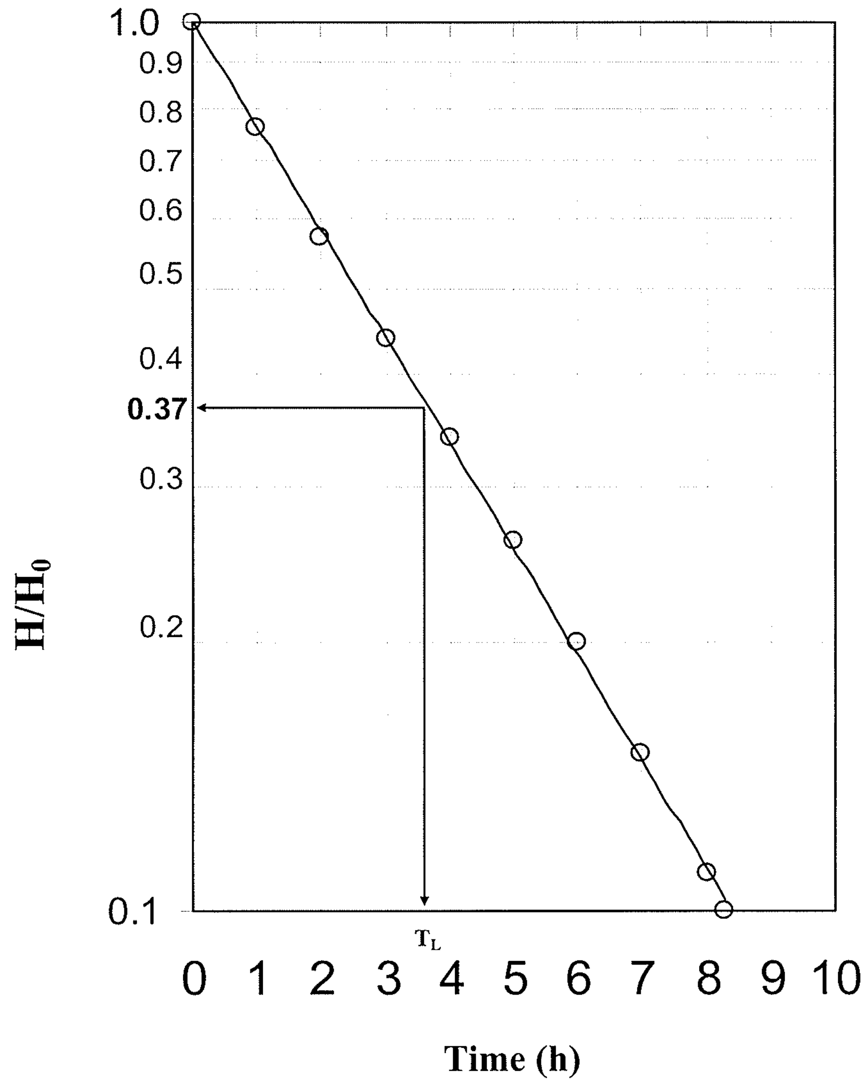

Figure 3.2 Plot of H/H_0 versus time for the determination of the Basic Time Lag, T_L, in Hvorslev's method. The Basic Time Lag occurs when the head ratio $H/H_0 = 0.37$.

The solution to equations (3.7), (3.8) and (3.9) when combined forms the time lag ratio equal to 1; given by:

$$\frac{t}{T_L} = ln\frac{H_0}{0.37H_0} = ln2.7 = 1 \qquad \textbf{(3.10)}$$

Hvorslev (1951) refers to the equalization ratio as "1 minus the head ratio". Therefore, for a head ratio of 0.37 the equalization ratio is 0.63 and is referred to as T_{63}. For example, an equalization ratio of 0.99 requires twice as much time as a ratio of 0.90. If a shape factor is known, then the hydraulic conductivity is estimated from the T_L and by rearranging equation (3.8) to yield:

$$K = \frac{A}{FT_L} \qquad \textbf{(3.11)}$$

If a shape factor is not known, then the area of hydraulic transfer and the shape factor can be empirically related by a constant M, as the ratio of A and F in equation 3.11. To determine the constant, M, empirically, the hydraulic conductivity needs to be determined by another method, such as a laboratory method (K_L). After K_L is obtained and T_L is determined from field measurements, the constant is given by:

$$M = K_L T_L \qquad \textbf{(3.12)}$$

where:

M = Constant, the ratio of A/F, (L).

Once the constant is determined, the field hydraulic conductivity, K_F, is estimated from the calibrated constant and the basic lag time obtained at other field sites using the relationship:

$$K_F = \frac{M}{T_L} \tag{3.13}$$

3.2.3 Electrokinetic phenomenon

When a voltage potential is applied across saturated clayey soil, water will generally migrate in the direction of the cathode. The process by which water moves under an electrical potential gradient is known as electro-osmosis. Electrokinetics is a term used to combine the coupled fluid flow and electrical current flow (Laursen, 1997). Researchers Mitchell (1991), Yin *et al.* (1996), Finno *et al.* (1996), and Laursen (1997), have observed and modeled this electrokinetic phenomenon on cohesive soils. Once again from Darcy's law, the hydraulic flux in one dimension is:

$$q_{h_x} = K_{h_x} i_{h_x} \tag{3.8}$$

Similarly, the electro-osmotic flux in one dimension is:

$$q_{e_x} = K_{e_x} i_{e_x} \tag{3.9}$$

where:

q_{h_x} = Hydraulic flux (ms^{-1}),

q_{e_x} = Electro-osmotic flux (ms^{-1}),

K_{h_x} = Hydraulic conductivity coefficient (ms^{-1}),

K_{e_x} = Electro-osmotic conductivity coefficient ($m^2V^{-1}s^{-1}$),

i_{h_x} = Hydraulic gradient (mm^{-1}),

i_{e_x} = Electrical potential gradient (Vm^{-1}).

The negative sign indicates flow in the direction of decreasing potential. Mitchell (1993), states that the coefficient of electro-osmotic conductivity range has a very narrow band from about $1x10^{-9}$ $m^2V^{-1}sec^{-1}$ to $1x10^{-8}$ $m^2V^{-1}sec^{-1}$ compared to the coefficient of hydraulic conductivity, which can vary by several orders of magnitude. By equating equations 3.8 and 3.9 we could determine the hydraulic gradient imposed by a given electrical potential across a given length of soil. For instance, if a clayey soil possessed the following properties:

Hydraulic conductivity = $1x10^{-9}$ ms^{-1}

Electro-osmotic conductivity = $1x10^{-9}$ $m^2V^{-1}s^{-1}$

If an electrical potential of 12 V DC was applied across 0.05 m of the clayey soil, then hydraulic gradient in one dimension would be:

$$i_{h_x} = \frac{k_{e_x} i_{e_x}}{k_{h_x}} = \frac{1 \times 10^{-9} \times \dfrac{12}{0.05}}{1 \times 10^{-9}} = 240$$

From this basic equation it is evident that when a low electrical potential is applied across a short distance of clay, it can produce a significant gradient, which can be utilized to measure the increase in resultant pore pressures. Mitchell (1993) describes the electro-osmotic processes and other conduction phenomenon such as coupled flows in great detail.

3.3 K-Probe module design

The K-Probe is designed from mostly commercially available components and materials. Figure 3.3 illustrates the schematic diagram of the disassembled and assembled K-Probe module. Appendix A contains the fabrication schematics for the K-probe module. The outer casing is made of a 0.025 m outside diameter aluminium pipe. The satin coating of the outer pipe was mechanically brushed off at the lower 0.25 m in order to establish a region of good electrical contact for the electro-osmotic process. The aluminium pipe served as the anode in the electro-osmotic process. At the brushed end of the pipe, a nylon sleeve was machined to fit inside the aluminium pipe that supported a 0.18 m long by 0.019 m diameter stainless steel water proof pressure transducer. This nylon sleeve was attached and held in place by three metal set screws. The nylon sleeve contained a cylindrical cavity at one end, which held the pressure sensing diaphragm in place. The transducer tip was protected by a stainless steel concave mesh.

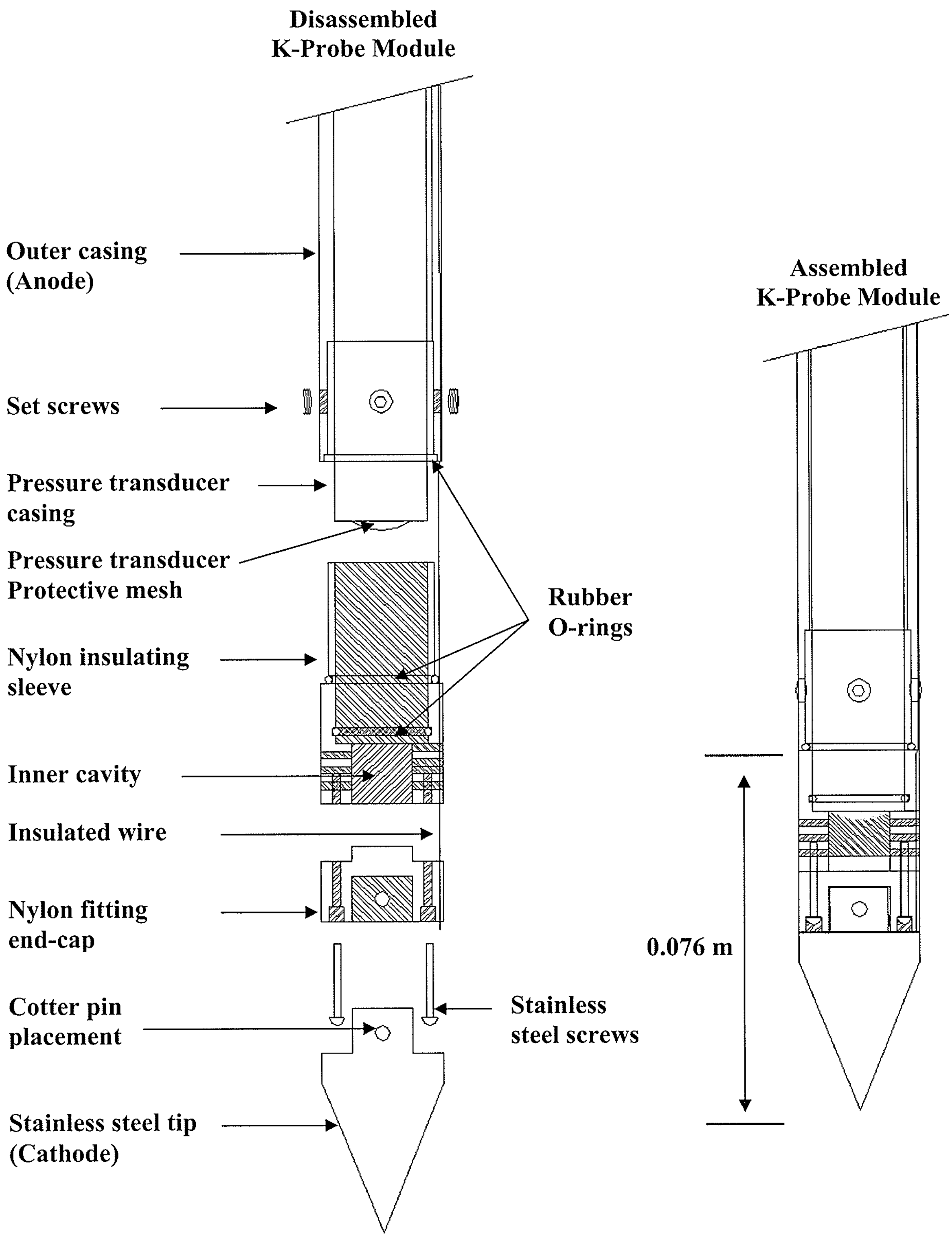

Figure 3.3 Schematic diagram of the K-Probe module.

The inner cavity contained 0.0016 m holes that extended radially outward for fluid passage. The nylon sleeve was also connected to a second nylon fitting end cap that held a stainless steel tip attached by a perpendicular cotter pin. During the electro-osmotic process, the stainless steel tip acted as the cathode via a thin 0.00025 m insulated wire that electrically connected the cathode tip to the surface of a 12 V battery source. A K-type thermocouple was connected 0.20 m above the pipe tip to measure pore fluid temperature near the tip during testing.

3.4 Instrumentation

3.4.1 Pressure transducer

The pressure transducer used in the K-Probe module was a submersible WL300 water level sensor produced by Global Water Instrumentation, Inc. The sensing element was a silicon diaphragm capable of measuring pressure from 0 to 4.57 m of water head. The excitation voltage was 10 to 36 V DC or 0.5 to 2.5 V across a 125 Ω resistor. The transducer had a linear full scale output ranging from 4 to 20 mA. A 7.62 m long waterproof polyethylene cable was attached to a 0.019 m diameter stainless steel housing cylinder, which was 0.20 m long. The cable contains a poly tube used to vent the transducer. The linearity and hysteresis was ± 0.1 full-scale and had an overall accuracy of ± 0.2% ranging from 1.6 to 23.9 C.

3.4.2 Data logging

A Fluke digital multimeter Model 890 with data logging capability was connected to the pressure transducer in series by connecting to a main terminal box with banana type connecters. Only one test could be recorded prior to the data being downloaded to a PC laptop using Fluke Forms^{TM} software via an optical-serial port interface.

3.5 Probe pressure transducer calibration

A pressure transducer calibration was conducted to form a linear relationship for output current generated by the transducer and the equivalent hydraulic gauge pressure in cm of water. This relationship was determined by constructing a cylindrical acrylic pressure vessel which was filled with water. At one end of the vessel, a nylon fitting was attached to 0.006 m clear Tygon^{TM} tubing and extended vertically up a metric tape measure. On the other end of the pressure vessel, a round hole was machined and square-grooved to allow two rubber O-rings to seal the probe after insertion. The tubing was connected to a reservoir (glass burette) filled with tap water and blue dye. The blue dye simplified observing the meniscus in the transparent tubing. A valve near the bottom allowed the water to fall, thus decreasing the pressure head in the vessel. The water was allowed to fall from 5 to 0 m by 0.1 m increments. The output from the pressure transducer was recorded in mA. Three tests were conducted to establish a linear calibration curve to convert the output current from the transducer to equivalent pressure in cm of water. In Appendix C, the relationships determined are provided by three figures representing the calibration trials. A table in Appendix C represents the summary of linear regression slopes and intercepts obtained from each trial. The slope and intercept averages were used to convert the output current in mA, to pressure in cm of water.

4.0 K-PROBE IN SITU HYDRAULIC CONDUCTIVITY TESTING

4.1 Introduction

The K-probe was devised using electro-osmosis to induce flow of water towards the cathode tip. An interface was developed to capture in situ pore water pressure dissipation rates after the applied current was removed. The probe was tested in a water retention pond having a clay bottom at depths ranging from 0.23 m to 0.50 m at 8 locations. The results were then compared to results from undisturbed cores that were removed from the same locations and tested in the laboratory.

4.2 History and testing site background

The testing site was located L.S.D. 5, section 10 – 8 – 3 east of the prime meridian approximately 10 minutes from Winnipeg south on Highway 75, in the rural municipality of Gleanlea, Manitoba, Canada. Figure 4.1 below illustrates the testing site location and Appendix B provides aerial photos of the testing site provided by the Manitoba Soil Survey in 1975. The testing site was previously a water retention pond used to water the livestock at the research station. The pond is no longer used. Water was pumped into the pond from the adjacent Red River located approximately 50 m to the south of the pond. The pond consists of a clay bottom with an overlying organic black mat that varies in thickness across the pond. At the time of testing, the height of the surrounding berm was approximately 2 m above the ice surface at the end of March 2003. The pond has a diamond or parallelogram shape with different side lengths. Figure 4.2 is an illustration of the pond with the location of testing holes A to H. This

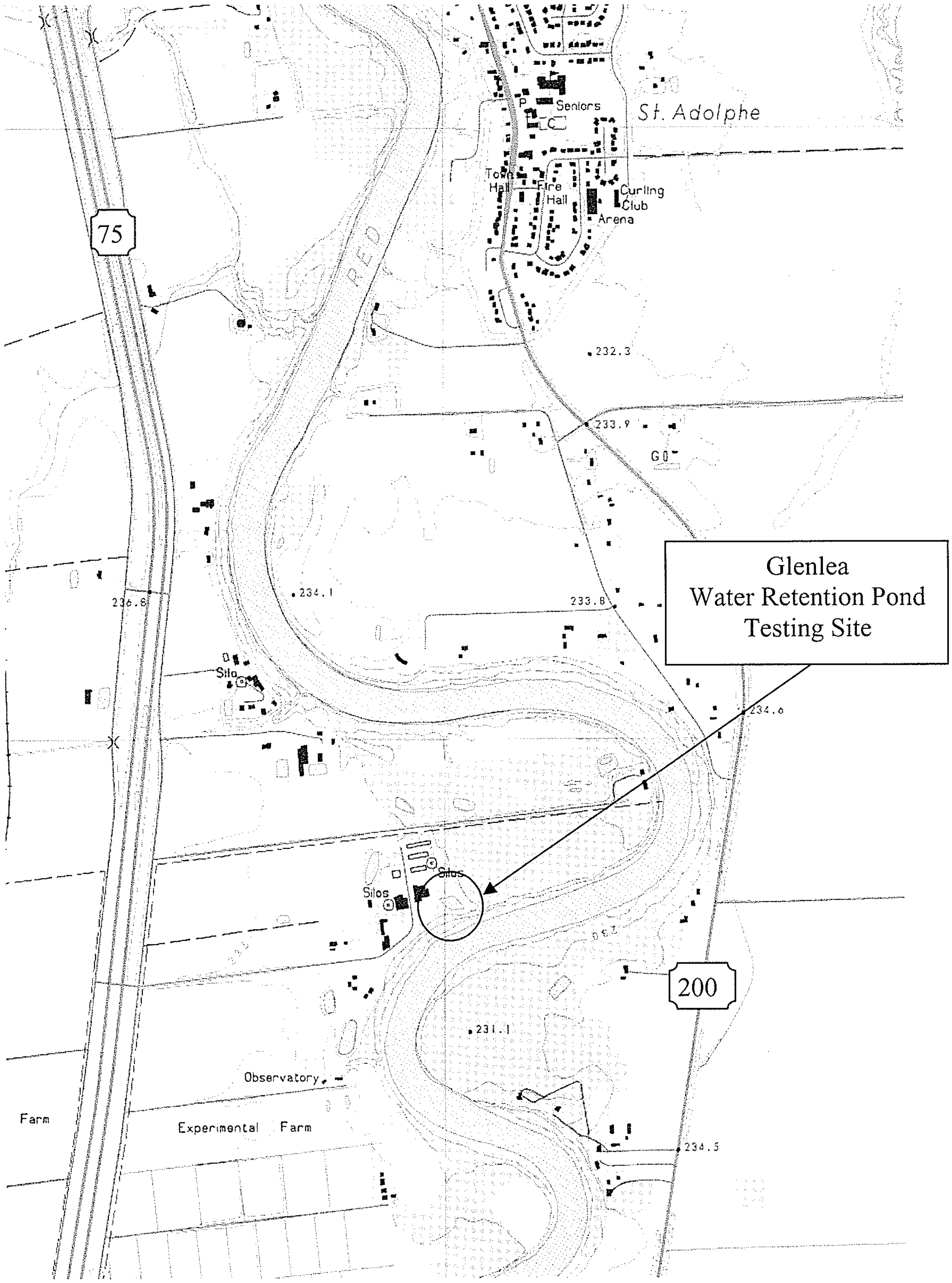

Figure 4.1 Adapted from the Manitoba Natural Resources Map A136, scale 1:20000 Produced by the Province of Manitoba, Surveys and Mapping Branch, (1990).

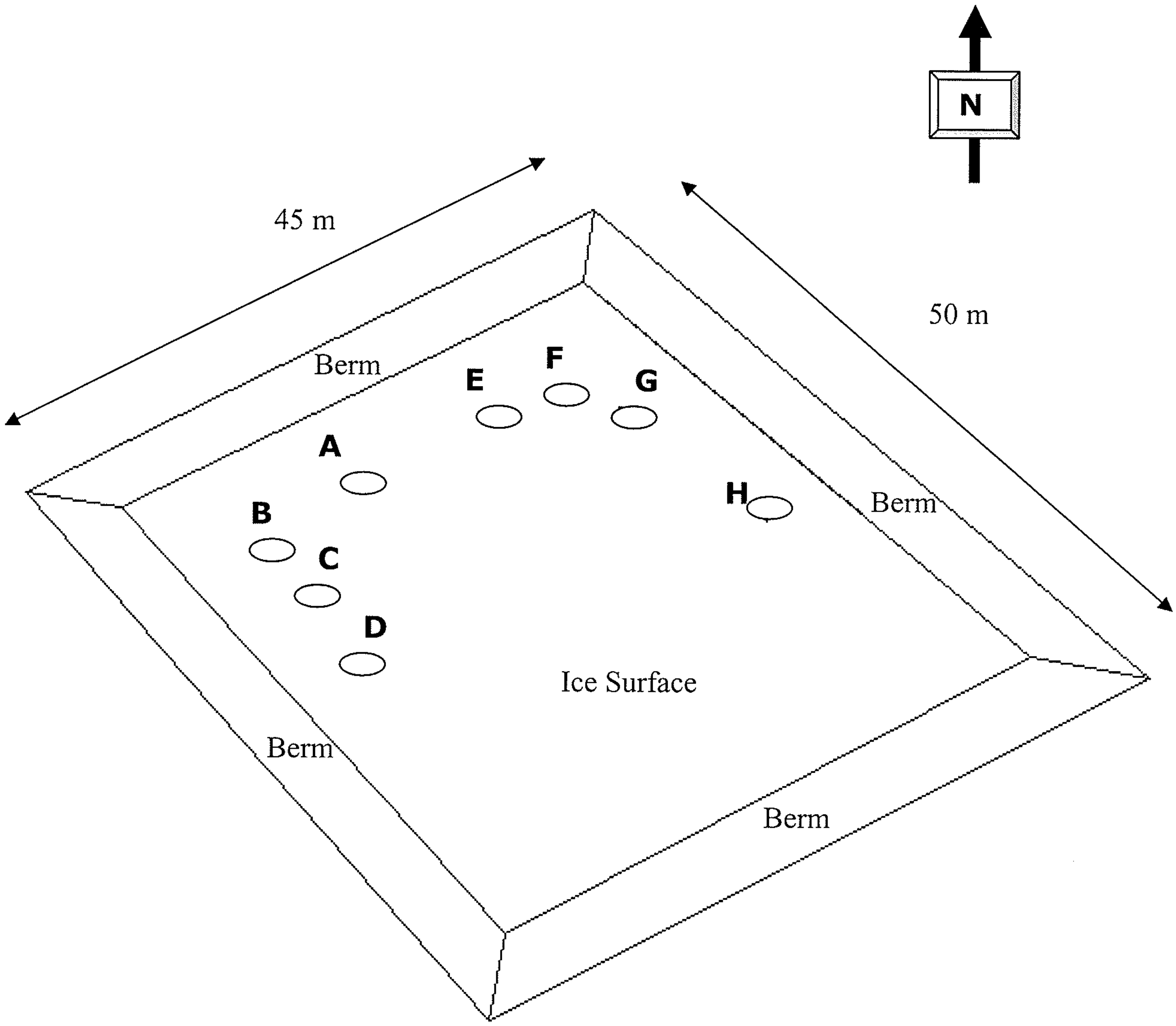

Figure 4.2 Glenlea Research Station water retention pond and testing locations A to H.

water retention pond was chosen as a testing site for its similarity in design (dug-out) of old swine manure storages, which did not have a compacted clay liner. A non-compacted clay liner provides a homogenous soil distribution, making it easier to test and calibrate the K-Probe. The pond also contains an organic mat upper layer, and thick clay construction materials similar to most earthen manure storages. Other similar reasons include: accessibility (short hauling distances for equipment and samples), favourable soil sampling conditions (thick ice for safety). Hog manure storages in the area had already begun to melt at the time of testing which made testing procedures unsafe.

4.3 Gleanlea testing area soil type and geology

The system of soil classification for Canada in 1970 consisted of the order, the great group, the subgroup, the series name, and the type. The soil represented at the Gleanlea site is: Chernozemic, dark grey, Orthic Dark Grey, St. Norbert, Clay (Manitoba Soil Survey, 1975 and Agriculture and Agri-food Canada, 1999). Table 4.1 describes the basic physical and chemical characteristics of the St. Norbert Clay.

Table 4.1 Analysis of St. Norbert Clay adapted from Manitoba Soil Survey (1975).

Horizon	Depth cm	Paricle Size %			Conductivity dsm^{-1}	C.E.C. m.e./100 g
		Sand	Silt	Clay		
Ahe	0-8	6	46	48	-	37.5
Btj	8-25	7	22	71	-	43.0
Bm	25-45	4	20	76	-	49.1
BCk	45-60	6	17	77	0.4	-
Ckg1	60-90	3	16	81	1.6	44.5
Ckg1	90-120	3	8	89	1.1	-
Ckg	120-180	7	19	74	1.1	29.4

The Glenlea testing area is comprised of Lacustrine and Alluvial surface deposits consisting of about 16 m of clay and 7.5 m of till underlain by the Red River Formation (Manitoba Natural Resources 1980). The formation is a Selkirk member consisting of dolomitic limestone and dolomite at an elevation of approximately 215 m above mean sea level (McFarleane and Lenton, 2002).

4.4 K-Probe testing procedures for measuring in situ hydraulic conductivity

4.4.1 Preliminary testing procedures

The testing procedures used in the water retention pond have been developed based on preliminary tests conducted in the summer of 2002 at both the Seine River and Red River located in Winnipeg, Manitoba. These sites were utilized as initial study areas for the naturally saturated clay conditions that exist near the sediment surface. After preliminary results from testing were evaluated, a similar procedure was conducted at the Gleanlea testing area.

4.4.2 K-Probe test locations

A number of holes were drilled through the ice at locations approximately 6 to 8 m from the edge of the pond, where water and ice depth did not exceed the probe length of 2.5 m. After drilling the holes with the ice auger, the water depth was checked using a measuring tape to gauge the sampling depth. The locations for core sampling sites were not completely random throughout the pond, but rather the sampling depth (distance from the ice surface to the pond sediment surface) governed the potential sampling locations. Testing locations near the pond center were in excess of 5 m, which were inaccessible by the K-Probe. Testing location holes were randomly drilled 7 to 20 m apart on the north-west and north-east and southwest sides, to test different areas of the retention pond in a water-ice level depth that ranged from 1.6 to 2.35 m. The randomization process included a 2m wide grid system approximately 6 m to 8m from shore. Table B of random digits in Moore (1995) provided the random values representing distances between holes and the distance perpendicular to the shore line from 6 to 8 m. The locations closest to the shore were not chosen as testing sites in order to maintain a positive pressure head of at least 1.6 m, which would promote naturally saturated clay conditions throughout the year.

4.4.3 K-Probe testing procedure

The probe was connected in series to a 12 V DC power supply, a terminal box, a data logging multimeter, and a laptop computer as illustrated in Figure 4.3. The multimeter reading was approximately 4 mA when exposed to atmospheric pressure. The Fluke Forms Software™ was initiated on the laptop computer to observe the output current in mA as a function of time (standard central time). Graphically, the Fluke Multimeter was programmed to take a reading every 5 s. This increment was sufficient based on previous experimentation. One second increments were not necessary and would produce a large number of data points during the slowly dissipating pore pressures.

Before the K-probe was immersed through the ice and into the clay sediment, a syringe was filled with water obtained from the pond, and injected slowly into the small cavity where the pressure transducer was located. This procedure ensured the removal

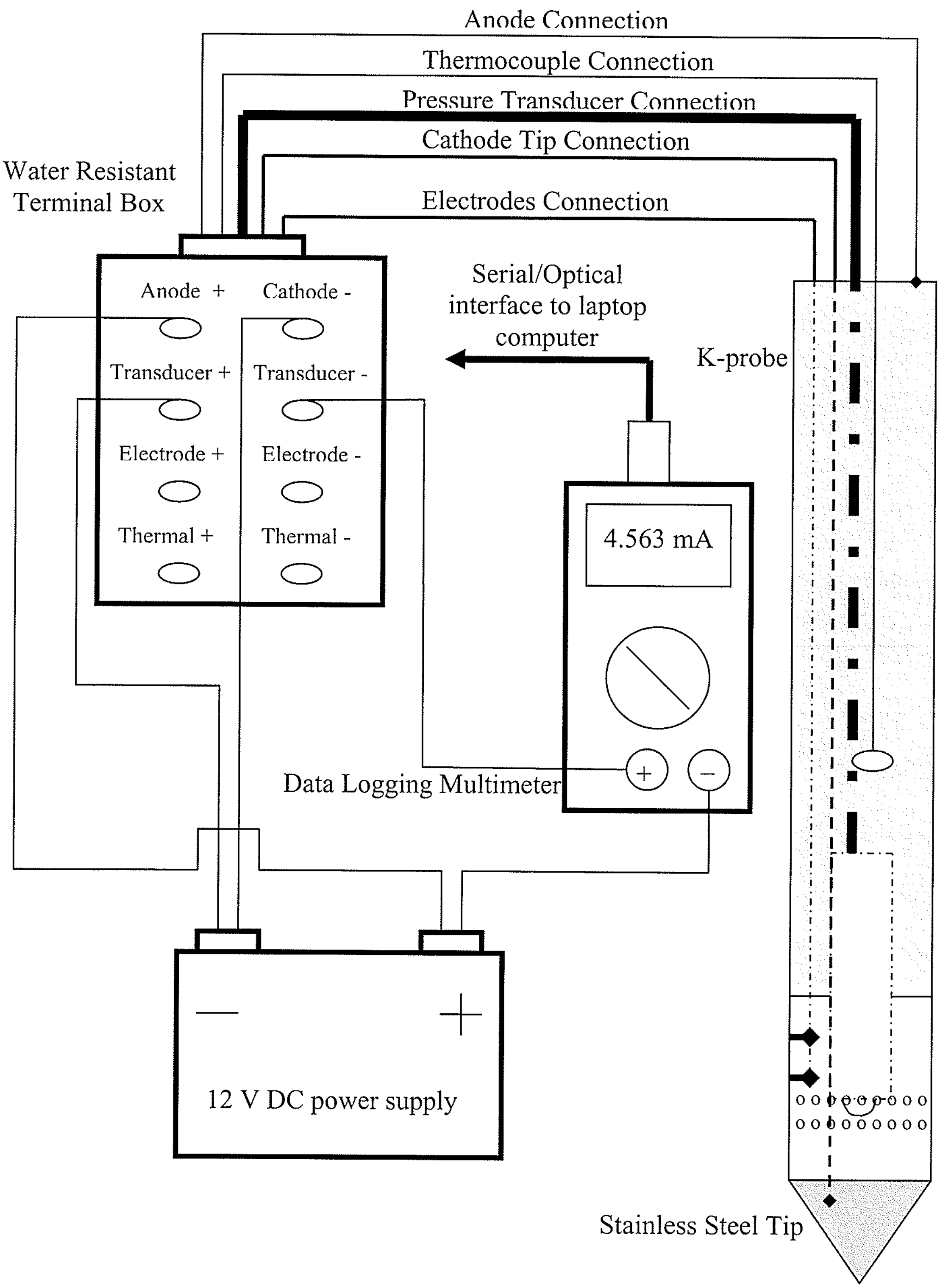

Figure 4.3 Schematic of the K-Probe electrical connections used in the field.

of air from the K-Probe module cavity to minimize the effects of air bubbles on the pressure readings. The multimeter data logging unit was initiated and the probe was then inserted into the water and agitated in order to remove any small air bubbles that may have been present near the pressure transducer. Upon probe insertion into the sediment layers, a depth measurement was made to correspond with an adjacent core sample removed previously. During the testing procedure, output pressure was graphically displayed on the laptop. This feature was important to identify irregular pressure patterns developing as the probe returned to equilibrium with the surrounding pore pressure.

4.5 Permeability data analysis

4.5.1 K-Probe pressure readings

The data collected in the field was logged and stored in a digital multimeter in the form of output current with respect to time (Figure 4.4). Using a serial-optical interface to a laptop computer, Fluke Forms Software was used to download the data from the multimeter. The data was then exported as a ".csv" file and imported into a spread sheet application program for analysis.

The data was initially plotted to visually describe the different stages of each in situ test. There were certain critical points in the curve seen in Figure 4.4 in the output data. Point (1) is the output current from the pressure transducer under atmospheric pressure. At point (2), the output has increased as the probe has been inserted into the water and into the soil. After a period of constant pore pressure, approximately 10.5 V was applied at point (3) for approximately 20 to 60 s. The period of current application

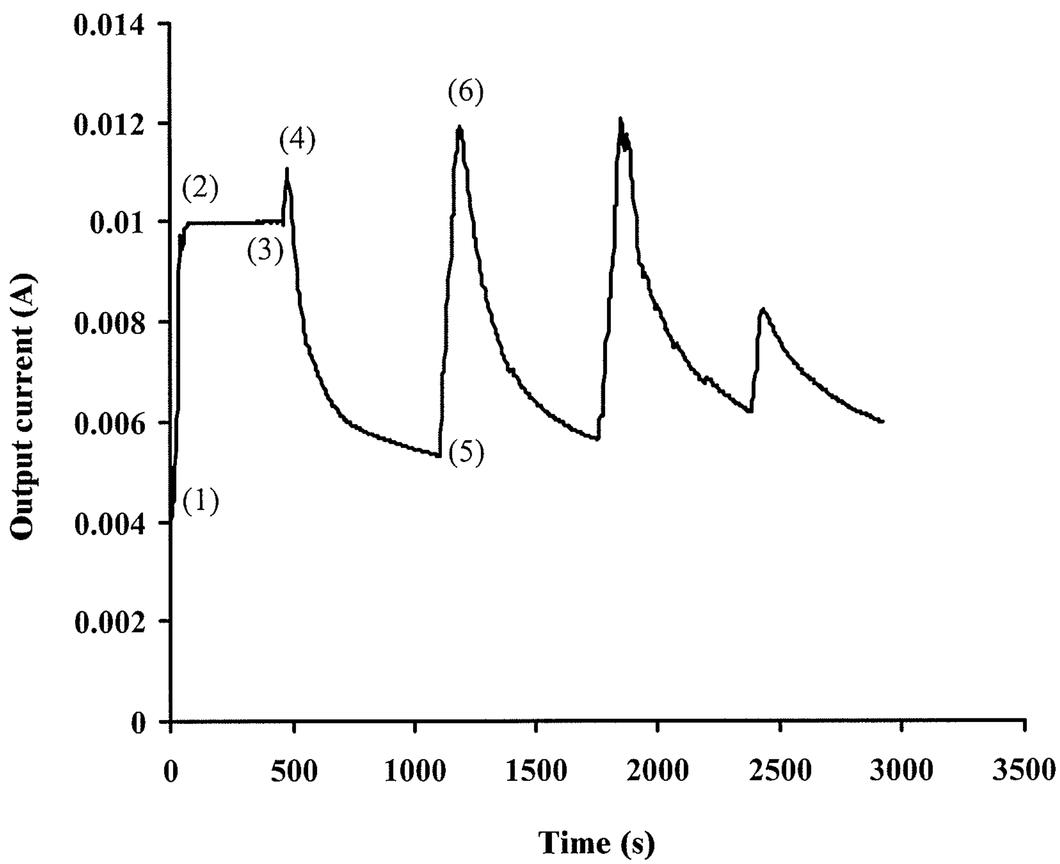

Figure 4.4 Output current logged over time from in situ hydraulic conductivity tests using the K-Probe. Initial output current prior to submergence, point (1). Output current after insertion into soil, point (2). Output current prior to application of 10.5 V, point (3). Output current after application of 10.5 V for 20 to 60 s, point (4). Output current after pore pressure dissipation rate was relatively constant, point (5). Output current after application of 10.5 V for 20 to 60 s, point (6).

depended on the rate of pore pressure increase. If the output from the transducer reached a maximum stabilized reading after 20 s at point (4), then the current was terminated and the output was allowed to drop. When the dissipating pore pressure rate fell to a pressure that became relatively constant with time, point (5), another current application was administered for 20 to 60s. The pressure then rose to point (6). If the output current from the probe did not increase after an application of 60 s, then the test was aborted. The probe was either moved to a new depth or a new location within the test hole. The electrokinetic impulses causing the adjacent pore pressures to dramatically increase near the cathode are illustrated by points (3) to (4) and (5) to (6). Only the falling data was used from the pore pressure readings after the current was removed. Using the K-Probe calibration curve, the output current logged in the field was converted into pressure head by the following linear relationship:

$$\textit{Water head in cm of water} = (\textit{output current } A) * (1000) * (28.21) - 114.07 \qquad \textbf{(4.1)}$$

The data was then reformatted with respect to the lowest pressure logged at the end of each trial, which represented the datum or zero gauge pressure. For each falling curve, or pressure dissipation test, a new max pressure logged (H_0) and head at time t_i (H) was calculated. This data formatting assigned the pressure differences as a function of time to establish a parallel analysis to Hvorslev's initial and final hydraulic heads H_0 and H described previously. The initial head, H_0, did not necessarily come from the first reading after the current was removed due to an observed uncharacteristic pressure drop immediately following pressure removal. This pressure drop can be described as a type of pressure rebound from the sudden removal of the electro-osmotic effects. This drop did not appear to be characteristic of natural pressure dissipation relating to the porous media. Therefore, double differentiating the decreasing water head over 5 s intervals was used as a data filter consisting of a determined upper and lower boundary. The upper and lower boundaries were chosen as:

$$\frac{d^2H}{dt^2} = \pm\, 0.002\ ms^{-2} \tag{4.2}$$

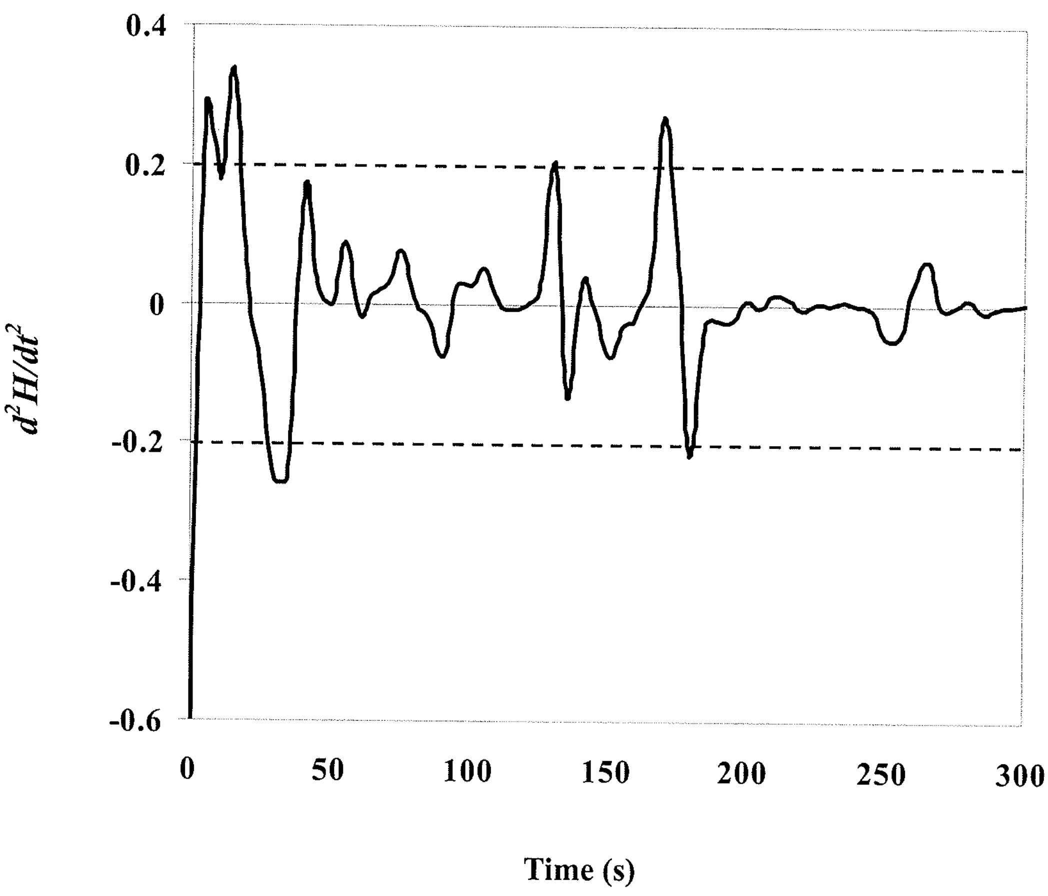

Figure 4.5 Double differentiation over 5 s intervals for pressure dissipation defines the set upper and lower boundaries of 0.002 ms^{-2} and -0.002 ms^{-2} respectively for filtering initial excessive pressure dissipation values.

The first value from the pressure data that fit into this upper and lower range was assigned H_0. All remaining data that fit in this range were used to calculate the hydraulic conductivity. The last value of the falling curve was assigned as the datum of lowest pressure, H. If the values were greater than 0.002 m, then the effect of unnatural dissipation pore pressures affect the falling curve slope, thus yielding higher hydraulic conductivities. If the boundaries were set less than ± 0.002 m, then the square of the correlation, r^2, decreased when a log-normal relationship (Figure 4.6) was generated for H/H_0 as a function of time. A second type of data filtering consisted of removing points that could have been caused by environmental factors such as momentary pressure pulses. These pulses could have been caused from a number of reasons such as wind moving the probe while testing on the ice surface, bubbles from electrolysis reactions, accidentally bumping the probe wire during testing or other experimental errors. However, if the data did not follow a similar pattern following the pulse, the test was abandoned and another probe location was tested. Figure 4.6 is defined by setting the intercept to 1.0; in theory the line should always start here at point $(0, H_0/H_0)$ or (0, 1). After the points are plotted, an exponential regression line is in the form of:

$$\ln(H_t/H_0) = mT \tag{4.3}$$

where:

H_0	=	Initial pressure head prior to removal of the current
H_t	=	Pressure head after some period of dissipation.
m	=	Slope, $d(H/H_0)/dt$, $(mm^{-1}s^{-1})$,
T	=	Time, (s).

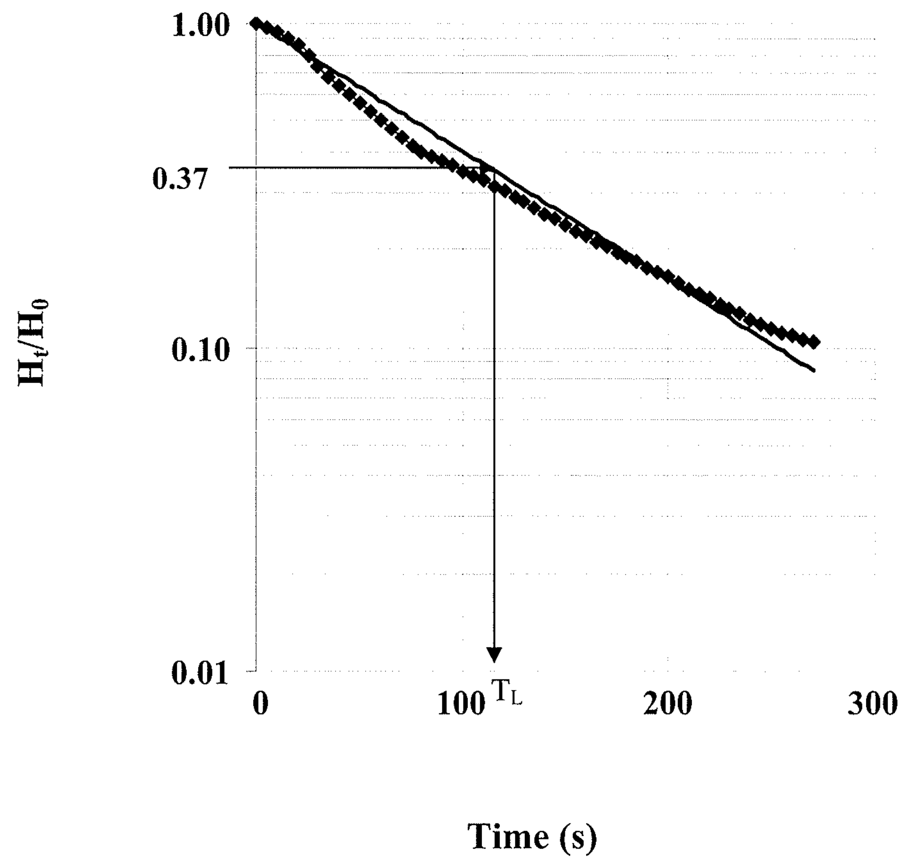

Figure 4.6 A natural log-normal pressure relationship is given with time as a function of initial pressure immediately after removing the 10.5 V DC. An exponential fitted line through the data values is used to establish the lag time, T_L, when the ratio $H_t/H_0 = 0.37$.

For all the K-Probe measurement data collected in the field, the relative pressure dissipation curves were plotted until the ratio H/H_0 was equal to 0.001 mm^{-1}. This was done to neglect the low pressure data that caused the curve to start rapidly decreasing. It was observed that at low gradients, when $H/H_0 < 0.001\ mm^{-1}$, a linear pressure dissipation curve was more representative of the data rather than an exponential pressure relationship seen in Hvorslev's analysis. However, if the data produced an r^2 less than 0.95, then the data was plotted to H/H_0 equal to 0.37. This was a third type of data filtering. The time lag T_{63} was determined for each trial by rearranging the relationship illustrated in Figure 4.6; when H/H_0 is 0.37. The subscript 63 represents the time of pressure dissipation that has fallen 63 percent of H_0. In Figure 4.6, the slope, of the exponential regression line is m = -0.0091 and the r^2 value is 0.977. The resulting time lag, T_{63}, can be determined from Equation 4.3:

$$T_L = \ln(H_t/H_0) / m$$

$$T_{63} = \ln(0.37) / (-0.0091)$$

$$T_{63} = 109\ s$$

Using the time lag variable T_{63}, The hydraulic conductivity can be determined by dividing the probe constant (M) by the T_{63}. From this relation, the raw data collected in the field is used to determine the hydraulic conductivity at a particular point within the EMS clay bottom.

5.0 IN SITU SOIL CORE EXTRACTION AND LABORATORY TESTS

5.1 Soil Core sample collection

5.1.1 Core sampling and collection instrumentation

American Standards for Testing of Materials (ASTM) documents D 1587 - 94, ASTM (2000), D 3550 - 84 (Re-approved 1995) ASTM (2000), D 4220 - 95 ASTM (2000), and D 6282 - 98 ASTM (2000), were used as core sampling guidelines. A 0.20 m diameter ice auger was used to drill holes through the ice surface through which the soil cores could be extracted. The core sampling tool was designed and fabricated at the University of Manitoba in the Department of Biosystems Engineering in 1996. The sampler consisted of an outer black iron pipe, a machined steel drive point shoe, steel couplings, a steel top impact shank, an inner tube sleeve core sampler, a plastic tube spacer, a metal adjustable clamp, and an adjustable steel pin. The black iron pipe consisted of 1.650 m sections with an outer diameter of 0.061 m, which was used as an outer driving tool. The tip of the pipe had a removable drive point shoe, which had an inside diameter of 0.040 m, slightly smaller than the inside diameter of the inner tube sleeve, of 0.045 m. The coring pipe had the capability of extending using a series of 0.055 m long threaded metal couplings. Attached to the top of the 1.65 m pipe, was a similar 0.350 m pipe extension that had a steel top impact shank, which mates with the bit of a jackhammer. The solid metal impact shank transferred the load from the jackhammer into the black pipe extension and into the underlying soil. The tube type inner sleeve consisted of a number of smaller acrylic cylinders which were 0.077 m and 0.052 m in length. The inside and outside diameters were 0.045 m and 0.052 m respectively. Each sleeve was individually cut from an extruded acrylic tube and machined to ensure smooth edges. The smooth edges were necessary for the lab testing portion of individual core sections. After individually labelling the sleeves, a 0.800 m tube was formed by assembling the cylinders and taping them together with plastic packing tape. A spacing pipe made of black 0.050 m O.D. water pipe was used to keep the tube in place against the coring tip when the coring tool was inserted into the soil. The coring tool was impacted using a 16 kg Hitachi jackhammer to a predetermined depth or until refusal. The coring tool was removed by using a 1 m long sliding jack. The supply power for the jackhammer came from a small gas powered Coleman electric generator. The jack was then forced up against a special adjustable angle iron clamp that was attached to the black coring pipe. The clamp featured adjustable slots for a 0.016 m steel pin, which projected perpendicular to the coring tool to supply a loading point.

5.1.2 Core sampling procedures

Eight cores were extracted from the eight locations in the Glenlea water retention pond in four days. The cores removed from the testing site did not exceed sampling depths of 3 m from the ice surface.

By lowering the coring tool slowly through the pre-drilled hole in the ice, the surface of the soil was detected. A measurement was taken on the coring pipe from the ice surface to make the elevation on the pipe. The tool was then lowered down until body weight could not penetrate the soil bottom any further. The jackhammer was then placed into position at the top impact shank. Hammering took place until a core sample length of 0.800 m could be removed. At this point, the jackhammer was removed and the sliding jack was moved into position against the steel pin inserted into the adjustable clamp. When little effort was needed to raise the coring tool with the jack, the tool was manually lifted out of hole. The tapered steel coring tip was then removed and the core sample was extracted. If there was excess core left in the black outer pipe section after removal of the acrylic tube, the remaining length was recorded for future reference. The overall length was recorded and any visual description logged. Using plastic wrap, both tubes ends were taped to prevent core loss, de-saturation, and gas exchange. The tubes were stored in a plastic container and submerged horizontally in pond water. They remained submerged until individual cores were prepared and tested in the lab.

5.2 Falling head method

The method for this procedure is described by the ASTM D 5084 – 90, however this test has been modified. The flexible wall permeameter has been adapted to allow the application of a uni-axial effective stress of 35 KPa using an oedometer consolidation cell apparatus. Figure 5.1 illustrates the apparatus used to measure undisturbed soil core samples obtained from the field.

A similar setup is described in Head (1982), for falling head permeability test in oedometer consolidation cell. This setup was used to effectively produce enough backpressure to the specimen to prevent bypass flow caused by macropores adjacent to the cylinder walls. The maximum ASTM recommended gradient for samples, which possess a K_s of less than $1x10^{-9}$ ms^{-1} is 30. The maximum gradient applied to the

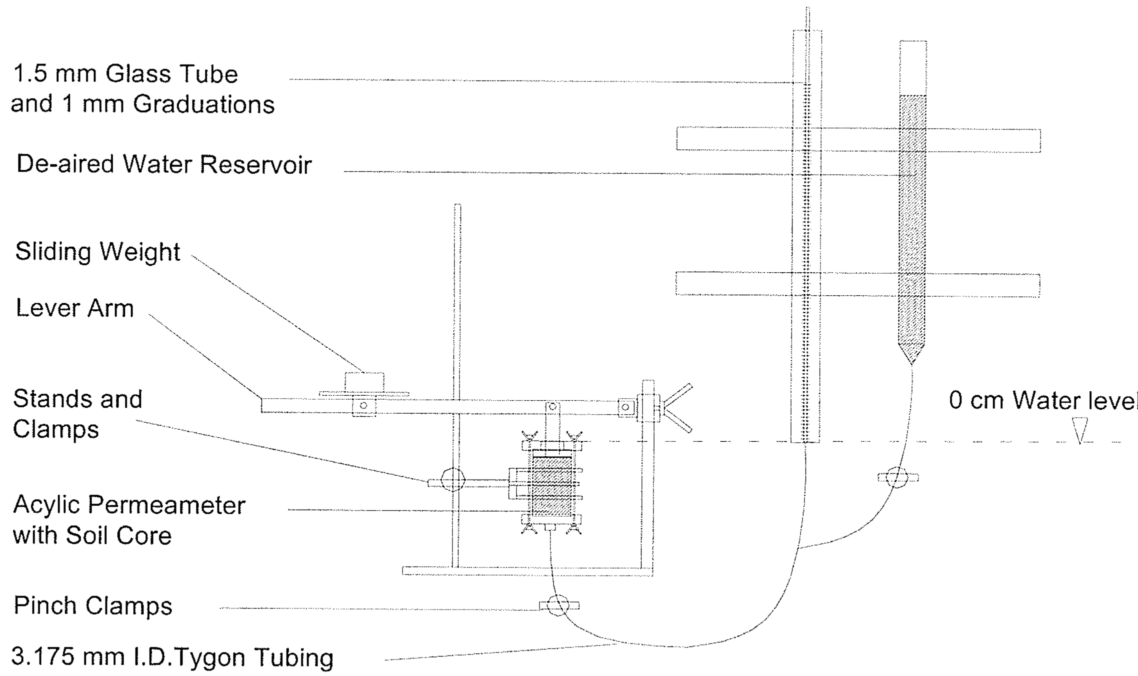

Figure 5.1 Diagram of the lever arm oedometer permeability apparatus.

samples ranged from 16 to 30. The gradient in this process is the ratio of the applied water head, m, to the length of specimen. However, it should be noted, that increasing the gradient, increases sample consolidation. Therefore the upper sedimentary layers, mostly comprising of black organic matter, were consolidated to some extent and actually decreased the K_F. The core samples were cut along the taped acrylic tube divisions that held the intact core in place (Figure 5.2).

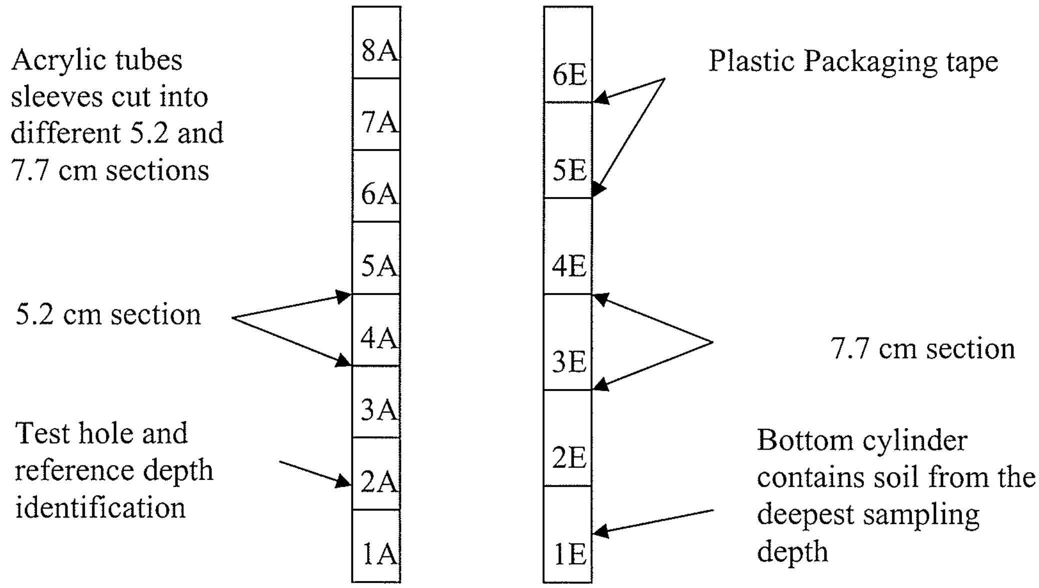

Figure 5.2 Diagram of acrylic tube sleeves.

Three cores were removed from each of the 8 cores collected from the lagoon. A thin metal wire (0.00025 m) was used to cut through the core assembly between the acrylic tube seams perpendicular to the longitudinal axis. The cores from the longer 0.077 m acrylic tube sections, were cut a second time to shorten the cores to a similar length to that of the cores collected in the shorter 0.052 m tubes. This would allow similar testing times.

ASTM D 5084 falling head - constant tail water pressure (Method B) was used in the laboratory to determine the saturated hydraulic conductivity, K_L, of the core samples collected from the pond at the Glenlea testing site.

The following relationship was used:

$$K_L = \frac{aL}{At} ln\left(\frac{h_1}{h_2}\right) \tag{5.1}$$

where:

K_L	=	Saturated hydraulic conductivity, (ms^{-1})
a	=	Cross-sectional area of the reservoir, (m),
L	=	Length of the specimen, (m),
t	=	Interval of time between t_1 and t_2, (s),
A	=	Cross-sectional area of the specimen, (m^2),
h_1	=	Pressure head across specimen at time t_1, (m),
h_2	=	Pressure head across specimen at time t_2, (m).

To correct K_L for changes in viscosity with temperature, K_L is standardized to 20 C by the following relation:

$$K_{20} = R_T K_s \tag{5.2}$$

where:

R_T	=	-0.02452 T + 1.495
T	=	Temperature is in degrees C

The sample length was taken to be the consolidated length after approximately 15 to 24 h after the backpressure and gradient were applied.

5.3 Probe constant determination

After the laboratory hydraulic conductivity (K_L) was determined from the core samples, a probe constant could be calculated by using equation 3.12 for each of the cores. Then one probe constant randomly selected from the testing area, was used to measure the field hydraulic conductivity (K_F) from the rest of the testing sites by using equation 3.13. A core sample was randomly selected by assigning numbers 1 to 8 for cores A to H each of the 8 testing locations and from a table of random digits. In Moore (1995), Table B, row 106, column 4 and the first digit was arbitrarily chosen; which produced a random number 7. Therefore test hole G would serve as a probe constant to predict the K of the other testing locations.

5.4 Core sample particle size analysis

The particle analysis utilized in this study was conducted on twenty-four samples, 3 from each of the 8 cores. The test method consisted of the Bouyoucos or hydrometer method based on "Stokes law" which is described in ASTM D 422-63 Particle-Size Analysis of Soils. Given a number of simplifying assumptions, Stokes law describes the settling velocity of spherical particles in Newtonian fluids. Stoke found that for a given fluid density and viscosity, a particle's terminal velocity will increase or decrease proportionally with the square of its spherical radius under the influence of gravity. This relationship for the settling velocity is simply:

$$V_t = r^2 C \tag{5.3}$$

where:

$$C = \frac{(\rho_s - \rho_f)g}{18\eta} \quad \textbf{(5.4)}$$

V_t	=	Terminal settling velocity, (ms^{-1}),
r	=	Particle radius, (m),
ρ_s	=	Particle density, (gm^{-3}),
ρ_f	=	Fluid density (gm^{-3}),
η	=	Fluid viscosity poises ($gm^{-1}s^{-1}$),
g	=	Gravitational acceleration (ms^{-2}).

5.4.1 Sample preparation

Oven dried core samples were crushed using a mortar and pestle and then sieved through a mesh 0.002 m or less in size and placed into 400 ml plastic containers. 100 ml of a dispersing agent was added to each cup filled with air dried soil. The dispersing agent was a solution consisting of 15.9 g of sodium carbonate, 71.4 g of sodium meta-phosphate, and 2000 ml of reverse osmosis (R-O) water. 300 ml of (R-O) water was then added to each cup. Using soil spatulas, each cup was mechanically processed to break down the soil clumps that formed with the addition of the dispersing agent and R-O water. This process could take up to 20 minutes per sample depending on the clay content. Increased clay content increased the break up time. After consistent soil-water slurry was produced, the samples were allowed to sit overnight in the dispersing solution.

The following day, each sample was individually transferred into a 1000 ml stainless steel milkshake cup and 200 ml of R-O water was added. A spray bottle was used to ensure an entire sample transferred from the plastic cup to the stainless mixing cup. The sample was then mixed in a milkshake mixer for five minutes at a speed which caused turbulent mixing to finish the dispersing process. Approximately 600 ml of the dispersed sample was then transferred into 1000 ml graduated cylinder and then filled with R-O water to make a 1000 ml solution.

5.4.2 Hydrometer readings

A blank sample mixed with 100 ml of the dispersing agent and 900 ml of R-O water was used to calibrate the ASTM 152H hydrometer for temperatures varying from 20 C and for the dispersing agent density differences. The hydrometer was graduated from 0 to 60 gL^{-1} in solution at 1 gL^{-1} increments. A blank off-set was taken to add or subtract the density and temperature variations from 0 gL^{-1} before any sample readings were measured. Using a graduated cylinder mixing rod which was propeller like at one end was then inserted into the samples where 15 s of bottom agitation was followed by 30 s of piston like strokes. This process allowed for a consistent agitation schedule to form a homogenous, dispersed solution. The mixing rod was then removed and the first reading was taken after a 40 s. It can be assumed that after 40 s all the sand had settled out of fluid suspension, and only the silt and clay remained in suspension. The hydrometer was then lowered down into the sample after 30 s and the reading was taken at the 40 s mark. After 6 h, another blank sample was measured, and a second reading was taken from the samples to determine the clay in solution only. The oven dry soil weight was determined from the air dry crushed core samples at a temperature of 105 C for 24 h.

The sand, silt, and clay fractions could be calculated as follows:

$$\text{Sand Fraction} = \frac{(\text{oven dry sample weight/litre solution}) - (\text{silt + clay weight/litre solution})}{(\text{oven dry sample weight/litre solution})}$$

$$\text{Clay Fraction} = \frac{(\text{oven dry sample weight/litre solution}) - (\text{clay weight/litre solution})}{(\text{oven dry sample weight/litre solution})}$$

$$\text{Silt Fraction} = 1 - (\text{sand + clay fractions})$$

6.0 RESULTS AND DISCUSSION

6.1 Hydrogeologic conditions in situ

The K-Probe's performance for electro-osmotic effects suggested that the probe was tested in saturated or near saturated in situ soil conditions. There was an attempt to verify the soil moisture conditions by using the electrodes installed on the probe using a four probe electrical conductivity measurement. However, the data collected from the electrodes was not reliable due to fluctuating data which was indicative of poor connections, water-entrapped-cross-connection, or faulty terminals. Therefore, indirect qualitative observations and direct laboratory moisture contents determined from oven dry methods were utilized to confirm the saturation. It was observed that when inserting the probe into deeper (approximately 0.80 m or more) hard clayey material, the effect of electro-osmosis was minimal. In this case, it was assumed, that unsaturated conditions were predominant at depths deeper than 0.8 m.

6.2 Probe constant determination

The probe constant is the most important parameter and key factor in determining the in situ hydraulic conductivity using the K-Probe. Originally, the probe constant for the K-Probe used in this study was calculated from one laboratory tested core sample, location G, which was randomly selected. However this constant was not used in the calculation of K, for the rest of the samples. Theoretically, the probe constant should have been the same for all the tests, given that the probe constant is a factor of the probe's geometrical shape and area of influence. This was found not to be the case. When calculating the mean probe constant for each core sample tested, the mean constant varied from 3.6×10^{-8} to 5.4×10^{-7} m (Table 6.1). It was then realized that the majority of this variation in the probe constant was the result of a minor design modification to the K-Probe tip that occurred after the first four locations A to D were initially tested. The modification consisted of adding a thin wire to the base of the anode near the K-Probe module and adding a sealing compound. Other errors in the probe constant derive from the laboratory analysis core sample hydraulic conductivity. Core samples may have been unequally consolidated during the falling head tests due to the density differences from the upper to the lower core samples. Each probe constant for both sets of four locations was randomly selected using random numbers listed in Table B located in Moore (1995). The reason for choosing a second probe constant was to compensate for a slight modification to the probe tip module between the first four measurements and the last four. As the shape factor is highly dependent on the geometry of the measuring device, this compensation was conducted. The K-Probe constant (M), was calculated to be 1×10^{-7} m and 2.4×10^{-7} m for location A and E respectively (Table 6.1). The ratio from the probe constants A and E is 2.4.

6.3 K-Probe hydraulic conductivities from the testing site

Sixteen hydraulic conductivities were determined from 8 locations (A to H) using equation 3.13 and are presented in Table 6.2. The first four locations (A to D) K_F's are derived from the probe constant at location A, while the last four locations (E to H) are derived from location E. The mean hydraulic conductivity from the K-Probe method varied from 6.2×10^{-10} ms^{-1} at location D to 6.2×10^{-9} ms^{-1} at location E

Table 6.1 In situ K-Probe constants determined from two current applications at locations A to H.

Test Location	K-Probe Constant, M†		
	Trial 1	**Trial 2**	**Mean**
A	8.8E-08	1.2E-07	1.0E-07
B	1.3E-07	9.2E-08	1.1E-07
C	3.6E-08	3.6E-08	3.6E-08
D	1.7E-07	9.3E-08	1.3E-07
E	1.9E-07	2.9E-07	2.4E-07
F	8.0E-08	1.2E-07	1.0E-07
G	5.2E-08	5.5E-07	5.4E-07
H	2.5E-07	3.3E-07	2.9E-07

†All K-Probe constant units are in m.

Table 6.2 In situ K-Probe Hydraulic conductivity determined from two current applications at locations A to H.

Test Location	K-Probe K_F†		
	Trial 1	**Trial 2**	**Mean**
A*	7.7E-10	5.6E-10	6.7E-10
B*	1.1E-09	1.5E-09	1.3E-09
C*	7.7E-10	7.8E-10	7.8E-10
D*	4.4E-10	8.0E-10	6.2E-10
E	3.0E-09	1.9E-09	2.5E-09
F	2.1E-09	1.4E-09	1.8E-09
G	8.2E-10	7.7E-10	7.9E-10
H	1.1E-09	8.2E-10	9.6E-10

†All K_F units are in ms^{-1}

*K_F derived from the average probe constant at location A ($M=1x10^{-7}m$).
All other K_F are derived from probe constant E ($M=2.4x10^{-7}$ m)

6.4 Spatial variability of K_F

A split-plot liner model was used to determine the variability of the K-Probe method and the field hydraulic conductivities determined in the lab with respect to location. An analysis of variance was completed using SAS 8.2 statistical software. Appendix E contains the raw data used for the analysis and the output data following ANOVA linear model computations. The split plot design model used in the analysis is:

$$\mathbf{K_{ijk}} = \mu + \mathbf{L_i} + \mathbf{E_{ik}} + \mathbf{M_j} + \mathbf{E_{ijk}} \qquad \textbf{(6.1)}$$

Where:

μ = Population mean of K_F

L_i = Main location effect (locations A to H)

E_{ik} = The error term for Location (location x rep)

M_j = Main method effect (K-Probe and Laboratory K_F)

E_{ijk} = Residual error

Initial analysis for the 8 locations (A to H), and 2 methods, were done at an alpha level = 0.05 and are tabulated in Table 6.3. The difference between the mean of the two methods (K-Probe method and Laboratory method) tested is not significant (P = 0.7360). However the location effect was significant (P = <0.0001). This analysis suggest that the K will vary significantly depending on where the measurement is taken.

Table 6.3 Split-plot linear model AVOVA† values for location and method.

Source of Variance	DF	Type III SS	Mean Square	F Value	P > F
Method	1	1.9 E-16	1.9E-16	0.12	0.7360
Location	7	9.9E-14	1.4E-14	8.79	<0.0001
Replicates	1	5.0 E-16	5.0E-16	0.31	0.5822

† ANOVA based on n = 32 observations

A secondary analysis was conducted to examine the variation of K with depth and location, by introducing laboratory K data collected as a function of depth. Each location had sub-locations represented by 3 depths. Each core was tested near the surface of the clay bottom, then near the mid point, which was the same point where the K-Probe measurement was taken, another was taken near the bottom of the core.

A split-plot procedure was used to identify variability between location and depth. Appendix E contains the raw data used for analysis and the output data following ANOVA linear model computations. The split-plot procedure used in the analysis is given by the liner model:

$$\mathbf{K_{ijk}} = \mu + \mathbf{L_i} + \mathbf{E_{ik}} + \mathbf{D_j} + \mathbf{E_{ijk}} \qquad \textbf{(6.2)}$$

Where:

μ = Population mean of K_F,

L_i = Main location effect, (locations A to H),

E_{ik} = Error term for Location, (location x rep),

D_j = Main depth effect, (K-Probe and laboratory K_F),

E_{ijk} = Residual error.

Analysis for the 8 locations (A to H), and 3 depths, were tested at an alpha level = 0.05 and are tabulated in Table 6.4. From Table 6.4, the the mean K_L varied significantly with depth (P = 0.0004). Location effect in this analysis was found to be significant (P = 0.0015). Figure 6.1, illustrates a graph of the mean K_L as a function of 3 depths.

Table 6.4 Split-plot linear model ANOVA† values for location and depth.

Source of Variance	DF	Type III SS	Mean Square	F Value	P > F
Location	7	3.9E-13	5.6E-14	4.29	0.0015
Depth	2	2.6E-13	1.3E-13	9.89	0.0004
Replicates	1	5.9E-18	5.9E-18	0.00	0.9831

† ANOVA based on n = 48 observations from laboratory K

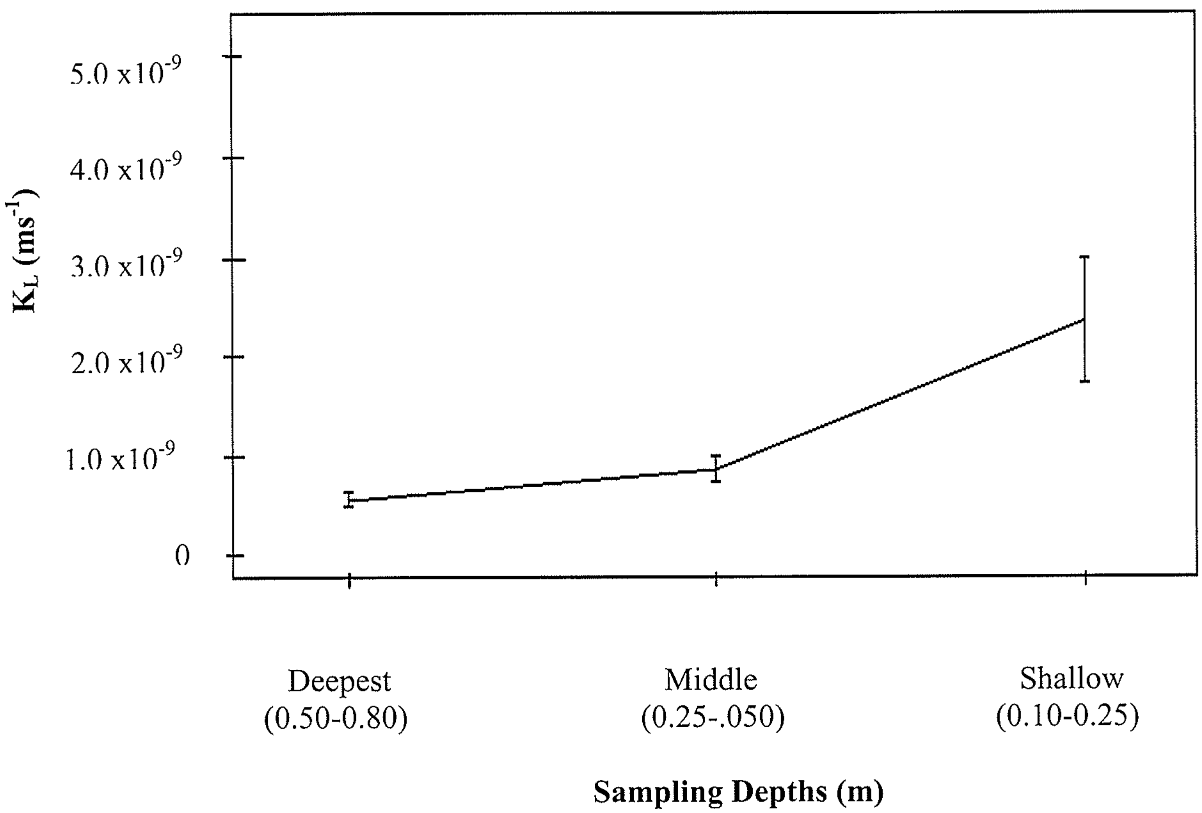

Figure 6.1 Mean hydraulic conductivity plot for 8 locations using the laboratory method, K_L, with one standard error shown as the error bars.

The variability of the mean K_L can clearly be seen from Figure 6.1; the upper layers have a higher K_L, than at the lower depths. If upper layers are scoured causing the lower layers to become exposed to a less compacted environment, then the clay liner integrity can become compromised (increasing K_F). This finding concurs with Rogoski (1990), in that the variability of in situ hydraulic conductivity changes within small distances. This researcher performed seepage tests on a layered system of compacted clay liners for land-filling purposes.

Studying the effects of scouring on effective K in situ would be required to assess the risk of seepage. As Rogoski (1990) suggests, in compacted clay liners where there is spatial variability in K at short distance intervals, there can be variation in K_F compared to K_L and potentially no correlation between K_F compared to K_L on a point to point basis. However, Rogoski (1990) did find that the distributions of K_F compared to K_L were linearly correlated. A very important difference between the present study and the study by Rogoski (1990) is the clay sample origins. In this study, the cores and K-Probe tests were obtained from a retention pond, which was dug from parent material that was naturally deposited, and therefore assumed to be more homogeneous within a small test volume. This was the main reason for choosing a non compacted liner bottom retention pond with naturally deposited saturated clay as opposed to a compacted clay liner.

A compacted clay liner would have presented a greater challenge for calibrating the probe constant essential to accurately measure K_F. In this study, the core removed from the bottom of the retention pond could not be the exact specimen the probe was inserted into. However, the depth of the probe module adjacent to the undisturbed core specimen was within a depth of 0.1 m and a radius of 0.1 m. Therefore in this study, the assumption for calibration of the probe constant depends on the homogeneity of approximately a 3 litre test volume of saturated clay.

6.5 Consolidation effects

Mitchell (1993) describes consolidation effects from electrokinetic phenomenon and the beneficial uses in contamination remediation and soil stabilization. In this study, the range of the electro-osmotic-effect for pressure induction decreased after the second current application (Figure 6.2). In addition, the hydraulic conductivity usually decreased from the initial trial to the second or third trial. This is most likely caused by the consolidation effects from the current application. Additionally, it was observed that the overall effect of pore pressure rise from current application decreased as the number of current applications increased (Figure 6.2). This observed response of consolidation from the pressure curves was not beneficial and could alter the K_F readings after multiple current applications. Therefore, only the first two current applications were used to determine K_F.

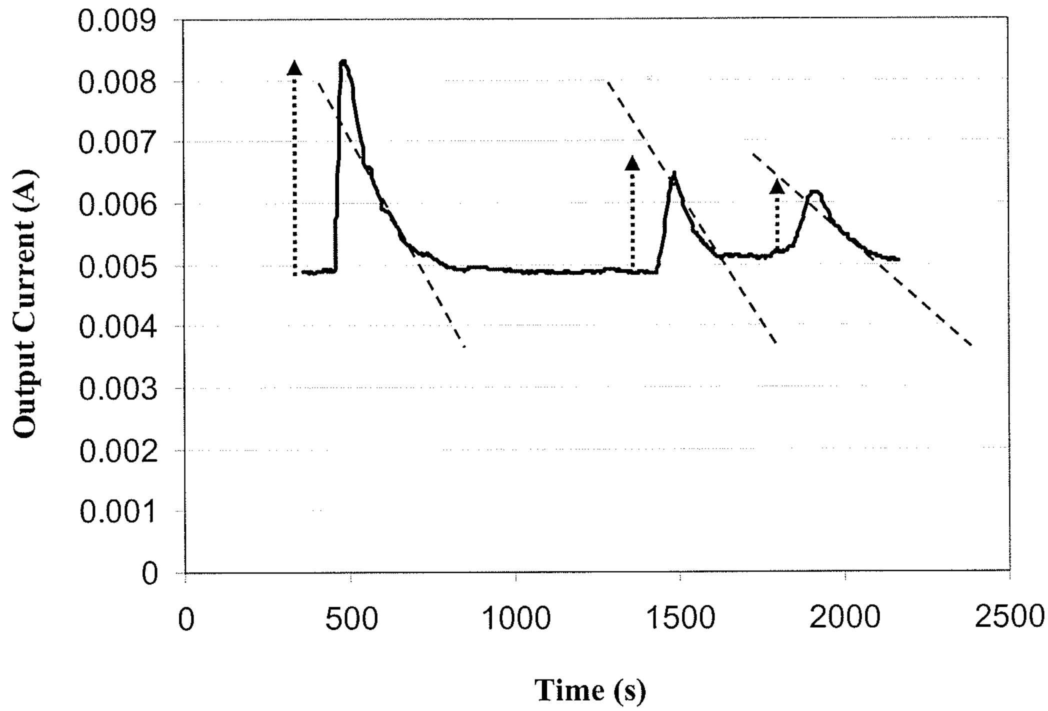

Figure 6.2 Output current as a function of time at location B. The arrows define pressure pore increases for 3 current (A) applications. The dashed lines illustrate the decrease in slope for the equilibrating pore pressures after 3 current (A) applications.

7.0 K-PROBE MEASUREMENT PROTOCOL

A protocol has been developed to identify the procedures and the number of samples required to effectively monitor seepage from clay lined earthen manure storage facilities using the K-Probe. Richards and Thomson (1989) developed a similar permeability protocol for compacted clay liners, in landfill applications, using 45 laboratory permeability tests The K-probe measurement protocol was developed from 24 samples from 8 acrylic tube cores and 8 K-probe in situ tests.

7.1 Objective

The objective of this protocol is to determine the number of measurements required to determine a statistically valid estimate of the bulk in situ clay liner hydraulic conductivity of anaerobic EMS.

7.2 Protocol assumptions

The protocol assumptions are as follows:

- The clay materials being tested are saturated and the K_F values determined in the field are log-normally distributed. The sample locations are chosen randomly.
- The clay liners of EMS are either a cut and fill or compacted clay liner design.
- Hydraulic conductivities tend to vary more than two orders of magnitude within the same hydrogeologic unit and an arithmetic population mean tends to give more weight to higher K_F (Fetter, 1994). Therefore, a geometric mean is used to calculate K_F at different testing locations.

- A minimum of two readings are collected by the K-probe at each testing location.
- The hydraulic conductivities determined in the lab are representative of the effective in situ hydraulic conductivities in the field.

7.3 In situ measurement statistical analysis

7.3.1 Permeability distributions

The permeability values from the core analysis results are expressed in log permeability units and the distribution is illustrated in Figure 7.1. The data is relatively log-normally distributed with an arithmetic mean log-permeability of -7.067 cms^{-1} (8.6 $x10^{-8}$ cms^{-1} linear scale) and an estimated standard deviation of 0.350 cms^{-1} (log-permeability scale). It is important to note that the arithmetic mean obtained from a log normal distribution is the same as the geometric mean from a linear scale model. Richards and Thomson (1989) determined a log normal distribution having a mean log permeability of -8.09 cms^{-1} (8.1 $x10^{-11}$ ms^{-1} linear scale) and an estimated standard deviation of 0.319 cms^{-1} (log permeability scale). It is important to note that the criterion for compacted clay liners in Manitoba is $1x10^{-9}$ ms^{-1} vs. $1x10^{-10}$ ms^{-1} for the landfill liner criterion needed for Richards and Thompson (1989) testing protocol. Richards and Thompson (1989) also noted that, if the number of samples tested increased infinitely, then the distribution would approach the normal distribution (where the skewness and kurtosis are equal to zero). This means every point in the clay liner needs to be tested which is impractical.

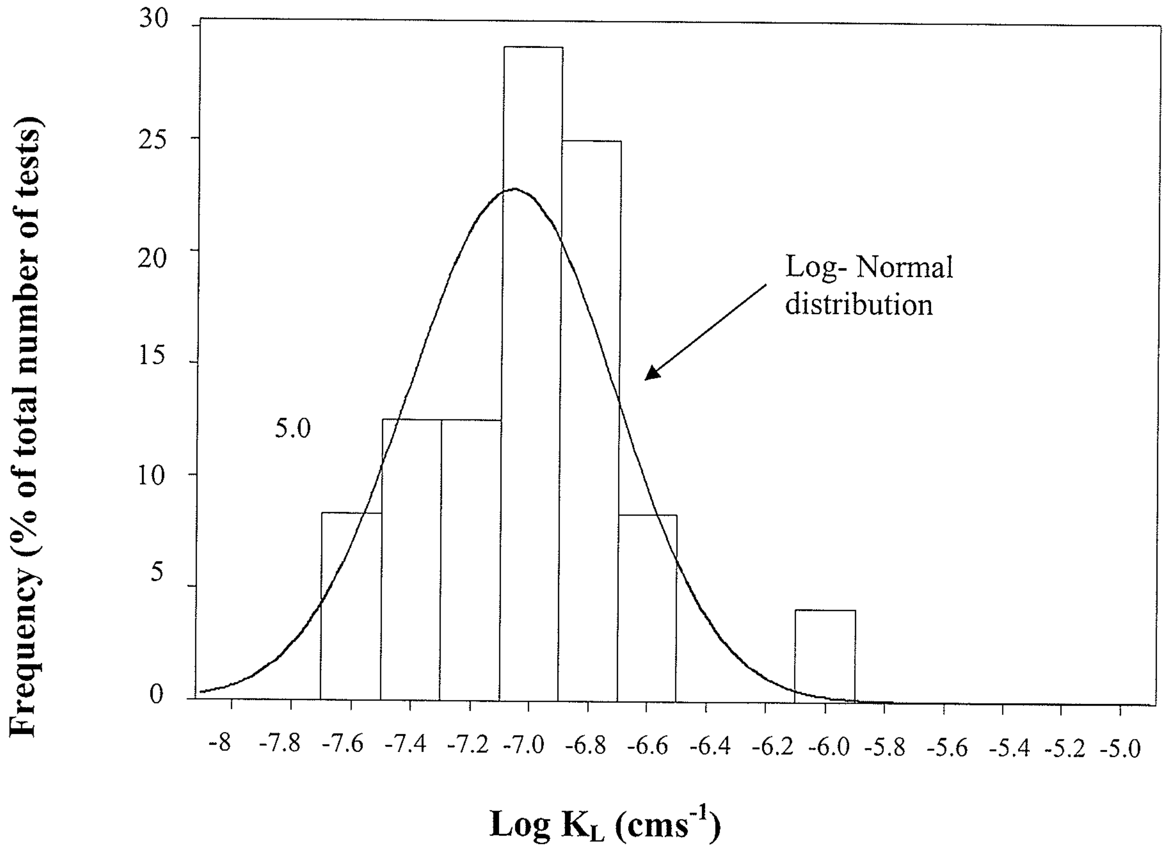

Figure 7.1 Distribution of log-normal and normal theoretical hydraulic conductivity from 24 core analysis.

A second method to test for normality of a distribution is a linearity investigation of the normal percentiles compared to the log K_F values illustrated in Figure 7.2. If the distribution was not normally distributed, then the values would not follow a straight line. However, this is not the case for this data. Figures 7.2, illustrates a linear pattern for the log K_F data. Richards and Thompson (1989), finally concluded that the permeability taken from the log-normal distribution represented the true (or best estimate) bulk permeability through the clay liner.

7.3.2 Sample size requirements

Continuing on the assumption that the permeability data is log-normally distributed, it is concluded that bulk hydraulic conductivity in situ is represented by the mean permeability determined from the previous distribution. A confidence interval used by Richards and Thompson (1989) forms a quality control measure for setting the acceptable limits or boundaries for permeability test results. For the measurement protocol, the confidence interval is given as:

$$CI = \overline{K}_F \pm \frac{t\sigma}{\sqrt{n}} \qquad (7.1)$$

where:

CI = Confidence interval,

$\overline{K}_F$ = Mean bulk hydraulic conductivity $1\text{x}10^{-7}$ cms^{-1} (-7.0 on the log permeability scale)

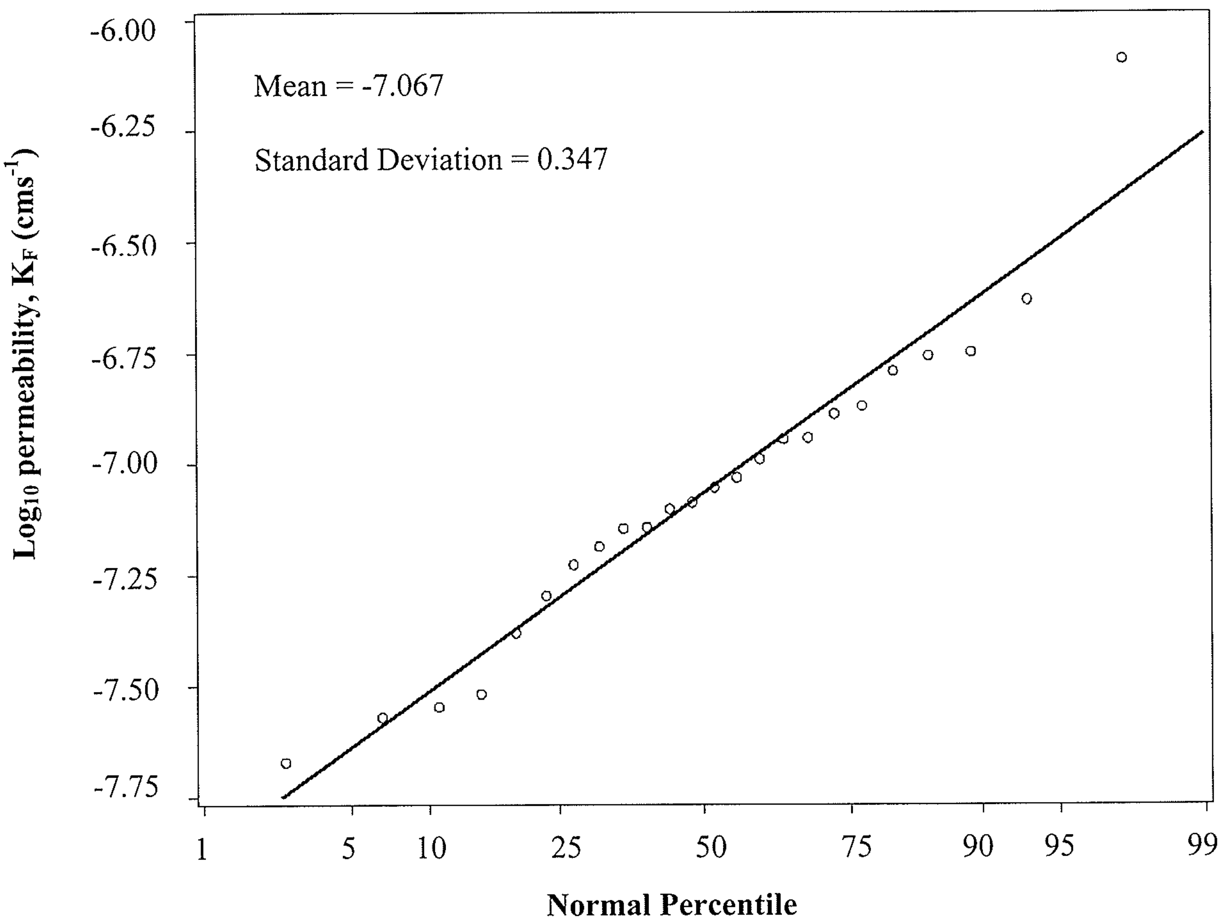

Figure 7.2 Normal probability plot relating the Log_{10} permeability to the normal percentile for use as a diagnostic tool. If the data is normally distributed, then the data will follow a straight line.

σ = Standard deviation from the log normal distribution, (0.350 log permeability units),

n = Number of samples,

t = Student's t-test, (based on n-1 degrees of freedom),

A quality control confidence interval for increasing number of samples is represented by Figure 7.3. All values which fall within this boundary are statistically acceptable for representing the desired geometric mean permeabilities. When the sample numbers reach 10 or greater, the confidence intervals collapse to where the increase in sample numbers do not change the level of variation in the K_F. It is beneficial to obtain the smallest number of samples without compromising the variance in the geometric mean K_F. Therefore, the number of samples used in this protocol to determine the mean K_F is 10.

From Figure 7.3, the range of mean K_F for 10 samples and 95% confidence is 1.52×10^{-9} ms^{-1} to 5.03×10^{-10} ms^{-1}, the upper and lower limit respectively. In other words, if a set of 10 measurements yield a mean K_F greater than 1.52×10^{-9} ms^{-1} then there would be a concern that there is a breech in compliance with the hydraulic conductivity regulation set at 1×10^{-9} ms^{-1} with 95 percent confidence. The lower confidence interval is dropped since it already falls below the Manitoba criterion of 1×10^{-9} ms^{-1}. Therefore, since the two methods used to determine K_F did not produce significantly different results at the $\alpha = 0.05$ level of significance, the number of samples required to be taken in situ by the K-Probe is 10.

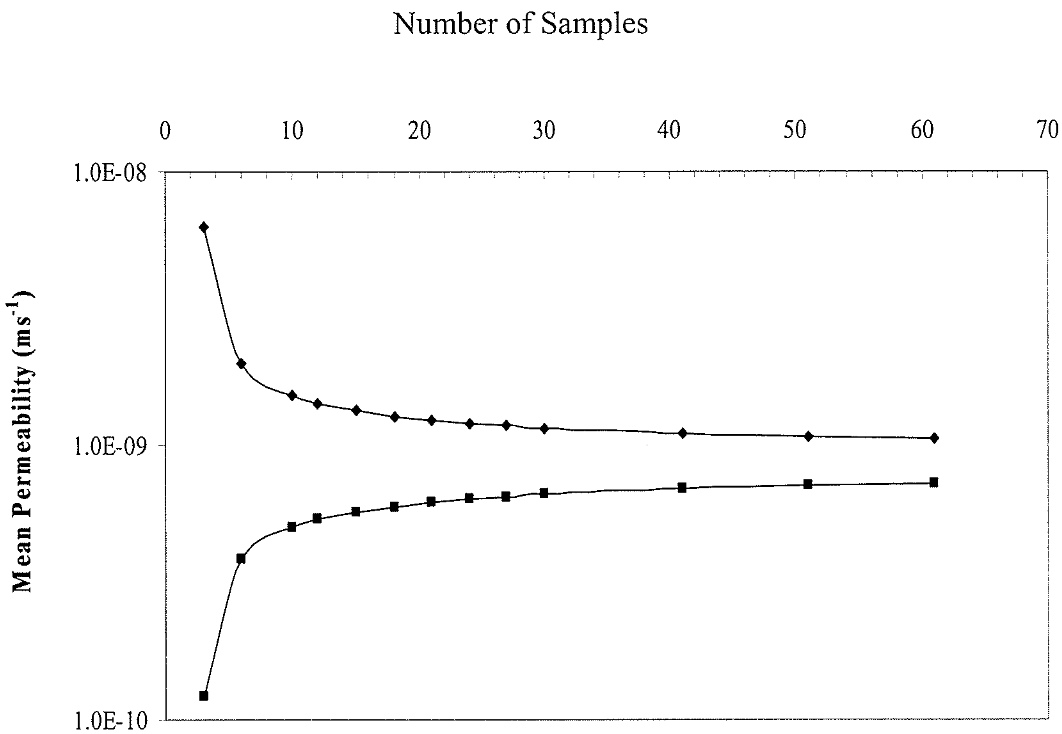

Figure 7.3 Variability in the 95% confidence envelope with increasing sample numbers.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary

A new in situ hydraulic conductivity probe has been developed as a measuring instrument to quantitatively monitor seepage below earthen manure storages. If the mean K_F is above the government set requirement of $1x10^{-9}$ ms^{-1}, more samples need to be retrieved from the field to ensure that government standards are being met. Unfortunately, this requires costly laboratory analysis. Currently, the number of measurements required is not outlined by Manitoba Conservation, nor are there any methods or statistical protocols in place for measuring K_F after EMS are in use. This study describes the method (K-Probe) and the measurement protocol that can be utilized to determine in situ hydraulic conductivity and assess the compliance with Manitoba's regulations.

The following conclusions can be drawn from this study:

- It was a common consensus among EMS consulting and management companies that they would benefit from an in situ hydraulic conductivity device during the EMS clay liner's construction stages.

- The K-probe method used to evaluate K_F in situ was not significantly different from the K_L method values determined in the laboratory ($P = 0.736$). Therefore the K-Probe can be used to determine in situ hydraulic conductivity.

- Electro-osmosis is an effective method to increase pore water pressures in saturated clay and can be used to determine the in situ hydraulic conductivity using data filtering and modified Hvorslev's analysis.

- Consolidation effects were observed in the output data after multiple current applications were applied. Therefore, only the first two current applications were used to estimate in situ hydraulic conductivity.

- This study concurs with Rogoski (1990), in that the variability of in situ hydraulic conductivity changes within small distances. The hydraulic conductivity varied significantly as a function of depth. The upper layers of the clay bottom had greater K_F than lower depths. This is an important result for EMS operations and maintenance. These liners can be damaged from scouring the upper soft clay layers and exposing densely compacted lower layers below. This would alter the designed soil compaction properties, thus changing the hydraulic conductivity of the liner.

- The K-probe measurement protocol requires that the number of measurements required for statistical analysis is 10.

8.2 Recommendations for future research

- Probe miniaturization will minimize the damage to the clay liner during the measurement process.

- The cost of probe materials can be significantly decreased by fabricating and calibrating small silicon pressure transducers (strain or capacitance type) used in the probe module.

- A force transducer in the probe module can be used to monitor the force required to insert the probe. As rocks can be present in the clay liner construction materials, dynamically produced force outputs would serve as a metering system to prevent damage to the K-Probe module. Furthermore, the information from the insertion force required could provide valuable information about the clay liner's compaction, as a function of depth.

- Examine the extent of electrokinetic consolidation effects on hydraulic conductivity of undisturbed clay core samples caused by multiple DC applications.

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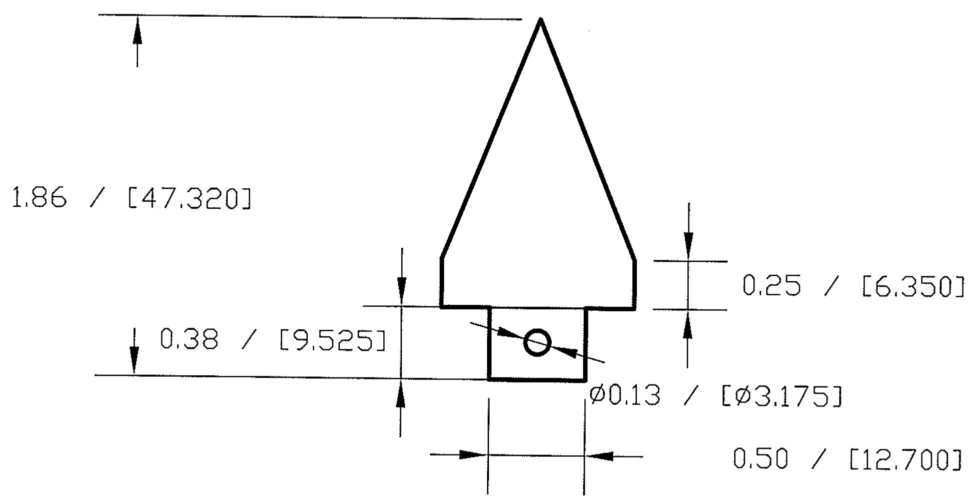
APPENDIX A

Probe AutoCad diagrams and schematics



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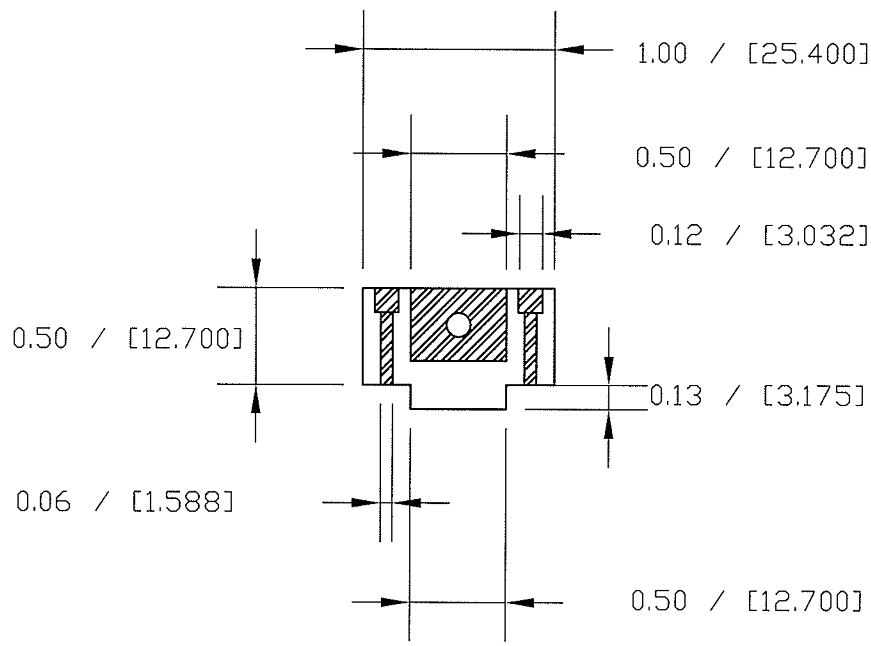
DRAWN BY MARTIN PETRAK	PAPER SIZE A	DWG NO. SCHM-1	REV 1
DIMENSION UNITS Inch / [mm]	SCALE 1/1	DATE MAY 2002	SHEET 1 of 5



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	DESCRIPTION: K-PROBE MODULE CATHODE TIP NYLON SADDLE CONNECTOR FABRICATION SCHEMATIC		
DRAWN BY MARTIN PETRAK	PAPER SIZE A	DWG NO. SCHM-2	REV 1
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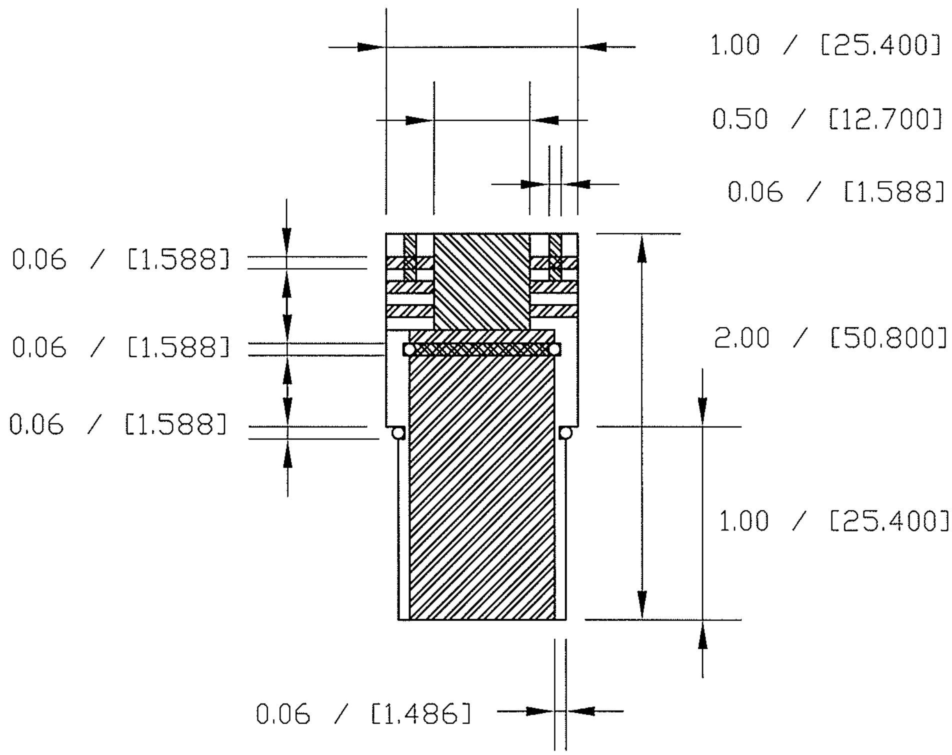
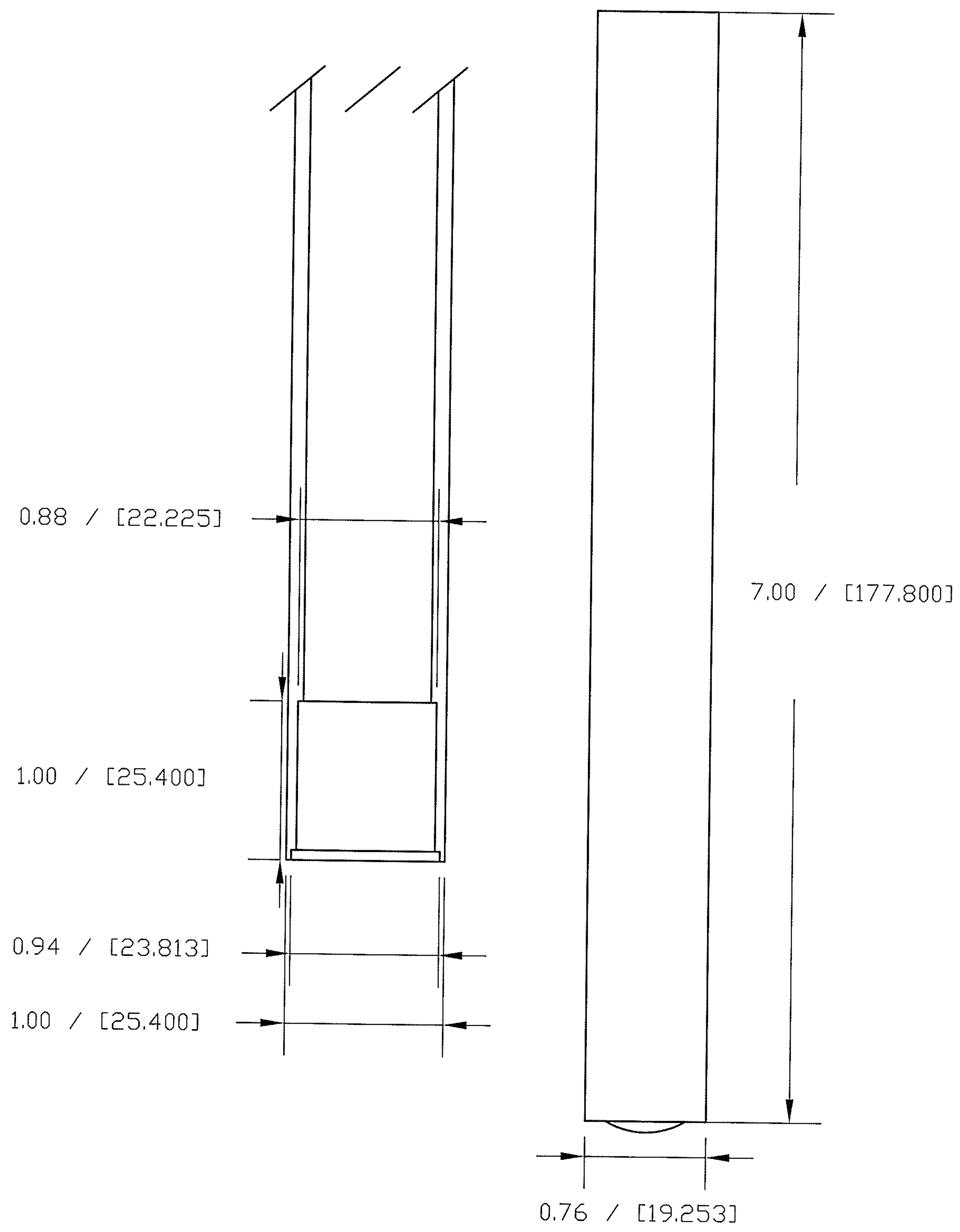
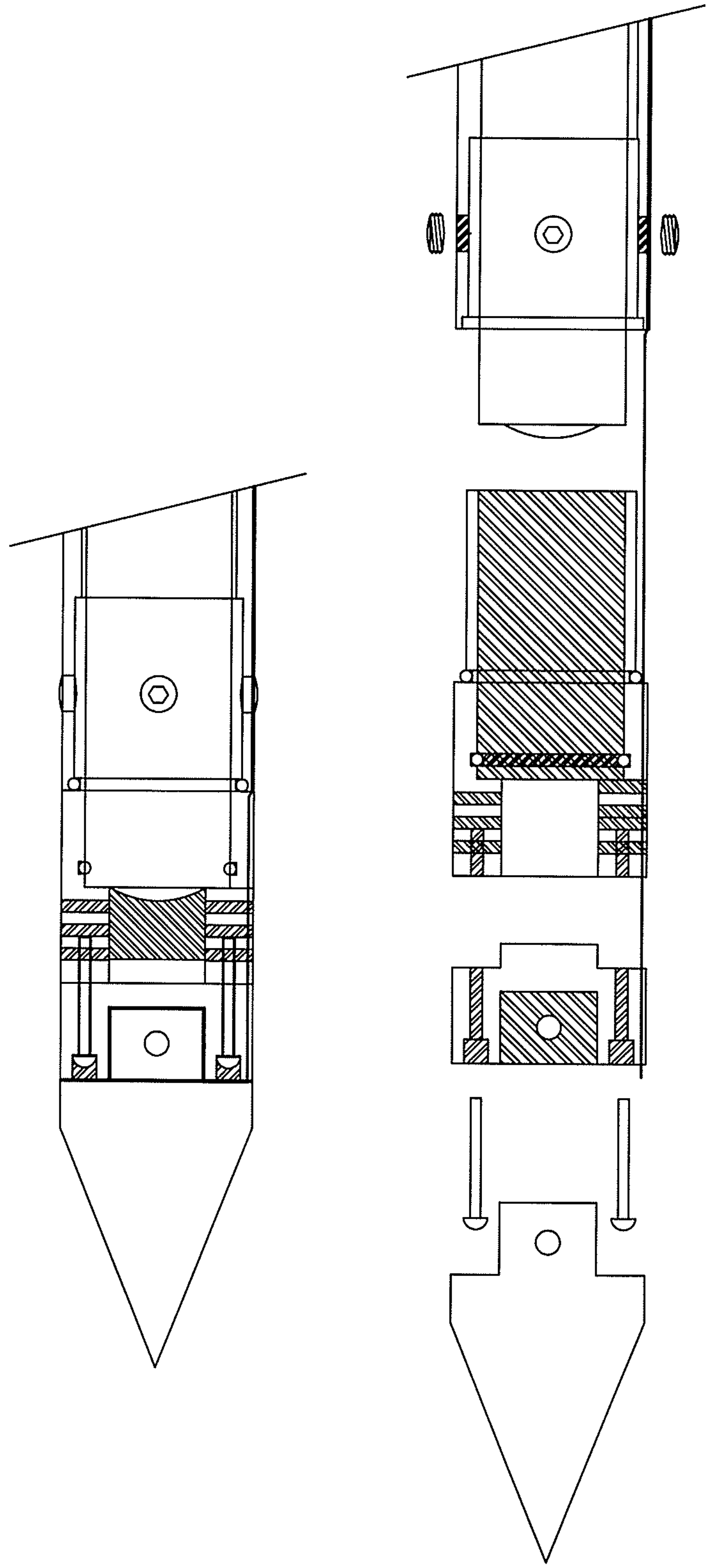


UNIVERSITY OF MANITOBA	DRAWING TITLE	TRANSDUCER JACKET	
DEPARTMENT OF BIOSYSTEMS ENGINEERING	DESCRIPTION	K-PROBE MODULE POUROUS TIP NYLON TRANSDUCER JACKET FABRICATION SCHEMATIC	
DRAWN BY MARTIN PETRAK	PAPER SIZE A	DWG NO. SCHM-3	REV 1
DIMENSION UNITS Inch / [mm]	SCALE 1/1	DATE MAY 2002	SHEET 3 of 5



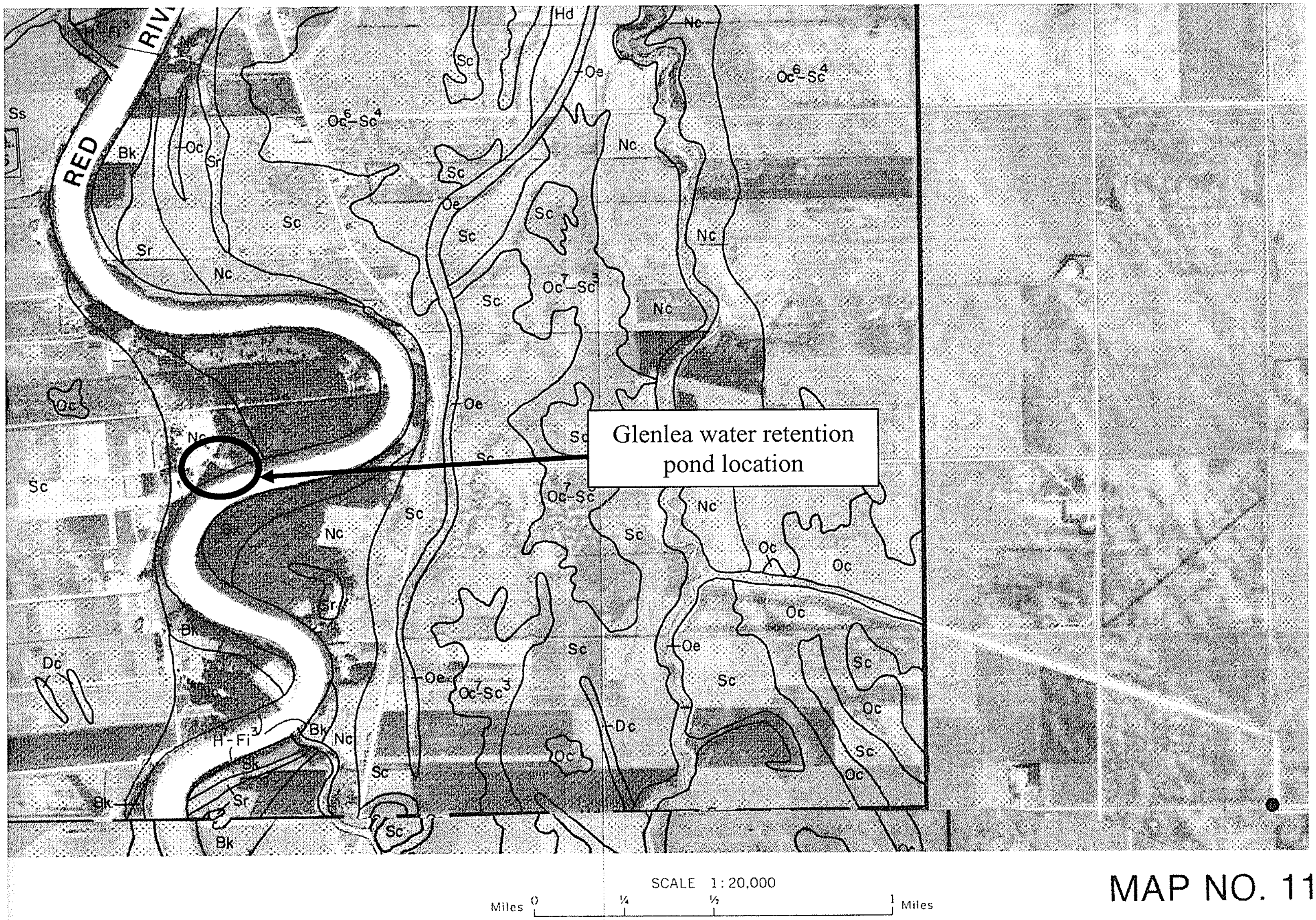
UNIVERSITY OF MANITOBA DEPARTMENT OF BIOSYSTEMS ENGINEERING	DRAWING TITLE PRESURETRANSDUCER AND HOUSING PIPE DESCRIPTION K-PROBE MODULE - FABRICATION SCHEMATIC LEFT - ALUMINUM PIPE HOUSING RIGHT - STANLESS STEEL PRES. TRANSDUCER		
DRAWN BY MARTIN PETRAK	PAPER SIZE A	DWG NO. SCHM-4	REV 1
DIMENSION UNITS Inch / [mm]	SCALE 1/1	DATE MAY 2002	SHEET 4 of 5



UNIVERSITY OF MANITOBA DEPARTMENT OF BIOSYSTEMS ENGINEERING	DRAWING TITLE: K-PROBE MODULE ASSEMBLY			
	DESCRIPTION: K-PROBE MODULE - FABRICATION SCHEMATIC LEFT - PROBE MODULE ASSEMBLED RIGHT - PRE-ASSEMBLY COMPONENTS			
DRAWN BY MARTIN PETRAK	PAPER SIZE A	DWG NO. ASSEM-1		REV 1
DIMENSION UNITS Inch / [mm]	SCALE 1/1	DATE MAY 2002	SHEET 5 of 5	

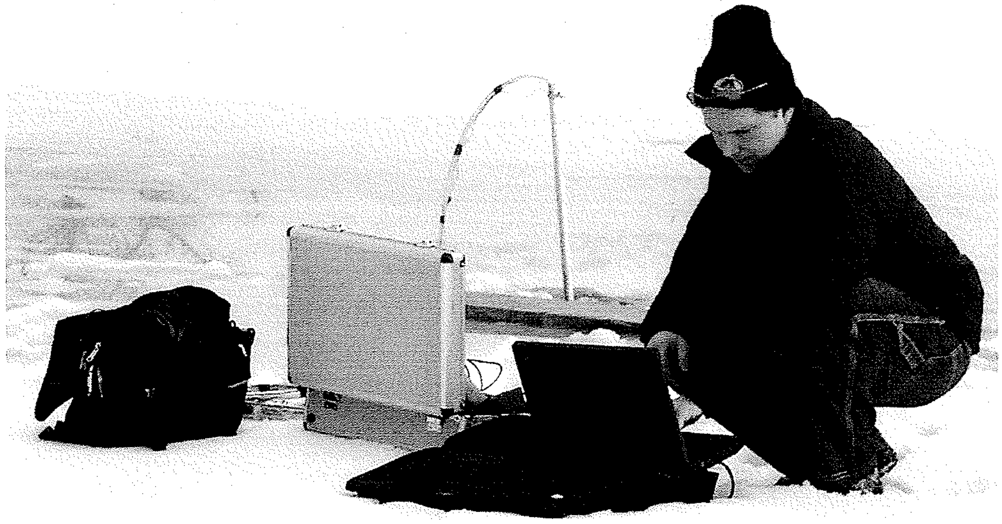
APPENDIX B

Preliminary testing and Glenlea water retention pond photos and aerial photos



MAP NO. 11

Arial photo of Glenlea Research Station and surrounding area. Clay type boundaries are illustrated by the contours provided by the Manitoba Soil Survey in 1975.



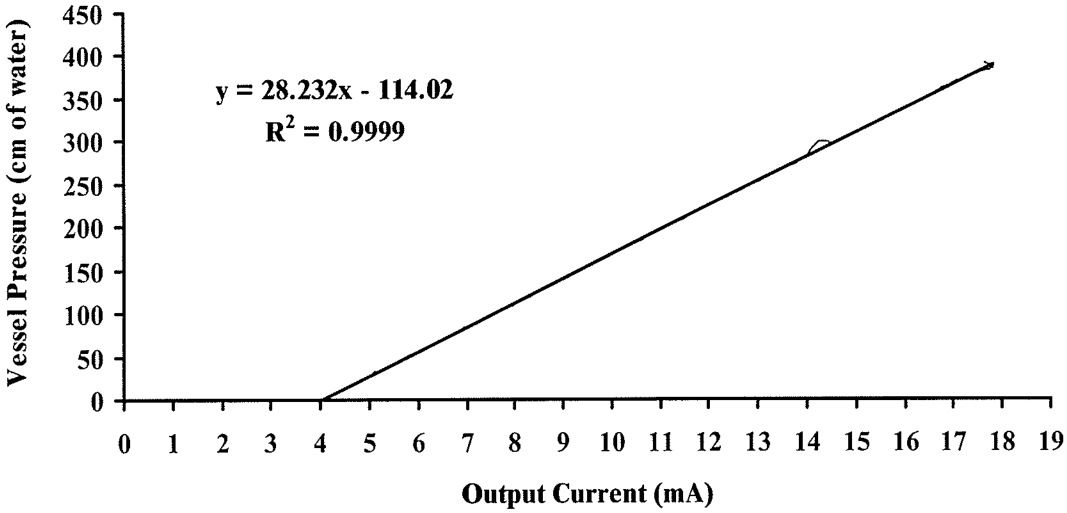
Preliminary data collection through the ice at the University of Manitoba,

South campus, Red River location, January 2003

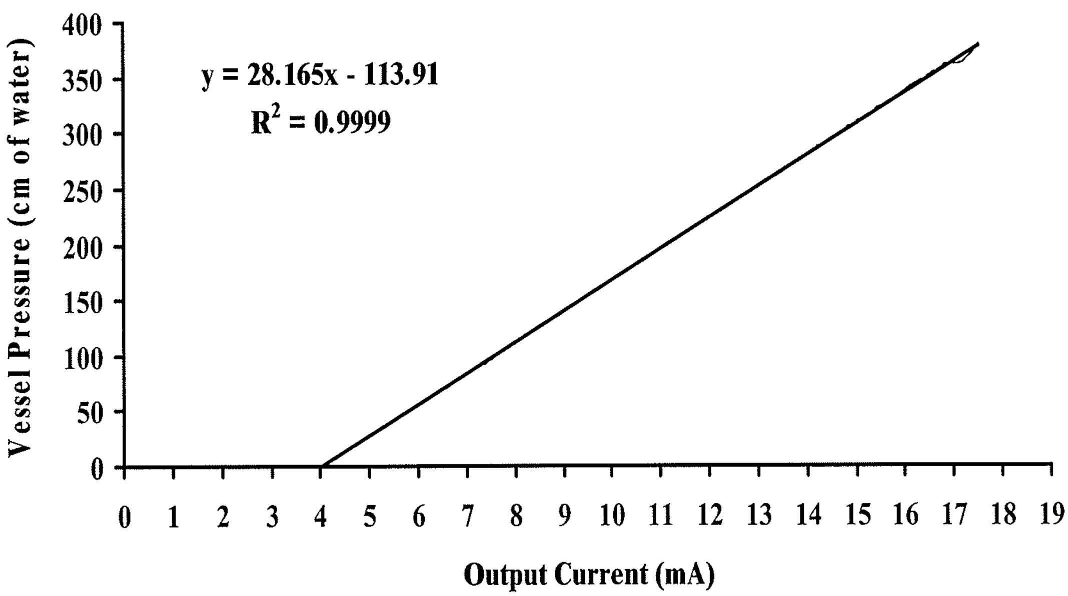
Photo of the water retention pond at Gleanlea, Manitoba, April, 2003.

APPENDIX C

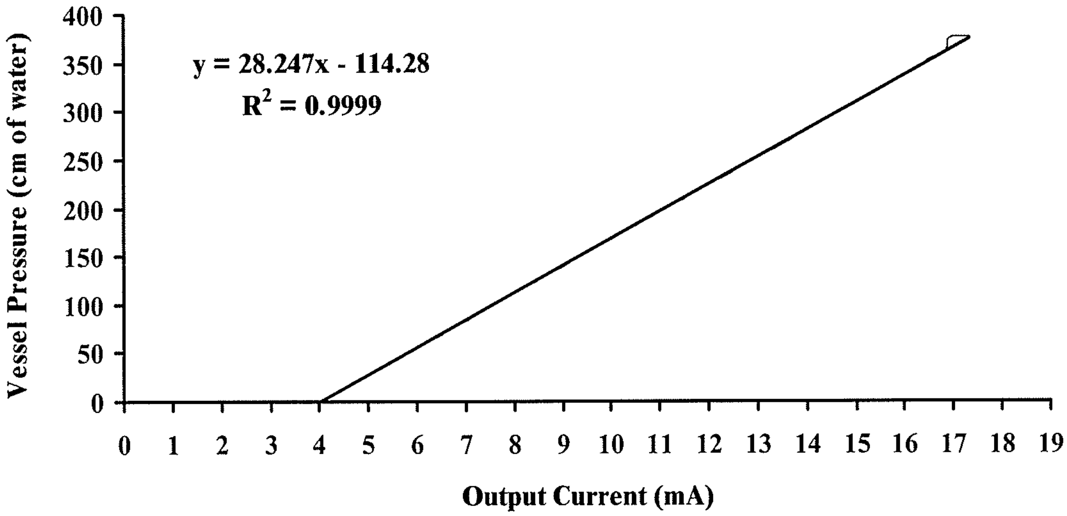
Probe Pressure Transducer Calibration Relationships



The figure above represents the 1st trial probe calibration relationship between output current and measured water pressure



The figure above represents the 2nd trial probe calibration relationship between output current and measured water pressure



The figure above represents the 3rd trial probe calibration relationship between output current and measured water pressure

The table below summarizes the probe calibration relationships between output current and measured water pressure

Trial	Slope	Intercept
1	28.23	114.02
2	28.17	113.91
3	28.25	114.28
Average	**28.21**	**114.07**

APPENDIX D

Particle analysis results using the ASTM Hydrometer Method

The results for the 24 core sample particle analysis using the ASTM Hydrometer Method are tabulated below:

CORE ID	Sampling Depth [m]	% silt + clay	% clay	% sand	% silt
A	0.53	0.999	0.906	0.001	0.093
	†0.38	0.998	0.809	0.002	0.189
	0.17	0.894	0.709	0.106	0.185
B	0.85	0.998	0.867	0.002	0.131
	†0.53	0.950	0.788	0.050	0.162
	0.38	0.942	0.610	0.058	0.332
C	0.57	0.999	0.851	0.001	0.148
	†0.33	0.998	0.888	0.002	0.110
	0.17	0.884	0.635	0.116	0.249
D	0.47	0.990	0.909	0.010	0.080
	†0.27	0.992	0.850	0.008	0.142
	0.12	0.934	0.772	0.066	0.162
E	0.50	0.999	0.873	0.001	0.127
	0.33	0.964	0.774	0.036	0.190
	†0.18	0.945	0.717	0.055	0.228
F	0.50	0.998	0.889	0.002	0.109
	†0.26	0.999	0.900	0.001	0.099
	0.12	0.991	0.773	0.009	0.218
G	0.36	0.998	0.868	0.002	0.129
	†0.23	0.996	0.638	0.004	0.357
	0.16	0.982	0.644	0.018	0.338
	0.10	0.988	0.668	0.012	0.320
H	0.70	1.000	0.862	0.000	0.138
	†0.40	1.000	0.887	0.000	0.113
	0.16	0.971	0.871	0.029	0.100

† Core samples used to compare Laboratory K with K-Probe K.

APPENDIX E

Statistical data determined from SAS 8.2

The data used for a general linear model ANOVA for a split-plot design, are tabulated in the table below. Column one, represents the n^{th} observation of K. The second column represents the 8 locations: A through H respectively. The third column, method, represents the K probe (1) and Laboratory (2) K. The number of replicates and observed hydraulic conductivity readings are given by the fourth and fifth columns respectively.

N	Location	Methods	Replicates	K_F (cms^{-1})
1	1	1	1	7.73E-08
2	1	1	2	5.64E-08
3	1	2	1	6.54E-08
4	1	2	2	6.5E-08
5	2	1	1	1.05E-07
6	2	1	2	1.52E-07
7	2	2	1	1.37E-07
8	2	2	2	1.32E-07
9	3	1	1	7.73E-08
10	3	1	2	7.83E-08
11	3	2	1	2.6E-08
12	3	2	2	2.79E-08
13	4	1	1	4.39E-08
14	4	1	2	8.04E-08
15	4	2	1	6.61E-08
16	4	2	2	7.81E-08
17	5	1	1	2.98E-07
18	5	1	2	1.92E-07
19	5	2	1	2.25E-07
20	5	2	2	2.43E-07
21	6	1	1	2.14E-07
22	6	1	2	1.42E-07
23	6	2	1	7.58E-08
24	6	2	2	6.74E-08
25	7	1	1	8.18E-08
26	7	1	2	7.69E-08
27	7	2	1	1.61E-07
28	7	2	2	1.95E-07
29	8	1	1	1.11E-07
30	8	1	2	8.18E-08
31	8	2	1	1.29E-07
32	8	2	2	9.79E-08

SAS 8.2 data out put for initial analysis for the 8 locations (A-H), and two K methods, are presented below in the general linear model (GLM) procedure and was tested at an alpha level = 0.05.

The GLM Procedure

Class Level Information

Class	Levels	Values
reps	2	1 2
treat	2	1 2
loc	8	1 2 3 4 5 6 7 8

Number of observations 32

The GLM Procedure

Dependent Variable: K

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	9	9.987141E-14	1.109682E-14	6.89	0.0001
Error	22	3.544855E-14	1.611298E-15		
Corrected Total	31	1.3532E-13			

R-Square	Coeff Var	Root MSE	K Mean
0.738039	35.09174	4.0141E-8	1.14389E-7

Source	DF	Type III SS	Mean Square	F Value	Pr > F
Replicates	1	5.024101E-16	5.024101E-16	0.31	0.5822
Method	1	1.878581E-16	1.878581E-16	0.12	0.7360
location	7	9.918114E-14	1.416873E-14	8.79	<.0001

The data used for a general linear model ANOVA for a split-plot design, are tabulated in the table below. Column one represents the n^{th} observation of K. The second column represents the 8 locations: A through H respectively. The third column shows the core sample depth tested in the lab. Sample one is taken at the deepest depth at a particular location, contrary to sample 3 taken nearest to the clay surface. The number of replicates and observed hydraulic conductivity readings are given by the fourth and fifth columns respectively.

N	Location	Depth	Replicate	K_s (cms^{-1})
1	1	1	1	9.31E-08
2	1	1	2	9.3E-08
3	1	2	1	6.54E-08
4	1	2	2	6.5E-08
5	1	3	1	1.79E-07
6	1	3	2	1.71E-07
7	2	1	1	5.23E-08
8	2	1	2	4.89E-08
9	2	2	1	1.37E-07
10	2	2	2	1.32E-07
11	2	3	1	8.06E-08
12	2	3	2	7.76E-08
13	3	1	1	4.12E-08
14	3	1	2	4.23E-08
15	3	2	1	2.6E-08
16	3	2	2	2.79E-08
17	3	3	1	9.8E-08
18	3	3	2	1.06E-07
19	4	1	1	2.81E-08
20	4	1	2	2.88E-08
21	4	2	1	6.61E-08
22	4	2	2	7.81E-08
23	4	3	1	1.65E-07
24	4	3	2	1.57E-07
25	5	1	1	5.34E-08
26	5	1	2	6.53E-08
27	5	2	1	3.85E-08
28	5	2	2	2.23E-08
29	5	3	1	2.25E-07

30	5	3	2	2.43E-07
31	6	1	1	8.75E-09
32	6	1	2	3.41E-08
33	6	2	1	7.58E-08
34	6	2	2	6.74E-08
35	6	3	1	9.53E-08
36	6	3	2	1.33E-07
37	7	1	1	7.90E-08
38	7	1	2	8.46E-08
39	7	2	1	1.61E-07
40	7	2	2	1.95E-07
41	7	3	1	8.53E-07
42	7	3	2	7.73E-07
43	8	1	1	9.32E-08
44	8	1	2	8.35E-08
45	8	2	1	1.29E-07
46	8	2	2	9.79E-08
47	8	3	1	1.3E-07
48	8	3	2	1.29E-07

SAS 8.2 data out put for K analysis for locations A to H, using the K_L method for depth and location are presented below in the general linear model procedure. This analysis was tested at an alpha level = 0.05.

The GLM Procedure

Class Level Information

Class	Levels	Values
rep	2	1 2
depth	3	1 2 3
location	8	1 2 3 4 5 6 7 8

Number of observations 48

Dependent Variable: K

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	10	6.504293E-13	6.504293E-14	4.98	0.0001
Error	37	4.831425E-13	1.30579E-14		
Corrected Total	47	1.133572E-12			

R-Square	Coeff Var	Root MSE	K Mean
0.573787	92.50549	1.14271E-7	1.23529E-7

Source	DF	Type III SS	Mean Square	F Value	Pr > F
replicate	1	5.917685E-18	5.917685E-18	0.00	0.9831
depth	2	2.58253E-13	1.291265E-13	9.89	0.0004
location	7	3.921703E-13	5.602433E-14	4.29	0.0015