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INVESTIGATION OF FLOW THROUGH PEAT FILTERS

**BY
PAULA KENNEDY**

**A Thesis presented to
the Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of
Master of Engineering*
Department of Civil and Environmental Engineering
Carleton University
Ottawa, Ontario
Canada**

JULY 3, 1998

*** The Master of Engineering program is a joint program with the University of Ottawa
administered by the Ottawa-Carleton Institute for Civil Engineering**

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0-612-36889-0

The undersigned recommend to the Faculty of Graduate Studies and Research, acceptance of the Thesis Entitled *Investigation of Flow Through Peat Filters* submitted by Paula Kennedy in partial fulfillment of the requirements for the Degree of Master of Engineering.

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ABSTRACT

Peat moss is a proven natural attenuant, in that groundwater is purified by flowing through peat in wetlands. This has led to peat filters being used in treating several types of waste streams, including septic tank effluent and landfill leachate.

To date, peat filters have been evaluated using a black box approach and the hydraulics and removal mechanisms occurring within the peat are not well understood. In order to determine what is occurring in these systems, investigations into the flow characteristics and the contaminant removal mechanisms are required. This research concentrates on evaluating and modeling flow through an existing peat filter.

A peat filter, which treats septic tank effluent, is located in Almonte, Ontario. The peat filter was instrumented with tensiometers and thermocouples to measure water pressure and temperature profiles with depth. The data was recorded using a remote data acquisition system. The septic tank effluent was intermittently pumped onto the peat filter, and data was recorded with time for several of these pulses.

The hydraulic properties of the peat used in the filter were determined in a laboratory study. The relationship between saturated hydraulic conductivity and dry density appeared to follow a log normal relationship. The conductivity was found to range between 7.9×10^{-4} to 4.0×10^{-5} m/s for a range of dry densities of 117 to 162 kg/m³. The estimated effective porosity ranged between 97 and 61 percent. The pressure-saturation relationship was determined in the laboratory and fit using the van Genuchten model. The values for α_d , α_w , n , the irreducible water saturation and the maximum entrapped air saturation were found to be 8m⁻¹, 20m⁻¹, 2.2, 0.245 and 0.07, respectively.

The peat filter was modeled using a one-dimensional unsaturated flow code. The laboratory values were initially used in the code and changed in attempts to obtain a better match with the field data. The unsaturated flow code was able to predict the pulses within the peat filter reasonably well.

Acknowledgments

I would like to thank my supervisor, Dr. P. Van Geel, for his help and guidance throughout this research project. He gave me the freedom to work independently, while helping whenever asked.

The Natural Sciences and Engineering Research council is greatly acknowledged for the funding provided throughout this project.

Thanks to my family and friends for their support throughout this research. I would especially like to thank both my parents for helping me achieve this goal, even during difficult times.

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SYMBOLS AND NOMECLATURE

α	pressure-saturation van Genuchten fitting parameter
α_d	van Genuchten fitting parameter for drainage relationship
α_w	van Genuchten fitting parameter for the wetting relationship
$\delta h/\delta L$	hydraulic gradient
Δh	change in total head
Δl	length of peat column
Δt	time step size
ΔV	change in volume of water between k and $k+1$
$\Delta x, \Delta y, \Delta z$	dimensions of the grid block
ϕ	porosity
ϕ_e	effective porosity
ϕ_j	porosity in grid block j
ϕ_t	total porosity
$\phi_{t,new}$	new total porosity
θ	moisture content
Θ	effective moisture content
Θ_d	apparent water content for main drainage branch
θ_p	water content at point p on log of capillary head versus saturation plot
θ_r	irreducible moisture content
$\omega_{S.L.}$	moisture content at shrinkage limit $\equiv$ shrinkage limit
θ_s^d	saturated moisture content for the main drainage curve
θ_s^w	saturated moisture content for the main wetting curve
Θ_w	apparent water content for main wetting branch
θ_{wd}	moisture content at a reversal from wetting to drainage
θ_{ws}	saturated moisture content

$\theta_{v,k}$	volumetric moisture content at point k on pressure-saturation curve
ρ_b	bulk density
ρ_d	dry density
$\rho_{d,new}$	new dry density
ρ_s	solid density of peat $\equiv 1.34\text{g/cm}^3$
ρ_w	density of water
Ψ	pressure head of water phase
Ψ_1	pressure head of water phase at Point 1 on Figure 4.3
Ψ_2	pressure head of water phase at Point 2 on Figure 4.3
Ψ_3	pressure head of water phase at point 3 on Figure 4.5
Ψ_4	pressure head of water phase at point 4 on Figure 4.5
Ψ_A	pressure head of water phase at point A on constant head apparatus
Ψ_B	pressure head of water phase at point B on constant head apparatus
A	cross-sectional area
A_c	cross sectional area of column
Al	aluminum
Al^{3+}	aluminum ion
b	intercept of calibration curve for transducer
BC	boundary condition
BOD	biological oxygen demand
C	conductivity of NaCl solution
Ca^{2+}	calcium ion
C_o	conductivity of tracer
CO_2	carbon dioxide
COD	chemical oxygen demand
DO	dissolved oxygen
DOC	dissolved organic carbon
d	slope of calibration curve for transducer
d_s	diameter of peat sample

Fe	iron
Fe^{3+}	ferric ion
g	gravitational acceleration constant = 9.8026 m/s^2
H_2O	water
h_A	total head of water phase at point A on constant head apparatus
h_B	total head of water phase at point B on constant head apparatus
h_c	capillary head
h_{dw}	capillary head at a reversal of process from drainage to wetting
h_p	total head at $\Theta = 0.5$
h_w	total head for water phase
$h_{w,j}$	total head for water phase at grid block j
$h_{w,j+1}$	total head for water phase at grid block $j+1$
$h_{w,j-1}$	total head for water phase at grid block $j-1$
$h_{w,n}$	specified total head boundary condition at bottom of column
$h_{w,1}$	specified total head boundary condition at top of column
h_{wd}	capillary head at a reversal of process from wetting to drainage
IC	initial condition
j	grid block index
$K(\theta)$	unsaturated hydraulic conductivity
$K(\theta)_j$	hydraulic conductivity of block j
$K(\theta)_{j\pm 1}$	hydraulic conductivity of block $j\pm 1$
$K(\theta)_{j\pm 0.5}$	hydraulic conductivity at the interface $j\pm 1/2$
$K(\theta)_{j+1/2}$	hydraulic conductivity at the interface $j+1/2$
$K(\theta)_{j-1/2}$	hydraulic conductivity at the interface $j-1/2$
K^+	potassium ion
$K_{rel}(\theta)$	relative permeability
K_{sat}	saturated hydraulic conductivity
L	end of discretized column
level ₁	level of water in buret when pressure is first change in pressure-saturation determination

level_2	level of water in buret at equilibrium in pressure-saturation determination
L_s	length of peat sample
L_t	length of tubing used in pressure-saturation relationship
m	pressure-saturation van Genuchten fitting parameter
m_a	mass of ambient peat
MAE	mean absolute error
mass_{in}	mass of water into the grid block
mass_{out}	mass of water out of the grid block
m_d	dry mass
ME	mean error
Mg^{2+}	magnesium ion
m_o	mass of organic portion
MOE	Ministry of environment
m_s	mass of soil particles
m_t	total mass
m_w	mass of water
$m_{w,SL}$	mass of water at shrinkage limit
N	nitrogen
n	pressure-saturation van Genuchten fitting parameter
Na^+	sodium ion
NH_3	unionized ammonia
NH_4^+	ammonia ion
NO_2^-	nitrite
NO_3^-	nitrate
p	point halfway between θ_{ws} and θ_r on $\log h_c$ versus apparent saturation plot
P	phosphorous
P_1	total pressure of water phase at Point 1 on Figure 4.3
P_2	total pressure of water phase at Point 2 on Figure 4.3
P_a	air pressure
P_c	capillary pressure

P_w	water pressure
q	Darcy flux
Q	flow rate
q_1	specified flux boundary condition at top of column
q_n	specified flux boundary condition at bottom of column
q_w	volumetric flux vector
rate	rate of evaporation through C-flex tubin
R_{nw}	parameter used in numerical implementation of trapped air
S	absolute value of slope of $\log h_c$ versus apparent saturation
S/S	source/sink term
$\overline{\overline{S_{dw}}}$	apparent saturation at a reversal of process from drainage to wetting
$\overline{S_{nr}}$	effective water saturation considering air entrapment
$\overline{^i S_{nr}}$	effective saturation with maximum allowed air entrapment
$\overline{S_{n,t}}$	effective saturation at intermediary points
S_p	absolute value of slope of Θ with respect to $\log h_c$ at $\Theta = 0.5$
S_r	irreducible saturation
SS	suspended solids
STE	septic tank effluent
S_w	water saturation
$\overline{\overline{S_w}}$	apparent saturation
$S_{w,j}^t$	water saturation at t
$S_{w,j}^{t+\Delta t}$	water saturation at $t+\Delta t$
$\overline{S_w}$	effective water saturation
$\overline{\Delta S_w}$	effective water saturation at reversal from main drainage to wetting
$\overline{\overline{S_{wd}}}$	apparent saturation at a reversal of process from wetting to drainage
$\overline{\overline{S_{w,d}}}$	apparent saturation for the main drainage pressure-saturation relationship
$\overline{\overline{S_{w,ds}}}$	apparent saturation for the drainage scanning curve

S_{ws}	saturated saturation
$\overline{\overline{S_{w,w}}}$	apparent saturation for the main wetting pressure-saturation relationship
$\overline{\overline{S_{w,ws}}}$	effective saturation for the wetting scanning curve
t	time for volume V to exit outflow of constant head apparatus
$t_{1/2}$	time at which $C/C_o=0.5$ during breakthrough
TDS	total dissolved solids
V	volume of water exiting the outflow in a known time t
V_{evap}	volume of water evaporated
v_s	seepage velocity
V_s	solid volume
V_{sample}	volume of peat sample extracted from filter
V_t	total volume
$V_{t,new}$	new total volume
V_v	total volume of voids
$V_{v,deadspace}$	volume of deadspaces
V_{ve}	effective volume of voids
$V_{w,init}$	volume of water initially in sample
$V_{w,k}$	volume of water at point k in pressure-saturation determination
$V_{w,k+1}$	volume of water at point $k+1$ in pressure-saturation determination
$V_{w,SL}$	volume of water at shrinkage limit
z	elevation head
z_1	elevation head at Point 1 on Figure 4.3
z_2	elevation head at Point 2 on Figure 4.3
z_3	elevation head at Point 3 on Figure 4.5
z_4	elevation head at Point 4 on Figure 4.5
z_A	elevation head at point A on constant head apparatus
z_B	elevation heat at point B on constant head apparatus

1 INTRODUCTION

1.1 OVERVIEW

Septic tank systems are commonly used in the treatment of wastewater in locations where sewers are not available. This typically includes rural areas where sewers are not a feasible option. In the United States, septic systems are not restricted to these areas as twenty-three counties have more than fifty thousand households with these systems installed. This large number of systems has made septic tank systems the number one contributor of wastewater discharged to soil and groundwater in the United States (Hagedorn et al., 1981). The septic system consists of a septic tank and a drainage field. The drainage field is a very important unit, as it furnishes most of the chemical treatment prior to the water being released to the environment. In many cases, problems arise with the drainage field. This has resulted in septic systems being the most frequently reported source of groundwater contamination (Hagedorn et al., 1981). One such problem is that the soil may be too permeable causing the water to flow through too quickly. This reduces the amount of contact time between the soils and wastewater, resulting in inadequate treatment. Another concern with drainage fields is the land requirements to treat large volumes of wastewater. For example, a school produces a large volume of wastewater which would require a large drainage field for proper treatment. Hence, a more efficient filter with better removal properties would be needed to reduce the area requirements for adequate treatment.

In the environment, peat is a known natural attenuant purifying water that passes through it. As a result, filters containing peat are now occasionally being used in place of

the drainage field where the aforementioned problems occur. A black box approach has been taken to observe the level of contaminant removal within peat filter systems. It was desired to conduct a more in-depth examination of the contaminant transport mechanisms occurring within the peat filter. To accomplish this, the study was divided into two phases. These include a hydraulic investigation and an investigation of the contaminant removal mechanisms.

The hydraulic properties are important as they govern how the septic tank effluent (STE) will flow through the filter. The hydraulic properties impact the retention time and flow path within the filter and therefore the contact times between the STE and peat material.

An in-depth examination of the contaminant transport may identify where in the peat layer different pollutants are removed. From this it may then be possible to determine which mechanisms are responsible for removing each contaminant, as well as removal efficiencies.

These two phases will give an overall understanding of how peat filters function. This will aid in the design and operation of future peat filters and suggest other possible situations in which they may be used. In addition, an improved understanding of the removal mechanisms may identify other possible filter media, such as compost, which could be used in a constructed filter. This may reduce the need to remove peat from natural wetlands which serve an important environmental function.

1.2 RESEARCH OBJECTIVES

The scope of this research thesis was to complete the first phase, the hydraulics study, of the overall examination of peat filters. The main objective was to model and examine the hydraulics of a peat filter. In order to achieve this research objective, the problem was broken down into three different studies. Fortunately, a peat filter treating STE was available for a field study. This filter was monitored to obtain actual data from a functioning filter. The hydraulic parameters of the peat used in the filter were determined in a laboratory study. The main properties of interest were the saturated hydraulic conductivity, the porosity and the pressure-saturation relationship. A final study involved a modelling exercise to determine how well a one dimensional unsaturated flow code can simulate the conditions recorded in the field. The numerical model was run using the parameters determined in the laboratory study. A trial and error approach was taken in an attempt to match the results with those found in the field. From these three studies it was hoped that some conclusions regarding the flow through peat could be ascertained.

2 BACKGROUND

2.1 SEPTIC SYSTEMS

A septic tank system consists of two separate units: a septic tank; and a drainage field (Hagedorn et al., 1981). The septic tank biologically treats raw sewage and wastewater and then discharges it onto a drainage bed, tile bed or sand filter (Jowett and McMaster, 1995). It operates under anaerobic conditions to partially decompose the organic matter. It also provides conditions that slow the movement of wastewater as it enters the environment and to settle out solids (Butler and Payne, 1995; Hagedorn et al., 1981). Within this tank, three zones are apparent. The first is a scum layer that forms on top of the wastewater. The second portion consists of the wastewater itself. The final zone is a sludge layer which forms as solids settle to the bottom of the tank (Butler and Payne, 1995). The degree of decomposition depends on the size of the tank, the frequency at which the tank is cleaned and the temperature of the water (Butler and Payne, 1995).

The septic tank treated water or septic tank effluent (STE) is subsequently discharged into the drainage field where it is allowed to filter down through the soil matrix (Jowett and McMaster, 1995). This percolation of wastewater through the soil medium is required for further purification before it re-enters the groundwater system (Hagedorn et al., 1981). The effluent is evenly distributed to the drainage field by a system of perforated pipes (Butler and Payne, 1995).

Several advantages of the septic system are: the relatively low capital and

operating costs; the low sophistication of the facility requirements and operating technology; the value of the nutrients which are added to the land; and the recharge of water to the groundwater (Schaub and Sorber, 1977). One major disadvantage of the septic tank system is that the resulting effluent is not necessarily of the high quality desired or required, leading to groundwater and soil contamination (Butler and Payne, 1995).

2.1.1 PROBLEMS WITH SEPTIC TANKS

The effectiveness of the treatment in the drainage field depends on: the physical, chemical and biological properties of the soil; the design of the drainage field; the method of STE application; location of the groundwater table; and the material underlying the soil (Schaub and Sorber, 1977). Some of the properties of the soil which will affect treatment are the permeability, the porosity, the formation of a "biological mat" and the ion exchange capacity. These will affect how fast the water flows through the soil, how many sites are available for purifying reactions to occur, the quantity of microbial species and the capacity to exchange ions. The soil depth and area of the drainage field are a few of the factors that govern what volume of STE the drainage field can treat properly. The STE can be applied either intermittently or continuously to the drainage field. The practised method will alter the aerobic regions located within the soil of the drainage field. The location of the watertable and the type of soil located beneath the drainage field will affect the water pressures and therefore the flow of STE within the drainage field.

The symptoms of a poorly functioning septic system are: odour; surface flooding; solids discharged from the tank; local surface water pollution; groundwater pollution; and

groundwater filling the tank (Butler and Payne, 1995). These symptoms quickly indicate to the operator if the system is functioning properly (Butler and Payne, 1995).

2.1.1.1 OPERATIONAL AND DESIGN PROBLEMS

The most common operational problems are blocked drainage fields and sludge filled septic tanks (Butler and Payne, 1995). The drainage field may block due to the carry over of solids from the tank into the drainage field (Butler and Payne, 1995). Inadequate frequency of cleaning the septic tank is the primary cause of the tank filling with sludge until the solids eventually spill into the field (Butler and Payne, 1995).

Another possible operational problem is associated with the “biological mat”. This mat is a dense microbial layer formed due to accumulation of micro-organisms and is very effective in removing bacterial species in wastewater (Hagedorn et al., 1981). The application of the STE may be such that the growth of micro-organisms is too fast, and the biological mat becomes so thick that it restricts flow (Hagedorn et al., 1981). Further discharge of STE to the drainage field would not be able to percolate through the soil, resulting in surface spill out (Hagedorn et al., 1981). In a properly functioning system, the biological mat should be of sufficient thickness to remove the bacteria but still allow flow (Hagedorn et al., 1981).

Some other problems arise due to improper design of the system. An error associated with the selection of a septic tank is that it may be inefficient or undersized to treat the volume of flow. This may be a result of a poor design or an increase in flow above that considered in design (Butler and Payne, 1995). Errors in the design of the

drainage field are that it may not be of adequate size required for the amount of flow or that the soil is not able to conduct the volume of wastewater. These design flaws may result in the drainage field clogging and water overflowing onto the surface.

2.1.1.2 EFFLUENT QUALITY

Even in ideal circumstances, effluent of the septic system can pose hazards to the environment and to the health of the general public (Jowett and McMaster, 1995). Some of the concerns are health hazards from pathogenic bacteria and viruses resulting from washouts of the biological mat and surface breakouts (Hagedorn et al., 1981). Elevated concentration of nitrogen (N) and phosphorous (P) in the water will result in premature and accelerated eutrophication, which is harmful to the groundwater and the environment (Jowett and McMaster, 1995). Jowett and McMaster (1995) cited the work of Johnson and Kross (1990) in stating that nitrate also poses health hazards since it has been linked to methemoglobinemia (blue baby syndrome) in humans and it is relatively difficult to remove from wastewater. The unionised ammonia (NH_3) form of nitrogen is also known to be toxic to aquatic life at concentrations exceeding 0.1mg/L.

The septic tank itself only reduces the organic content and fecal bacterial populations. Any further contaminant removal occurs as the wastewater percolates through the soil of the drainage field and it therefore depends on the nature of the soil.

2.1.1.3 LIMITATIONS

Several limitations exist with the use of septic tank systems. Two examples of

these limitations are the loading rates to the system and the location possibilities.

The loading rates to the soil medium must be kept sufficiently low that clogging within the drainage field does not occur (Jowett and McMaster, 1995). It is also necessary for there to be some aerated regions for certain reactions to occur. To minimise potential clogging and improve aeration, some systems have been designed to periodically pulse the drainage field with STE rather than to continuously feed the system (Howett and McMaster, 1995).

In some cases, there is no suitable soil or adequate volume of soil for the drainage field within the property lines. As a result, alternative foreign media have been used in place of conventional drainage fields. Peat moss is one such material that is currently being used. The remainder of this thesis will concentrate on peat filters treating STE.

2.2 PEAT

Peat is an inexpensive and abundant material that has been proven to be very effective in wastewater treatment. In nature, contaminant levels in groundwater are reduced considerably as water flows through a natural peat bog. This property has led to the use of peat as an alternative medium for constructed filters. Peat is very inexpensive (approximately 0.09 \$(US) per kg) in comparison to commercial ion exchange resins (4.40-22.00 \$(US) per kg) (Couillard, 1994). Hence, peat filters may be viable option in less populated areas where wastewater treatment plants are not a realistic option and where local soil conditions are poor (Guntenspergen et al., 1980).

2.2.1 FORMATION

Peat is a partially fossilised substance that is dark in colour and of a spongy consistency (Rana and Viraraghavan, 1987; Couillard, 1994). It is formed in areas where the rate of decomposition of the organic material is slower than the rate of the addition of new plant material (Couillard, 1994). The decomposition which does occur is fairly rapid and occurs in surface regions that are aerobic (Given and Dickinson, 1975 and Waksman (1942), cited by Rana and Viraraghavan, 1987). In the poorly aerated areas, deeper in the wetland, the rate of decomposition is very slow. The environmental conditions that are most often associated with the generation of peat are: saturated or almost saturated conditions; low concentration of oxygen; low temperature; low pH and low amounts of nutrients (Williams and Crawford, 1983). The resulting peat substance is a complex material which has multiple components and is highly heterogeneous (Volarovich et al., 1972).

Peat's major constituents, on a molecular level, are cellulose and lignin (Rana and Viraraghavan, 1987). Lignin bears polar functional groups such as alcohols, aldehydes, ketones, acids, phenolic hydroxides and ethers (Adler and Lundquist, 1963, cited by Coupal and Lalancette, 1976). They also found that these functional groups are involved in the chemical bonding occurring between peat and other substances, such as metals.

On a larger scale, peat is composed of high molecular weight waxy products derived from decomposition, plant residue, and low and high molecular weight polymers (Lishtvan, 1981). These basic units of peat are permeable to water and other ions (Lishtvan, 1981).

2.2.1.1 PEAT TYPES

Peat has numerous applications and has been employed in chemical technology, for medicinal purposes and for environmental protection (Lishtvan, 1981). Peat is a variable material and the type of peat used in a particular operation is determined by the characteristics desired for the intended process.

The type of peat is determined by the environmental conditions that existed during its formation. These factors affect the inorganic and organic contents, the pore structure and many other characteristics (Asplund, 1989). These variations result in different peat types such as sphagnum, carex, wood and bryales peats (Asplund, 1989).

Peats can be categorised into three different groups, indicated by the degree of decomposition. The state of decomposition is indicated by the fibre content of the peat. Fibres are pieces of plant tissue that are greater than 0.10mm in size and make up the bulk of a young, newly formed peat (Boelter, 1969). During decomposition these fibres break down into smaller products and therefore the fibre content decreases as decomposition progresses. The first category, 'sapric', is the most decomposed form of peat, with less than one third of the total organic material made up of fibres (Boelter, 1969). The second group is 'hemic', where between one and two thirds is made up of fibres and the last group is 'fibric', with greater than two thirds fibre content (Boelter, 1969).

2.2.2 ZONES WITHIN THE PEAT

Loxham and Burghardt (1983) defined two different zones found within peat. These are the hydrodynamic active and inactive transport zones. The active zone consists

of the peat that has pore sizes which are large enough such that flow will occur in these areas. This is where the bulk of the transport occurs. The inactive zone, also known as the dead zone, is made up of pores which are less than one micron in size. These small pores do not contribute to the flow within the peat. The bulk of the peat is made up of dead zones as most of the pores are quite small.

2.2.3 PROPERTIES OF PEAT

The properties of peat can be divided into physical, chemical and hydraulic categories. The physical characteristics of the peat include the moisture and fibre content, density, total porosity, specific surface area, particle size distribution and pore size distribution (Asplund, 1989). The chemical characteristics include the ash content, pH and chemical composition (cellulose, hemicellulose, humus substances, lignin and bitumen) (Asplund, 1989). The hydraulic characteristics of peat are the hydraulic conductivity, pressure-saturation relationship, water holding capacity and effective porosity.

As stated earlier, the properties of peat are affected by the environmental conditions that existed during its formation, the temperature, the pH and the degree of decomposition (Couillard, 1994). Also when peat is removed from its natural state, drained, dried and milled, significant changes occur in the properties of that peat (Couillard, 1994). Some of these changes are loss of moisture holding capacity and changes in porosity and permeability (Couillard, 1994). Therefore, research using peats as a filter medium is restricted to peat which has been dried and milled. Once the peat is placed in a filter, some of the factors affecting the properties are the pH and state of

decomposition.

The pH not only influences the properties but the structure of the peat (Couillard, 1994). Usually, peat has a fairly acidic pH of about 4 due to the presence of humic acids (Couillard, 1994). At a pH greater than 9, the structure of peat will degrade, whereas at a pH less than 3 its chelating capacity will decrease (Couillard, 1994). It has also been determined that the quantity of ionised functional groups will increase with pH (Volarovich et al., 1972).

2.2.3.1 HYDRAULIC PROPERTIES

The hydraulic properties are important for the use of peat filters in wastewater treatment to ensure proper hydraulics and adequate treatment. The hydraulic conductivity of peat can range by a factor of over 5000 (Nichols and Boelter, 1982). As the hydraulic conductivity is a function of the magnitude and distribution of the area available for flow, it changes with the quantity and size of the pore spaces. The size of the pore spaces is affected by the particle size distribution, the peat type, the dry density, the degree of humification, water content, depth and degree of decomposition. As decomposition progresses, the fibers break down into smaller particles, thereby also affecting the size of pore spaces. For example, the hydraulic conductivity of a slightly decomposed fibric peat can be as high as 3.9×10^{-4} m/s, whereas for a highly decomposed sapric peat, the hydraulic conductivity can be as low as 6.9×10^{-8} m/s (Boelter, 1969).

The water retention will vary greatly depending on the type of peat, its porosity and pore-size distribution (Boelter, 1969). Peat is a very hydrophilic substance due to the

presence of active functional groups. These groups are capable of linkage, resulting in larger retention of water (Lishtvan, 1981).

The pressure-saturation relationship is important when considering the hydraulics. Da Silva et al. (1993) found that about 50% of the water in a saturated peat was drained out of peat when the negative pressure was increased to 2.5kPa (= 0.255 equivalent m H₂O). Further increases in the negative pressure found less drastic water loss and at a suction of 12kPa (= 1.225 equivalent m H₂O) the medium still contained more than 20% water by volume (da Silva et al., 1993). Peat exhibited large hysteretic properties between the drying and wetting curves. Da Silva et al. (1993), cited the work of Beardsell and Nichols (1982) stating that one reason for the large hysteresis may be that at the end of the draining phase the peat will have become more hydrophobic. This causes a lower water content at a specific pressure for the wetting than drainage curve. It was also found that the water content at zero suction varied between the wetting and drying curves, most likely from entrapped air (da Silva et al., 1993).

2.2.3.2 PHYSICAL PROPERTIES

Peat is a highly porous material with a porosity of about 80 to 90% and has been found as high as 95% (Couillard, 1994; Boelter, 1969). Williams and Crawford (1983) cited the work of Given and Dickinson (1975) stating that the pores in the peat often hold large amounts of static water leading to a high internal liquid hold-up within the peat matrix. This large retention time allows significant amounts of ion exchange, microbial activity and other reactions to occur within these areas (Williams and Crawford, 1983).

The specific surface area of peat has been determined to be approximately $2.0 \times 10^5 \text{ m}^2/\text{kg}$ by Couillard (1994) and $2.2 \times 10^5 \text{ m}^2/\text{kg}$ by Tinh et al. (1971). These two values are relatively similar and probably vary due to the peats being of different type and/or source. The large surface area provides numerous sites for adsorption to occur within the peat.

2.2.4 TREATMENT USING PEAT

Peat filters operate under similar principles as conventional septic systems in that they have no requirements for biological or chemical additives, and they remove or greatly reduce the quantities of suspended solids (SS), biological oxygen demand (BOD), pathogenic organisms, P, N, heavy metals and other pollutants (Loxham and Burghardt, 1983; McKee and Connolly; Farnham and Brown, 1972). In some cases, peat has been found to remove 82% of P, 90% of N, 95% of BOD, >99% of pathogenic organisms as well as oils, heavy metals, odours and pesticides (Farnham and Brown, 1972; Couillard, 1994).

Hydraulically, the wastewater flows from points of high to low head. The flow will be faster in larger pores than in smaller pores due to viscous forces. Since the flow will tend to follow the path of least resistance, its flow pattern tends to be tortuous. In peat, the flow tends to be slow enough such that laminar flow can be assumed and Darcy's law is valid (Loxham, 1980). The processes governing the transport of pollutants are advection, dispersion and diffusion. Advection is the movement of contaminants with the bulk flow and dispersion is the spread of the front due to the porous media. Diffusion is a

molecular process by which contaminants move by a concentration gradient. These processes are complicated by the interaction of the peat with wastewater.

2.2.4.1 ION EXCHANGE

Ion exchange is a process involving the exchange of ions between the liquid and solid phase or between two solids that are close in proximity. The charge on the soil surface arises due to different processes such as isomorphous substitution and the magnitude of the total charge depends on the location of the charge site (Ellis, 1974).

Peat has good ion exchange properties due to the presence of humic acids (Viraraghavan and Kikkeri, 1988). Peat exhibits high cation exchange capacities, but also low anion exchange capacities (Couillard, 1994).

The principal cations that are exchanged are: Ca^{2+} (calcium ion), Mg^{2+} (magnesium ion), Fe^{3+} (ferric ion), Al^{3+} (aluminum ion), K^{+} (potassium ion), Na^{+} (sodium ion) and $(\text{NH}_4)^{+}$ (ammonium ion) (Lishtvan, 1981). The functional acid groups, mainly COOH and OH attached to the humic substances, are the principal sites for the ion exchange (Lishtvan, 1981; Volarovich et al., 1972). The cations appear to be bonded by heteropolar interactions and only about 50% of the functional groups participate in the ion exchange reactions, while the remaining functional groups are attached to each other or to water molecules (Lishtvan, 1981; Volarovich et al., 1972).

2.2.4.2 ADSORPTION

Peat moss has an adsorption capacity that is about two to three times lower than

that of activated charcoal (Tinh et al., 1971). Molecules are adsorbed to the peat matrix in an extremely thin layer of molecules that are in contact with the solid matter of peat (Ellis, 1974). Physical adsorption is by van der waal forces that are fairly weak bonds that can be broken easily, thereby desorbing the attached molecule (Tinh et al., 1971). Another type of adsorption, chemisorption, is essentially irreversible and it involves chemical interaction at the surface of the peat (Tinh et al., 1971). The major interactions between the polar groups and peat during adsorption are chelation and formation of other complexes by chemical bond formation (Couillard, 1994).

To evaluate the adsorption capacity of a solid for a given solute, several factors are important. The most important are the particle diameter, surface area, concentration of solute, impurities within the peat, time of contact and concentration of adsorbent (Tinh, V.Q. et al., 1971; Couillard, D., 1994).

2.2.4.2.1 HEAVY METAL REMOVAL

Peat has a high adsorption capacity for transition metals and polar organic materials due to its polar character (Couillard, 1994; Rana and Viraraghavan, 1987). The pH of peat and presence of other ions will affect the sorption capacity for a specific ion (Coupal and Lalancette, 1976). At a pH greater than 8.5 the peat itself is not stable, whereas at a pH less than 3 most metals will be leached from the peat (Coupal and Lalancette, 1976). At pH values between 3 and 8.5, peat can adsorb up to 4% of its own weight with most heavy metals (Coupal and Lalancette, 1976). Most of the metals in the wastewater are in the form of sulphides or hydroxides, and these compounds are adsorbed

physically to the peat particles (Coupal and Lalancette, 1976). The metals that are not in those forms are removed by chemisorption (Coupal and Lalancette, 1976). Peat has been found to effectively remove such elements as antimony, copper, cadmium, lead, mercury, nickel, uranium, zinc, zirconium, selenium, chromium, argon, beryllium, and silver (Virarghavan and Ayyaswami, 1987)

2.2.4.2.2 OIL REMOVAL

Depending on the type of wastewater being treated, oil may be a significant component. It has been observed that peat can remove between 90 and 99% of the free-phase or dissolved hydrocarbons contained in wastewater, depending on the type of oil (Mathavan and Viraraghavan, 1989). Peat has been found to be capable of extracting up to eight times its own weight in oil (Couillard, 1994). A moisture content of about 40% has been found to give the optimal amount of oil adsorbed (Viraraghavan and Ayyaswami, 1987). Couillard (1994) referenced the work of Cohen et al. (1991) which stated that the best peats for oil adsorption are those low in fibre content, low in birefringent organics, high in ash content, high in guaiacyl lignin pyrolysis products and high in furan pyrolysis products.

2.2.4.2.3 PATHOGENIC MICRO-ORGANISM REMOVAL

Micro-organism removal is an important issue for the treatment of wastewater due to the health and environmental implications. Viruses, fecal coliforms and other bacteria are among those causing great concern.

Adsorption is a primary process responsible for bacteria removal from bypassing water (Nichols and Boelter, 1982). Nichols and Boelter (1982) found from Beard (1940) and Cuthbert et al. (1950) that bacteria can survive for a few weeks in a weak acid to weak alkaline solution, whereas in an acid peat environment the bacteria will die off in a few days. It is thought that with the short survival times and good adsorption properties, peat provides excellent bacteria removal (Nichols and Boelter, 1982).

Adsorption is the most rapid mechanism of virus removal and occurs within 15 minutes of contact with peat (Nichols and Boelter, 1982; Sobsey, et al., 1980). Virus retention and survival in peat is a function of several factors, such as: peat type; chemistry of the STE (pH, concentration of total dissolved solids (TDS)), cations and organics; and the hydraulic characteristics (Sobsey et al., 1980). Viruses are generally negatively charged at pH values greater than 5 and positively charged for pH values less than 5 (Nichols and Boelter, 1982). Therefore, given peat's large cation exchange capacity and its acidic nature, virus removal is enhanced at lower pH values where the viruses are positively charged. Another mechanism for virus removal is attributed to microbial activity (Sobsey et al., 1980).

Sobsey et al. (1980) found that the reduction of fecal coliforms exceeded that of virus removals by 1.3 to 3.3 orders of magnitude under all hydraulic loading rates tested. Bacteria are larger in size than viruses making other mechanisms of removal, such as entrapment, and sedimentation possible. Viruses have such a small diameter that adsorption is the only potential mode of removal. The added removal mechanisms for bacteria account for the increased removal efficiency.

2.2.4.3 FILTRATION

The process of filtration is governed by the particle size distribution, the density and the void fraction (Tinh et al., 1971). Particles that are larger than the pore it encounters are filtered out of the wastewater (Couillard, 1994). Compounds may combine to form larger compounds which then may be subsequently filtered. One such example is when iron, calcium and/or aluminum ions form a compound with phosphorous ions which is subsequently filtered out of solution (Couillard, 1994).

2.2.4.4 MICROBIAL ACTIVITY

In a peat system, a diverse microbial community is present (Lens et al., 1994). Some of these are individual and flocculated bacteria, rotifers, ciliates and fungal spores (Lens et al., 1994). These micro-organisms initially consume the dissolved organic carbon (DOC) (Coulson and Butterfield, 1978). Once the DOC is removed, the microorganisms will consume the organic matter of the peat itself causing the peat to disintegrate and organics to be leached out. If the wastewater loading rate is too low, there will be inadequate DOC and the treatment will be limited by the peat decomposition (Nichols and Boelter, 1982). Other micro-organisms that are present are *Nitrobacter* and *Nitrosomonas*, which are responsible for nitrification (Nichols and Boelter, 1982). The microbial activity is a function of the availability of nutrients, the concentration of inhibitory compounds, physiochemical factors, or competitive interactions (Williams and Crawford, 1983).

As discussed in Section 2.1.1.1, when STE is added to the peat filter a biomat

forms at the interface of the peat and the gravel distribution system. The biomat will grow as a result of microbial growth due to the influx of organics within the STE (Anderson, 1981). The biomat is a very important feature of the peat filter as it greatly reduces the quantity of organics and bacteria in the water.

Brooks (1989) found through her research of peat filters that better removal efficiencies were observed in the winter months. She suggested that fungi, rather than bacteria, may be responsible for contaminant removals as bacteria die off at cold temperatures (Brooks, 1989).

The moisture content affects the microbial activity within peat. Couillard (1994) interpreted from Valentin (1986) that the activity decreases slightly when the volumetric water content exceeds 85% and stops completely for a volumetric water content less than 30%. A state between these two values will result in good microbial activity.

2.2.4.4.1 NITROGEN AND PHOSPHOROUS REMOVAL

Wastewaters contain large quantities of nitrogen and phosphorous that require adequate removal, such that the health and environmental effects are eliminated.

For peat filters, nitrogen removal from wastewater is attributed to volatilisation, adsorption, uptake by roots in the vegetation layer, nitrification and denitrification processes and biological uptake (Couillard, 1994; Lens et al., 1994).

Ammonia has also been found to be highly volatile in peats with low moisture content, releasing the nitrogen to the atmosphere (Couillard, 1994).

The ammonia molecule has a pyramidal configuration and a high polarity that

promotes physical sorption to peat (Couillard, 1994).

The vegetative cover is responsible for large quantities of N removal (Couillard, 1994). Farnham and Brown (1972) found the N content of grass clippings from the vegetative cover of a peat filter to be approximately 4.9%, showing good removals.

Nitrification is the first step in N removal by the nitrification-denitrification process. The two bacteria that are accountable for this process are *Nitrosomonas* and *Nitrobacter* (Tchobanoglous et al., 1991). *Nitrosomonas* oxidise NH_4^+ to nitrite (NO_2^-) which is subsequently converted to nitrate (NO_3^-) by *Nitrobacter*. Both of these reactions require large quantities of oxygen, resulting in the necessity of an aerobic region within the peat filter. Lens et al. (1994) referenced the work of Focht and Verstraete (1977), which suggested that it takes several weeks to establish the nitrifying bacteria after the initial loading, as they are slow growers in comparison to heterotrophs. The nitrifying bacteria are very sensitive to many inhibitors that may slow their growth and activities (Tchobanoglous et al., 1991). Brooks (1989) found from Hawkes (1983) that the optimum pH range for *Nitrosomonas* and *Nitrobacter* are 7.0 to 9.0 and 7.0 to 8.6 respectively, and from Alexander (1977) that an optimum temperature range exists between 30 to 35°C with very slow nitrification below 5°C.

Nitrification does not remove the nitrogen but alters its form in the aqueous phase. Denitrification, the next step, is the conversion of nitrate to nitrogen gas for removal. This process is accomplished biologically under “anoxic” (without oxygen) conditions. Some of the kinds of bacteria that can carry out this conversion include *Achromobacter*, *Aerogacter*, *Alcaligenes*, *Bacillus*, *Brivibacterium*, *Flavobacterium*, *Lactobacillus*,

Micrococcus, *Proteus*, *Pseudomonas* and *Spirillum* (Tchobanoglous et al., 1991). Dissolved oxygen (DO) will suppress the enzyme required for this reaction to occur necessitating a region within the peat filter where little to no DO is present (Tchobanoglous et al., 1991). Another important parameter is the pH. The optimal pH generally ranges between 7 and 8 with different optimums depending on the type of bacteria (Tchobanoglous et al., 1991). Brooks (1989) thought that the acidic environment found within peat limits bacterial nitrogen removal. Fungal activity may also be partly responsible for nitrogen removal. Some fungal species that are known to consume all forms of nitrogen were found within peat filters (Brooks, 1989).

Nichols and Boelter (1982) investigated the nitrogen removals of a constructed peat filter with time. During the first three years of operation, good nitrogen removal was observed. However they found in the fourth year that the nitrogen removal efficiency had decreased significantly. It was presumed that the net removal was being counteracted with nitrogen release. They suggested that the amount of N removed by plant roots had reached equilibrium with the amount of N released from the decaying plant roots. They also suggested that the exchange sites would eventually become saturated with N. Microbial immobilisation may also subside after sufficient N has been added for all the microbial needs.

Farnham and Brown (1972) interpreted from Kaila (1956) that the large quantities of P already contained within virgin peat are due to plant residue and the P contained in the micro-organisms. Large portions of P are removed from wastewater by adsorption, microbial assimilation and filtration (Farnham and Brown, 1972; Couillard, 1994).

The organic colloids within the peat have no or little capacity to remove P (Nichols and Boelter, 1982). In acid peats, the phosphate is retained by iron (Fe) and aluminum (Al) compounds (Farnham and Brown, 1972). Therefore, the amount of P removed increases as the ash, Al and Fe contents within the peat increase.

The vegetative cover is also responsible for some P removal, but the amount is quite small in comparison to the other mechanisms (Nichols and Boelter, 1982; Farnham and Brown, 1972). Farnham and Brown (1972) found that the P content of grass clippings from the system was found to be approximately 0.8%. As the N content was greater than this, more significant N removal occurs by root uptake. Farnham and Brown (1972) did however find that systems seeded with quackgrass were found to remove more P, over those with no vegetation. It was thought that the grass would reduce the water content by drawing out water through its root structure. The roots from the grass itself may also provide pathways into the peat causing the water to pass through more evenly. The presence of the grass may also enhance microbial assimilation of P, while also increasing filtration due to a thicker "biological mat".

2.2.4.4.2 ODOUR REMOVAL

Odours from many industrial and municipal treatment plants are a large problem, leading to complaints from surrounding communities. This is a large issue for wastewater treatment plants, food-processing industries and fat-rendering operations (Boehler, 1989). Boehler (1989) suggested that odour treatment occurs as a result of biological oxidation of the odoriferous substances (Boehler, 1989). The first industrial scale biofilter was

constructed in 1979 by George Liebe, in Germany, to eliminate or significantly reduce the odour problems from a manure drying and fat-rendering operation (Boehler, 1989).

Peat filters are capable of efficiently removing a wide range of odours. Therefore, from an ecological and economical point of view, they were found to be one of the more practical methods of treating odour problems in wastewaters (Boehler, 1989; Beerli, 1989). The micro-organism community adsorbs the pollutant and catabolizes it into cell mass, heat and products of respiration (carbon dioxide (CO_2) and water (H_2O)) (Boehler, 1989). The first step in this process is the adsorption of the target pollutant to the peat particle and then to the biofilm surrounding the peat (Beerli, 1989). The final step of the process is that the carbon, N and P (the odour causing molecules), now contained within the biofilm, are assimilated by the microbial population. The odour is thus eliminated from the water (Beerli, 1989).

Important factors for maintaining the growth and activity of the microbial community are water content, temperature and oxygen supply (Boehler, 1989). The water content is the most important of these factors, and the activity of the micro-organisms may be greatly reduced if it is not at optimum (Boehler, 1989). The temperature is the second most important parameter, with an optimum between 20 and 40°C (Boehler, 1989). The necessary nutrients for growth must be in adequate supply (Boehler, 1989).

For the filter to function well, the ratio of surface area of microbial mass to odorous compounds must be maximised and the humidity level maintained (Boehler, 1989). Peat meets both of these criteria well due to its porous structure and its high water holding capacity (Boehler, 1989).

2.2.5 PEAT FILTER DESIGN FOR TREATING STE

The actual design of the peat filter depends on the site, the volume of STE and type of peat. However, there are some general guidelines that can be followed.

Couillard (1994) provided a general design for a peat filter. The base of the filter is composed of sand overlain by gravel. The peat is then placed on top of the gravel. The STE is distributed through perforated pipes surrounded by a gravel drainage system located near the top of the peat layer. The top of the peat layer is then seeded with grass to form a vegetative layer. Collection pipes are located in the lower gravel layer to collect the treated water exiting the base of the filter. The sand and gravel layer hold the peat bed in place, and the gravel has a high permeability thus aiding the drainage of water out of the filter. As previously stated, the vegetative layer aids in the removal of N and P and helps maintain an aerated region.

Clogging is a serious constraint in the design and operational considerations of a peat filter (Lens et al., 1994). It has been found through experience that applying STE to the filter may result in temporary or permanent clogging problems within about 100 days of first application (Lens et al., 1994). The cause of the clogging is due to a build up of a suspended solid (SS) layer, or an excessively thick biomat (Lens et al., 1994). As the humification of peat increases the amount of clogging has been found to also increase (Buelna and Belanger, 1989). There are several methods by which the clogging problem can be diminished. By removing the SS from the STE prior to applying it to the peat filter, the accumulation within the filter diminishes (Lens et al., 1994). Another method of reducing clogging is by backwashing and raking the system, also cutting down the SS

layer (Couillard, 1994).

When the peat is removed from its natural state it is dried and milled before it is used in a peat filter. Upon drying, the fibres of the peat separate and fine particles are excluded resulting in a high void fraction (Couillard, 1994). By properly separating the fibres, uniform permeability through the peat layer can be ensured (Couillard, 1994). The best peats for biofiltration are fibrous and intermediately decomposed peats, because they reduce the internal clogging problem and eliminate the yellow colour of the treated effluent (Buelna and Belanger, 1989).

Some general design parameters such as hydraulic loading rate, moisture content and bulk density, have been determined. The hydraulic loading to the peat filter is an important operational consideration. Rock et.al. (1984) found that a maximum hydraulic loading of 8.1cm/d functioned effectively, whereas, Brooks et al. (1984) determined that the maximum should only be 4.1cm/d. Farnham and Brown (1972) found through testing that a hydraulic loading of 6.25mm/hr (15cm/d) was handled well by all peats considered. These inconsistencies amongst researchers may be due to different peat types and methods of application. Therefore, the peat in question should be examined to determine what loading rate it can handle. A moisture content of less than 30% obtains maximum absorbency within the peat layer and also results in the desired hydraulic properties of the biological mat (Coupal and Lalancette, 1976). The dry density of peat in the filter should be in the range of 100 to 150kg/m³ to minimise clogging and to have a desired permeability (Brooks, 1987).

2.2.6 PRE-TREATMENT OF PEAT

When peat is removed from its natural state, it is usually washed and sieved (Couillard, 1994). It can then be used directly in this form, however, the efficiency of the removal may be affected by the natural characteristics of peat. Some of these adverse effects are low mechanical strength, poor chemical stability and a tendency to shrink or swell (Couillard, 1994). To overcome these problems thermal, chemical or dewaxing pre-treatment practices may be carried out on the peat (Couillard, 1994).

Couillard (1994) found from (Thun et al., 1983) that surface thermolysis treatment has been found to increase the mechanical strength of peat, making the bed less susceptible to clogging.

Heat treatment made the peat more hydrophobic, resulting in it being comparable to other oil-sorbing agents made of plastic, mineral, wood and wood hydrolysis waste (Couillard, 1994). However, Poots and MacKay (1980) found that the adsorptive capacity of peat decreased with heat treatment. They thought that the reduction in adsorption was due to: cross-linking of bonds; the wettability of peat decreasing through drying; shrinkage of pores reducing effective surface area; and wax diffusion towards the surface resulting in decreased wettability.

Chemical pre-treatment may include the treatment of peat with sulphuric acid, hydrochloric acid or any other chemical. Peat treated with sulphuric acid, showed increased cation exchange properties, improved hydraulic properties and a reduction in the fulvic acids leached from the peat (Smith et al., 1977). Peat chemically treated when wet was found to have a greater increase in divalent cation capacity over peat treated when dry

(Smith et al., 1977). Poots and MacKay (1980) tried pre-treating peat with urea formaldehyde to increase the number of adsorption sites. It was found that at all concentrations of urea formaldehyde, the treated peat was less efficient than the untreated peat.

The waxes contained within the peat cause a decrease in the amount of adsorption, as they cause poor contact between the peat and contaminants (Poots and MacKay, 1980). Pre-treating peat by removing the waxes thus increases the adsorptive capacity due to improved contact, resulting in increased mass transfer (Poots and MacKay, 1980).

2.2.7 DISADVANTAGES/FAILURES

Some disadvantages of using a peat filter to treat STE are: the yellowish colour of the effluent; low chemical oxygen demand (COD) removals; and low pH due to peat degradation and leaching of fulvic acids and protons from peat (Couillard, 1994; Fuchsman, 1980). However, even with these problems the quality of the effluent still satisfies the US EPA criteria for discharge into receiving waters (Couillard, 1994). The yellowish colour is due to leaching of fulvic acids from the peat matrix (Fuchsman, 1980). The COD which is removed by peat is counteracted by the addition of fulvic acids, causing initial net COD removals to be low (Rana and Viraraghavan, 1987). With time, the colour diminishes and greater COD removals are attained because the amount of leachable fulvic acids decreases (Rock et al., 1984).

2.2.8 LIFE TIME OF PEAT FILTERS

The peat filter treatment capabilities will be reduced over time due to such factors as clogging, saturation with P, and decomposition (Nichols and Boelter, 1982). Eventually the adsorption sites of the peat system will be exhausted. This raises the issue as to how long a peat filter can be used to obtain adequate removals, prior to regeneration or replacement. Its slow rate of decomposition suggests that its life time may be comparable to that of other on-site systems (Rock et al., 1984). However, the life time of a specific peat filter, for treating STE, is largely unknown and is thought to only have about 14 to 23% of the life time of activated-carbon filters (Nichols and Boelter, 1982; Virarghavan and Ayyaswami, 1987). Nichols and Boelter (1982) investigated a peat filter which, after 8 years of use, showed no signs of approaching failure.

The life time of the peat filter will be dependent on the peat type, peat texture, the formation of the biomat, the saturated peat conditions and the influent characteristics (Anderson et al., 1981). As the wastewater loading increases, the lifetime of the peat filter will be proportionately shorter (Nichols and Boelter, 1982). The physical changes to the structural integrity of the peat filter, due to the application of STE, are largely unknown (Couillard, 1994). The degree of peat decomposition is an important factor as the peat must be replaced once it reaches a highly decomposed state. Couillard (1994) found that the addition of wastewater to the peat filter did not increase the rate at which the peat decomposed. However, Rock et al. (1984) thought that the rate of decomposition would increase with wastewater addition. Their reasoning was that decomposition is affected by nutrients which are provided by STE in large quantities. Once a peat filter reaches the end

of its life, two options are available. One is to completely remove the peat and install a new filter in its place, and the second option is to regenerate the removal properties of the peat.

2.2.8.1 REGENERATION OF PEAT FILTERS

There are several possible methods by which a peat filter could be regenerated. One idea suggested, involves the application of heat to the filter to potentially renew the peat filter (Poots and MacKay, 1980). This is not a viable option as the combustion properties of peat are such that it may burn. Poots and MacKay (1980) referenced Rodman and Shunney (1971) which stated that another method of regeneration is by re-activation of the adsorbent properties of peat by using a virile aerobic biological culture. This method has been found to work successfully on activated carbon filters but has yet to be tested on peat. The culture desorbs and bio-oxidizes the desorbed matter, thereby regenerating the filter (Poots and MacKay, 1980). Other methods may exist, such as backwashing, which is used with activated carbon. This method may not be possible since high flowrates need to be passed through the peat which is difficult due to its restrictive hydraulic properties.

2.2.8.2 DISPOSAL OPTIONS OF SPENT PEAT

Since peat is inexpensive and abundant, the mode of regeneration has to be easy and cost effective. To date, it does not appear from the literature that regeneration is the most cost effective choice. Therefore, the topic of interest then becomes how to dispose

of the spent peat in an environmentally safe manner.

Prior to deciding how spent peat should be disposed, small scale tests should be conducted to determine: the preferential adsorption of specific contaminants to peat; the desorption characteristics; the background concentrations in peat and the water passing through the peat; and conclude whether the peat was pre-treated prior to its use (Cameron, 1978). Once these properties have been determined there are several different options of disposal or use of the spent peat and are as follows:

1. Use as daily cover in a landfill system (Coupal et al., 1976 from Pohland, 1975).
2. If there are no materials present which are toxic to plants, then the nutrients within the used peat can be agglomerated to make fertiliser (Tinh et al., 1971 from Ruel and Sirianni, 1970).
3. If no harmful contaminants are contained within the peat, then it could be combined with other materials to form building materials (Tinh et al., 1971 from Coupal et al., 1970).
4. The peat moss could also be dewatered and burned to produce heat to provide energy for agglomerating remaining peat moss or for production of activated carbon from peat moss (Tinh et al., 1971 from Hassler, 1963). If burning the peat is an option, it is beneficial to precipitate the metals as hydroxides rather than sulphides, since the sulphides will produce sulphur dioxide upon combustion (Coupal and Lalancette, 1976).
5. Another useful option of the peat is to employ it as diking material around a landfill

(Cameron, 1978).

2.3 SUMMARY

Peat appears to be a substance providing good overall contaminant removal from STE. It is inexpensive and abundant providing viable alternatives in wastewater treatment. Peat is an unusual soil in that it is highly porous, has high water holding capacity and low permeability. Determining the values of these parameters is rather difficult due to its swelling and shrinking properties. There is still a need for research to be conducted in the area of the hydraulics and removal mechanisms to determine exactly what is occurring within a filter. This will aid in improving the design of filters and determine other potential uses. It may also help in identifying other soil types or materials that may be used in place of peat, as wetlands play an important role in the environment.

3 PEAT CHARACTERIZATION

The hydraulic properties of peat have been investigated by several researchers in the past. As described in Section 2.2.3, these parameters vary with type of peat, degree of decomposition and conditions during formation. It was therefore important to determine the value of the parameters for the specific peat used in the peat filter studied as part of this research. A sample of the peat used in the filter was obtained for analysis purposes. The parameters that were examined include the moisture, ash and organic contents, the shrinkage limit, the pressure-saturation relationship, the effective and total porosity and the saturated hydraulic conductivity. Some of these properties are a function of the quantity and sizes of pore spaces. As the density increases, the peat compacts more and the quantity of pore space declines. Hence, several parameters are a function of density. The saturated hydraulic conductivity is a function of the cross-sectional area available for flow. It would therefore be expected that as the density increases, the conductivity should decrease. The porosity is also directly related to the volume of the pores and therefore decreases as density increases. The pressure-saturation relationship is affected by the density, as the size and quantity of different pore sizes affects the capillary forces.

3.1 MOISTURE, ASH AND ORGANIC CONTENT

For characterization of the peat in question, the ambient moisture, ash and organic contents were determined. The moisture content was expressed on a dry basis. The procedure used to determine these contents followed that described in ASTM D2974-87,

with a few alterations (See Appendix A, Section A1 for raw data and explanation). Two types of peat samples were investigated. The first will be referred to as the “Whole Sample” consisting of a representative sample of peat. The second, a “Partial Sample”, was also a representative sample but with the large sticks removed by passing the sample through a number 4 sieve.

Table 3.1, contains the averages and standard deviations of the results for each of the moisture, ash and organic contents for each of the two samples. In both cases, six samples were prepared and analyzed to determine the averages and standard deviations. The average organic content ranged from 82.9 to 85.9% by dry weight. This shows that this peat is composed of mainly organic groups, which were described in Section 2.2.1. The remaining portion of the peat was made up of inorganic components. The moisture content was determined on each of the samples at their ambient conditions. For the Whole Sample it was found that the ambient moisture content was about 10.3% and for the Partial Sample it was 21.9%.

Table 3.1 - Moisture, ash and organic contents for the Whole and Partial Samples

	Moisture Content		Ash Content		Organic Content	
	average (%)	standard deviation	average (%)	standard deviation	average (%)	standard deviation
Whole Sample	10.3	2.9	17.0	4.0	82.9	4.0
Partial Sample	21.9	6.2	14.1	10.7	85.9	10.7

The values of the organic and ash contents were similar for both sample types. In the case of the moisture content, the average values differed by 11.6%. The sticks which were removed in the Partial Sample contained very little moisture. Since these sticks

remained in the Whole Sample, the mass of water would be relatively unchanged but the mass of solids would be increased thereby decreasing the moisture content. In all cases, a larger variability existed for the data of the Partial Sample, than that of the Whole Sample, as depicted by the greater standard deviations. The Partial Samples may have had increased variability due to the removal of part of the sample. The larger variance may be attributed to the size and selection of the sample. A large stick within the original sample will impact the parameters determined for that sample.

3.2 COLUMN APPARATUS

A column apparatus was used in the determination of the saturated hydraulic conductivity and the effective porosity. As both of these properties are a function of compaction it was desired to evaluate them at different densities. The columns provided a suitable method of doing this as they could be packed over a range of densities. Six columns were packed at varying compactions and the parameters were determined to obtain a relationship with density. The dimensions and materials of the column, along with the peat preparation and packing method, are given in Appendix A (Section A.2).

3.3 SATURATED HYDRAULIC CONDUCTIVITY

The saturated hydraulic conductivity was determined using the packed column apparatus described in Section 3.2. Saturated hydraulic conductivity is a function of density and was determined using the procedure described in ASTM D4511. The method is essentially the constant head test, in which flow measurements were taken as a constant

head difference existed across the column, such that the saturated hydraulic conductivity could be calculated using a re-arranged form of Darcy's law.

3.3.1 DARCY'S LAW

The fundamental equation for flow through porous media is the Darcy relationship. In 1856, Henry Darcy observed flows through a sand filter and drew some conclusions. He found that the flow was proportional to the change in head and inversely proportional to the length of the column (Bear, 1972). From these observations, the following equation was obtained:

$$q = \frac{Q}{A} = -K_{sat} \cdot \frac{\partial h}{\partial L} \quad \text{Darcy's Law} \quad (3.1)$$

where q is the Darcy flux vector, Q is the flowrate, A is the cross-sectional area, K_{sat} is the saturated hydraulic conductivity and $\delta h/\delta L$ is the hydraulic gradient (Bedient et al., 1994).

The negative sign indicates that the flow of water is in the direction of decreasing head. This equation has been found to hold for systems with laminar flow and is valid for Reynolds numbers less than one and perhaps up to as high as 10 (Bedient et al., 1994). As stated in Chapter 2, Section 2.2.4, the rate of flow through peat is sufficiently low such that Darcy's law is valid.

By re-arranging the Darcy relationship (equation 3.1), the saturated hydraulic conductivity could be determined.

$$K_{sat} = -\frac{Q \cdot \Delta L}{A_c \cdot \Delta h} \quad (3.2)$$

In this equation A_c , Δh and Δl are the cross-sectional area of the column, the head change across the column and length of the column, respectively. The method by which each of these components was measured and the raw data can be found in Appendix A, Section A.3.

3.3.2 SATURATED HYDRAULIC CONDUCTIVITY RESULTS

Six different dry densities of peat were considered. The densities ranged from 117kg/m^3 and 162kg/m^3 . This range contains those densities which were found to exist in the field, as will be discussed in Chapter 4. Table 3.2 presents the averages and standard deviations of the results based on three constant head tests. The raw data obtained during the determination of the hydraulic conductivity can be found in Appendix A, Section A.3.

Table 3.2 - Average saturated hydraulic conductivity and standard deviation at different dry densities

Dry Density (kg/m^3)	Average Saturated Hydraulic Conductivity (m/s)	Standard Deviation (m/s)
117	7.9×10^{-4}	0.18×10^{-4}
126	4.1×10^{-4}	0.12×10^{-4}
135	2.6×10^{-4}	0.06×10^{-4}
144	1.5×10^{-4}	0.08×10^{-4}
153	0.9×10^{-4}	0.01×10^{-4}
162	0.4×10^{-4}	0.01×10^{-5}

Figure 3.1 presents a plot of the saturated hydraulic conductivity versus dry density and a plot of the log of the saturated hydraulic conductivity versus dry density. Usually, the hydraulic conductivity for soils follows a log-linear relationship. This was

Figure 3.1 - Saturated hydraulic conductivity as a function of dry density on a regular plot and on a semi-log plot. Head 1, Head 2 and Head 3 are the highest, mid and lowest head differences used in the parameter determination, respectively.

confirmed for this peat from the linearity of the data points on the semi-log plot. An approximate equation of the line was found to be $K_{sat} = 10^{(-27.071 \cdot \rho_d + 2.068)}$ with an R^2 value of 0.9937 and where ρ_d is the dry density. This high R^2 value shows a very good relationship between the dry density and the log of the saturated hydraulic conductivity.

3.4 POROSITY

Peat has a highly porous structure with numerous dead zones, as previously described in Sections 2.2.2 and 2.2.3.2. As its name implies, limited flow occurs within these dead zones which has led to a distinction between two different porosities. The primary, or total porosity, considers all pore spaces, whereas, the secondary or effective porosity includes only those pores which contribute to flow.

3.4.1 TOTAL POROSITY

The total porosity was calculated from the first principles of geotechnical engineering. From its definition, the total porosity is the ratio of void to total volume:

$$\phi_t = \frac{V_v}{V_t} = \frac{V_t - V_s}{V_t} \quad (3.3)$$

where V_v , V_s and V_t are the void, solid, and total volumes, respectively and ϕ_t is the total porosity. Appendix A, (Section A.4), contains the methodology and data used in the calculations. For these ensuing calculations, the density of the peat solids, and the dry density were required. The value of the solid density of this particular peat was not known or experimentally determined. However, values for five different peats were found

in the literature. They ranged between 1220kg/m^3 to 1570kg/m^3 , with an average of 1390kg/m^3 and a standard deviation of 140kg/m^3 (Poots and MacKay, 1980; Pro-Moss). It should be kept in mind that using this value was an assumption, and has a direct impact on the total porosity results. The values of the total porosity, calculated using both the average and maximum solid density, for the six different dry densities are presented in columns 3 and 4 of Table 3.3. The total porosity is quite high and ranges between 88 and 93% over the range of considered dry densities. In Figure 3.2, the relationship of total porosity with dry density is illustrated showing a linear relationship with total porosity decreasing with increasing dry density. There are two lines representing total porosity on this figure, each calculated from a different solid density of peat.

Table 3.3 - Total and effective porosity as a function of dry density.

#	$\rho_d (\text{kg/m}^3)$	Total Porosity		Effective Porosity	
		$\rho_s=1340 \text{ kg/m}^3$	$\rho_s=1570 \text{ kg/m}^3$	Average	Std. Dev.
1	117	0.916	0.925	0.95	0.004
1a	117	0.916	0.925	0.79	0.009
1+1a	117	0.916	0.925	0.87	0.092
2	126	0.909	0.920	0.82	0.123
3	135	0.909	0.914	0.88	0.062
4	144	0.903	0.908	0.92	0.050
4a	144	0.903	0.908	0.70	0.006
4+4a	144	0.903	0.908	0.83	0.124
5	153	0.896	0.902	0.80	0.051
5a	153	0.896	0.902	0.76	0.080
5+5a	153	0.896	0.902	0.78	0.064
6	162	0.883	0.897	0.92	0.065
6a	162	0.883	0.897	0.73	0.007
6+6a	162	0.883	0.897	0.85	0.104

Figure 3.2 - Total and effective porosity as a function of dry density

3.4.2 EFFECTIVE POROSITY

The effective porosity, as previously defined, represents only those pores which contribute to flow. This consists essentially of all of the pores excluding the dead zones present in the peat. The effective porosity is defined as follows:

$$\phi_e = \frac{V_{ve}}{V_t} = \frac{V_v - V_{v,deadspace}}{V_t} \quad (3.4)$$

where ϕ_e is the effective porosity and $V_{v,deadspace}$ and V_{ve} are the deadspace void and effective void volumes, respectively.

The effective porosity was determined from first principles using a contaminant transport approach and was carried out using the column apparatus previously described. Chloride was used as the conservative tracer to determine the breakthrough curve. The time to reach a concentration of $C/C_0=0.5$ was used to determine the seepage velocity, from which the effective porosity was calculated using the following equation:

$$\phi_e = \frac{q}{v_s} \quad (3.5)$$

where v_s is the seepage velocity. A description of the methodology and results can be found in Appendix A (Section A.4). A summary of the averages and standard deviations at different dry densities can be found in columns 5 and 6 of Table 3.3. The numbers in the first column of Table 3.3 with an attached "a" indicate that the column was re-packed a second time to verify any irregular results. A visual representation of the results is presented in Figure 3.2. A linear line was regressed to fit the effective porosity with dry density. The equation of the line was found to be $\phi_e = -0.0008 \cdot \rho_d + 0.9487$ with a low R^2 value of 0.0162 showing a poor relationship.

On average, the effective porosity was found to be in the middle to high eighties. This value was higher than anticipated. As stated in Chapter 2 peat usually contains a large number of dead end pores and hence the effective porosity should be lower than what was observed. It was thought that this may be a result of the method by which the effective porosity was determined. The column was saturated by forcing water through the peat and this may have opened some of the dead zone pores thereby artificially increasing the effective porosity.

3.4.3 TOTAL VERSUS EFFECTIVE POROSITY

A graph showing the results of both the total and effective porosities, is provided in Figure 3.2. As the effective porosity is only a portion of the total porosity, it was expected to be lower than the total porosity at all densities. This is observed in most cases, however, on occasion, the calculated effective porosity determined from the laboratory column experiments did exceed that of the calculated total porosity determined using the given range of solid densities. This may be due to laboratory error, problems with the initial packing of the column or with the assumptions used in calculating the total porosity. If the solid density of peat was assumed to be a larger value, then the calculated total porosities will also increase. The total porosities calculated using the maximum solid density of 1570kg/m^3 were approximately 1% greater than those using 1340kg/m^3 . This small increase in total density did not greatly improve those cases where the calculated effective porosity appeared to exceed that of the calculated total porosity. As the bulk of the points were below the line and large amounts of variability were observed, it was

assumed that any erroneous values were due primarily to laboratory errors, and not the assumed value of the solid density.

3.4.4 DEAD SPACE PERCENTAGE

The quantity of dead space was computed as the difference between the total and effective porosity, expressed as a percentage of the total void space. The dead space was calculated for each experimental value of the effective porosity and the average was found to be 11% with a standard deviation of 8.2% using a solid density value of 1340kg/m^3 . At the higher solid density of 1570kg/m^3 , the average was 13% with a standard deviation of 9.1%. These values have a 2% difference in their results, but neither are as high as expected. This value may be artificially low due to the removal of sticks from the peat placed in the column used to determine the effective porosity. Likewise, the errors discussed in the previous section will directly affect the dead space calculation.

3.5 SHRINKAGE LIMIT

Peat exhibits large amounts of swelling when peat is wetted and considerable shrinkage when drained. These shrinkage properties could cause difficulties in evaluating the pressure-saturation relationship, which will be described later. It was therefore desired to determine the shrinkage properties of this peat. The shrinkage limit is a parameter which can be used to evaluate the shrinkage properties. The shrinkage limit is defined as that gravimetric moisture content below which no further changes in the sample volume will occur. At moisture contents greater than the shrinkage limit, it is observed that as the

moisture content decreases the volume of the sample also decreases. Once the sample reaches the shrinkage limit, which is a moisture content unique to that soil, there will be no further decrease in the sample volume. The procedure for determining the shrinkage limit was derived from first principles. The methodology can be found with the raw data in Appendix A, Section A.5. The results of the shrinkage limit are summarized in Table 3.4.

Table 3.4 - Results of shrinkage limit experiments

Dry density at shrinkage limit (kg/m³)	Calculated shrinkage limit assuming $\rho_s=1340\text{kg/m}^3$	Calculated shrinkage limit assuming $\rho_s=1570\text{kg/m}^3$
163	4.2	6.5
148	3.6	5.6

The shrinkage limit was calculated at both the average and maximum peat solid density. The results show the shrinkage limit ranging between 3.6 and 6.5.

3.6 TWO-PHASE PRESSURE-SATURATION RELATIONSHIP

A relationship between the capillary pressure and saturation exists and varies depending on soil type. The pressure-saturation relationship will vary based on sizes and frequency of the pores. A typical plot of this relationship is shown in Figure 3.3A. Three different ways of expressing the saturation are the actual, effective and apparent saturations. These will be explained in more detail and are used to facilitate the use of this relationship in the numerical model.

Figure 3.3 - Pressure-saturation relationship shown in terms of (A) the actual, (B) effective and (C) apparent saturation

3.6.1 CAPILLARY PRESSURE

The capillary pressure is calculated as the difference between the non-wetting and wetting phase pressures. When considering a two-phase air-water system, the capillary pressure is expressed as

$$P_c = P_a - P_w \quad (3.6)$$

where P_c , P_a and P_w are the capillary, air and water pressures respectively. By assuming that the air is infinitely mobile, the air pressure can also be assumed to be effectively atmospheric or zero. Based on the assumption that the air pressure is zero, equation 3.6 reduces to a function of the water pressure alone:

$$P_c = -P_w = -\rho_w \cdot g \cdot (h_w - z) \quad (3.7)$$

where h_w and z are the total and elevation heads for the water phase, respectively, ρ_w is the density of water and g is the gravitational acceleration constant.

This relationship can also be expressed in terms of capillary head (h_c) where

$$h_c = \frac{P_c}{\rho_w \cdot g} = -(h_w - z) \quad (3.8)$$

3.6.2 VAN GENUCHTEN RELATIONSHIP

Van Genuchten (1980) derived an equation for the relationship between volumetric moisture content and the capillary pressure or head. This relationship has been found to be generally good for medium and coarse textured soils, but less accurate for fine textured soils (Bedient et al., 1994).

$$\Theta = \left[\frac{1}{1 + (\alpha \cdot h_c)^n} \right]^m \quad (3.9)$$

With

$$\Theta = \frac{\theta - \theta_r}{\theta_{ws} - \theta_r} = \overline{S_w} = \frac{S_w - S_r}{S_{ws} - S_r} \quad (3.10)$$

where $\overline{S_w}$ and Θ are the effective saturation and effective water content respectively. S_{ws} , S_r and S_{ws} are the actual, irreducible and saturated water saturations, respectively, and θ , θ_r and θ_{ws} are the actual, irreducible and saturated volumetric water contents, respectively. The irreducible and saturated water contents are defined as those water contents for which the gradient ($d\theta/dh$) becomes zero, at dryer and wetter states, respectively (van Genuchten, 1980). The parameters α , n and m are values used to fit the curve to the relationship of pressure and effective saturation of the soil in question.

The purpose of using the effective saturation is that it scales the saturations to a range between zero and one, thereby removing the irreducible saturation (see Figure 3.3B). By comparing the relationship between the capillary head and actual saturation (Figure 3.3A) with that of the capillary head and effective saturation (Figure 3.3B), the only difference is the scale on the x-axis. Essentially, the effective saturation is used as an

alternative method of expressing the saturation of a soil sample. The apparent saturation is defined in Section 3.6.2.2.

3.6.2.1 HYSTERESIS

When a negative water pressure is exerted on an initially saturated soil, water will drain out of the system following the primary “drainage” moisture retention curve (see Figure 3.3C). If the sample is drained to the point where no further water could be removed, when the sample is rewetted with water it will follow the primary “wetting” curve (Bedient et al., 1994). In general, the two curves will have differences indicating that the moisture content is not a unique function of the capillary pressure and therefore has hysteretic properties (Bedient, et al., 1994).

The hysteretic properties in the pressure-saturation relationship have been attributed to contact angle, irregular pore geometry, and nonwetting fluid entrapment effects (Hillel, 1980; Corey, 1986; Honarpout et al., 1986; Willhite, 1986).

3.6.2.2 AIR ENTRAPMENT

In many soil systems, air becomes trapped below saturated regions. Trapped air is that soil air which is sealed off from other regions of air by water (Stonestrom and Rubin, 1989). When there is trapped air present, the wetting curve will have a lower saturation at zero capillary head (see Figure 3.3A).

Another method of designating the saturation is that of the apparent saturation (see Figure 3.3C). In this case, the wetting curve saturations have been scaled to range

between zero and one, essentially neglecting the quantity of air entrapment. These curves can also be represented by the van Genuchten equations for both the main wetting and drainage curves (equations 3.11 and 3.12). The air entrapment is considered separately as discussed in Chapter 5.

Each of these different curves can be represented by a form of Van Genuchten's equation, where that of the main wetting branch is:

$$\overline{\overline{S_{w,w}}} = \Theta_w = \left[\frac{1}{1 + (\alpha_w \cdot h_c)^n} \right]^m \quad (3.11)$$

and that of the main drainage is

$$\overline{\overline{S_{w,d}}} = \Theta_d = \left[\frac{1}{1 + (\alpha_d \cdot h_c)^n} \right]^m \quad (3.12)$$

where α_w and α_d are the fitting parameters for the main wetting and main drainage curves, respectively and n is the fitting parameter for both curves. Θ_w and Θ_d are the apparent water contents and $\overline{\overline{S_{w,w}}}$ and $\overline{\overline{S_{w,d}}}$ are the apparent saturations for the main wetting and drying branches respectively. The assumption of using the same value of n for both wetting and drainage was found to be a good approximation for an air-water system (Kool and Parker, 1987).

Researchers have proposed that soil air becomes entrapped or isolated during wetting due to several possible mechanisms:

1. Preferential wetting of some pores over others due to differences in pore water velocities (Peck, 1969; Fayer and Hillel, 1986).
2. Negative pore water pressures near the wetting front (Constantz et al., 1988).

3. Sealing of air-filled pores by water obstructions during wetting (Stonestrom and Rubin, 1989).
4. Dissolution of gases (Stonestrom and Rubin, 1989).

The trapped air will have an effect on several parameters such as the hydraulic conductivity and the effective porosity. Where this trapped air is located within the system depicts the degree to which it affects the hydraulic conductivity and the effective porosity. For example, if the air is located in the dead spaces of the peat matrix, then the hydraulic conductivity and effective porosity will remain relatively unchanged since no or little flow occurs in these areas anyway. Whereas, if the air is located in the active zone, the hydraulic conductivity and effective porosity can be much lower than that of the air-free system (Constantz et al., 1988).

Constantz et al. (1988), analyzed five different soils measuring the amount of trapped air and the effects on the hydraulic conductivity. They found that the average hydraulic conductivity with trapped air was $0.18 \cdot K_{\text{sat}}$, with a standard deviation of $\pm 0.05 \cdot K_{\text{sat}}$. Whereas the entrapped air volume measurements had a mean of 11.8% with a standard deviation of 6.1%. By observing that the standard deviation of the hydraulic conductivity is much lower than that of trapped air, it can be concluded that the volume of trapped air is not a good indicator of the reduced hydraulic conductivity.

3.6.3 VAN GENUCHTEN PARAMETER FITTING METHOD

Assuming that the value of the irreducible saturation is well defined for a system, van Genuchten (1980) proposed a method of determining α and n , assuming that $m = 1 - \frac{1}{n}$. He developed an equation, with m as the only unknown.

$$S = 2.303 \cdot \frac{m}{1-m} \cdot \Theta \cdot (1 - \Theta^{1/m}) \quad (3.13)$$

Where S is the absolute value of the slope of Θ with respect to $(\log h_c)$ (see Figure 3.4).

Figure 3.4 - Graphical representation of determining S_p for the van Genuchten parameter fitting method.

Van Genuchten (1980) stated that the best location to determine the slope, S_p , is half way between θ_r and θ_{ws} . Letting p be the point on the moisture retention curve at which $\Theta=0.5$, the coordinates for this point can be derived from equations 3.9 and 3.10 to give the following:

$$\theta_p = \frac{\theta_{ws} + \theta_r}{2} \quad (3.14)$$

$$h_p = \frac{1}{\alpha} \cdot (2^{1/m} - 1)^{1-m} \quad (3.15)$$

where θ_p and h_p are the moisture content and capillary head at midpoint ($\Theta=0.5$).

Re-arranging equation 3.13, the parameter m can be determined using the following equations

$$m = 1 - \exp(-0.8 \cdot S_p) \quad (0 < S_p < 1) \quad (3.16)$$

$$m = 1 - \frac{0.5755}{S_p} + \frac{0.1}{S_p^2} + \frac{0.025}{S_p^3} \quad (S_p > 1)$$

with S_p being the slope at midpoint.

Once m is known, n can be determined from the initial assumption that $m = 1 - \frac{1}{n}$

and α can be determined from equation 3.15.

This method was carried out on the data obtained from the drainage relationship to determine α_d and n . As it was assumed that n was the same for the wetting curve, only α_w needed to be calculated, from equation 3.15.

3.6.4 PRESSURE-SATURATION RESULTS

As stated in Section 2.2.3.1, peat exhibits hysteretic properties and air entrapment, and therefore both the drainage and wetting curves of the peat were examined. The relationship was evaluated using the pressure cell method. The procedure, apparatus and raw data can be found in Appendix A, Section A.6. The experiment was carried out on three different peat samples to obtain multiple data sets. The van Genuchten parameters were determined for the apparent saturation-capillary head relationship as this is the required format for the numerical model discussed in Chapter 5. The fitting parameters for the van Genuchten curve, the irreducible moisture content and the quantity of trapped air, were obtained using the fitting method and are presented in Table 3.5. A visual representation of the results of the capillary head versus apparent saturation of each dataset can be found in Figure 3.5. The figure contains the data points as well as the van Genuchten curve fitted to the data.

Table 3.5 - Van Genuchten fitting parameters for the three datasets.

	Dataset 1	Dataset 2	Dataset 3
Dry Density (kg/m^3)	119	100	162
Effective Porosity	0.87	0.87	0.85
$\alpha_d (\text{m}^{-1})$	8.6	8.2	7.1
$\alpha_w (\text{m}^{-1})$	16.1	20.0	21.2
n	2.3	2.2	2.2
S_r	0.16	0.27	0.33
maximum trapped air	0	0.14	2×10^{-3}
Mean absolute error (MAE)	0.033	0.030	0.031

Figure 3.5 - Three pressure-saturation relationships determined on the peat. Both the raw data and van Genuchten curves are presented

The results of the pressure-saturation relationship illustrate the hysteresis between the main drainage and wetting curves. In the approximation of the equations for both branches of the curves, the same value of n was used. This appeared to give reasonably good results, but a better fit to the wetting curve could be obtained if n was not assumed to remain constant (i.e. n drainage equals n wetting).

The capillary rise determined in the lab was considerably lower than expected. Due to the pore size distribution and the hydrophilic properties of peat it was expected that the capillary rise would be higher than what was observed.

In Section 2.2.3.1 the pressure-saturation results from some research conducted by da Silva et al. (1993) were presented. To compare the data from the current research with that from the literature, a plot of the capillary head versus actual saturation was prepared (Figure 3.6). By investigation of this figure, the saturation at corresponding capillary heads could be determined. Da Silva et al. (1993) found that at capillary head of 0.255m equivalent water represented a saturation of 50%, compared with 46% in this research. They also stated that a capillary head of 1.225m equivalent water corresponded to 20% saturation. For the current research, the saturation was found to be approximately 19%. From this comparison it was found that the results obtained in this research compared well with that found in the literature.

The values of α and n for the van Genuchten relationship appear to be relatively similar for the three datasets. However, the maximum amount of trapped air differed greatly, from zero to 0.139. This may be a result of different dry densities, or simply the sample itself. From this result it was difficult to determine whether air

entrapment was a major factor. The irreducible saturation also varied by a large amount from 0.160 to 0.334 over the three datasets.

Figure 3.6 - Capillary head versus actual saturation for Dataset 2.

The table also contains the mean absolute error (MAE), as an indicator of how well the van Genuchten relationship approximated the datapoints. All three of the datasets show a relatively small MAE, showing that the van Genuchten model approximates the pressure-saturation relationships well in this case.

3.7 SUMMARY

The peat was found to be highly organic with an organic content ranging between 82.9 and 85.9%. This peat was also found to have a low ambient moisture content, with the Whole Sample having an average of 10.3% and that of the Partial Sample with an average of 21.9%. The saturated hydraulic conductivity was found to range between 4.2×10^{-4} to 7.9×10^{-4} m/s with a relationship with dry density as $K_{sat} = 10^{(-27.071 \cdot \rho_d + 0.068)}$ with a high correlation of 0.9937. The effective porosity was quite high, with an estimated average in the mid eighties. The pressure-saturation relationship showed large amounts of hysteresis and varying amounts of trapped air. The van Genuchten parameters were found to be approximately 8m^{-1} , 20m^{-1} and 2.2 for α_d , α_w and n , respectively. The irreducible saturation ranged between 0.16 and 0.33 and the maximum amount of trapped air ranged between 0 and 0.139.

4 FIELD STUDY

A field study was conducted to obtain real data from an existing peat filter. This provided a means of observing what actually occurs within a functioning filter. The filter was designed and constructed by a consulting engineering firm. The layout, calibration and installation of the instrumentation and all data acquisition presented in this chapter were carried out as part of this research.

4.1 PEAT FILTER DESCRIPTION

The peat filter that was used for this investigation was located in Almonte, Ontario. The system functioned in the same general fashion as a conventional septic system, except that the peat filter replaced the drainage bed. The system treated the wastewater generated from an elementary school. The filter was designed to handle the wastewater created by a population of 750 students. The loading of STE to the filter commenced in September, 1996. The system consisted of a septic tank, a pumping house, two distribution systems and two peat filter beds.

The wastewater generated in the school was initially sent to the septic tank for preliminary treatment. The STE was then discharged to the pumping house, where it accumulated until a specific volume of 1.380m^3 was attained. Once the STE reached this volume, it was pumped to a distribution system, which disbursed the STE evenly across one of the filter beds. The next time the system reached this volume, the STE was spread onto the other bed. The two beds provide redundancy, in case one of the beds

experienced clogging or any other problems. It was observed that, on average, the STE was pulsed to the filter every one to two hours, during school hours. It took approximately four to eight minutes for the entire volume of STE to be pumped to the distribution system and onto the filter.

The peat filter consisted of several soil layers which served different purposes (see Figure 4.1). The bottom layer consisted of a sloped clay layer, with a very low hydraulic conductivity. The clay acted as a barrier soil to restrict the flow of the treated effluent from exiting to the environment via the bottom of the filter. The treated effluent collected on top of the clay layer, traveled down grade to the back of the filter and was discharged to the environment via a sand infiltration gallery. The treated effluent passed by a stand pipe from which samples could be taken to determine removal efficiencies as a requirement for MOE regulations. On top of the clay was a sand layer followed by a gravel layer which provides support for the peat bed. On top of this was the actual peat, which was seeded with grass and covered with straw to minimize erosion. Distribution pipes were located at a depth of approximately 0.5m below final grade, and were surrounded by gravel to provide even distribution of STE to the top of the filter.

The peat was of sphagnum type, which was taken from a bog in New Brunswick. The peat was dried and milled after removal from its natural environment. The peat had an initial moisture content of 50 to 60% at the time of placement, with a pH of 3.5 to 4.5. The peat was placed in lifts, hand raked and compacted to 125 to 150kg/m³.

Figure 4.1 - Instrumentation layout

4.2 INSTRUMENTATION LAYOUT

At the time of construction, a two meter deep plywood box was placed in the peat filter. The bottom of the box sat on top of the sand layer and provided access to allow for instrumentation over the entire depth of the peat filter. The equipment was placed into the filter through holes drilled in the walls of the box. Transducers and tensiometers were used to monitor water pressures and thermocouples were used to record temperatures. A remote datalogger system was programmed to record the instrument data at specified time intervals. A schematic of the cross-section of the instrument layout is presented in Figure 4.1. This figure illustrated the location of each of the tensiometers and thermocouples. In addition, peat samples were physically removed from six locations identified on Figure 4.1 to determine the density profile with depth. The markings for the tensiometers indicate the point where the measurement was being taken, not where the tensiometer assembly entered the box.

The aim of monitoring the water pressures within the peat filter was to observe the pressure response due to the vertical flow of the STE through the peat filter each time the system was pulsed. The water pressure could then be related to the degree of saturation. The tensiometers were placed in a vertical column since it was assumed that the flow was primarily in the vertical direction. The elevations of each of the pieces of equipment is shown in Table 4.1.

The thermocouples were placed at several different depths within the peat filter. This was to observe how the temperatures varied with time at several locations within the peat. The temperature was of interest for several reasons. The first was to see how these

filters would react in a northern climate. If the STE were to freeze during the cold winter months, then flooding could become a problem. The second reason will be more relevant once the contaminants are investigated as the temperature may affect removal efficiencies. The main reason the temperatures were monitored during the research was to initiate the collection of temperature data.

Table 4.1 - Elevations of each instrument as shown on Figure 4.1.

0.30m tensiometers		0.60m tensiometers		Density measurements		Thermocouples	
Label	Elevation (m)	Label	Elevation (m)	Label	Elevation (m)	Label	Elevation (m)
T30-1	1.82	T60-1	1.63	D1	2.0	T1	1.85
T30-2	1.44	T60-2	1.42	D2	1.79	T2	1.79
T30-3	1.26	T60-3	1.20	D3	1.57	T3	1.49
T30-4	1.08	T60-4	1.03	D4	1.40	T4	1.32
T30-5	0.90	T60-5	0.85	D5	1.22	T5	1.01
T30-6	0.72	T60-6	0.67	D6	1.04	T6	0.98
						T7	0.83
						T8	0.79

The top of the box was lined with 0.10m of Styrofoam insulation to keep the temperature within the box above freezing. This was important as the tensiometers were filled with water and would be destroyed if the water froze. One thermocouple was placed inside the box and one outside of the box, to monitor the temperatures within and outside the box, respectively. These values could then be compared to see how well the insulation worked and to ensure that the inner box temperature remained above zero.

4.3 FIELD DENSITY

As stated in Chapter 3, several of the hydraulic parameters change with density. It was therefore necessary to determine the dry bulk density of the peat in the filter at different depths. Samples of peat were extracted at six different depths in order to determine the dry density. The locations of each of these points are shown on Figure 4.1. Appendix B contains the sampling method used to extract the peat samples and the raw data and calculations obtained throughout the experiment. The density determined at each location and the corresponding elevations are provided in Table 4.2.

Table 4.2 - Field bulk density with labels corresponding to Figure 4.1.

Label	Elevation (m)	Dry Density (kg/m ³)
D1	2.00	130
D2	1.79	150
D3	1.57	140
D4	1.40	160
D5	1.22	180
D6	1.04	170

The peat filter was compacted in 0.15m lifts. Each lift was compacted by people wearing snow shoes walking over the entire surface. Over time the peat will settle due to the weight of the overlying soil. As the overlying weight increases with depth, it would be expected that the density would also increase with depth. From the results it was observed that in general the density did increase with depth in the filter. This trend, however, was not found between the two lowest elevations. However, they are relatively close in magnitude and the differences may be due to sampling errors.

4.4 DATALOGGER

A remote datalogger was used to record the response from each piece of instrumentation installed within the filter. The datalogger used was a Delta-T DL 3000 datalogger which could be programmed to take readings at specified recording intervals. The logger contained twenty-four channels. It was possible to program the logger such that it recorded the proper type of reading for each instrument.

4.5 WATER PRESSURES

Tensiometers (Soilmoisture #2725A) were used to measure the negative water pressures. Transducers (Soilmoisture #5301) attached to the top of the tensiometers returned an output current, ranging between 4 and 20mA (see Figure 4.2). As the datalogger can only accept voltages, the output current was passed through a 62.5Ω resistor (see Figure 4.3). This provided a voltage which could then be recorded by the datalogger. The transducers were powered by a 24V battery source which was stored in a weather proof metal box. Seven of the twelve transducers had a pressure range of 0 to -1bar, and the remaining five had a range of 0.1 to -0.4bar. The pressure range of each transducer is shown on Figure 4.1. The transducers had a specified accuracy of 0.25% of full scale. Therefore the maximum error to be expected on the 1bar range transducers would be $\pm 0.0025\text{bar}$ (0.25cbar) and that of the 0.5bar range transducers would be $\pm 0.00125\text{bar}$ (0.125cbar).

Two separate vertical columns with six tensiometers each were installed (see Figure 4.4). One column had tensiometers with a 0.30m length and the other with 0.60m

Figure 4.2 - Side view of tensiometer and transducer as installed in box

Figure 4.3 - Schematic of the transducer wiring for power and output illustrated.

Figure 4.4 - Side view of tensiometer columns.

lengths. The purpose of having two different tensiometer lengths was to observe whether the assumption of vertical flow was reasonable.

4.5.1 ASSEMBLY AND INSTALLATION OF TENSIO METER

The tensiometer was assembled by initially attaching the porous cup to the tensiometer tube. The pores of the porous cup were then saturated to eliminate any air present. This was done by filling the tensiometer tube with water and letting it sit for at least five minutes. The transducer was then attached to the tensiometer and the tube was filled to the top with water. A vacuum pump was used to draw out any trapped air contained within the tube. It was required to have the tube free of air such that it would not affect the pressure measurement. The top of the tube was then sealed with a reservoir cap. Before installation into the filter, a long metal rod, with a slightly smaller diameter than that of the tensiometer tube, was pushed into the peat to clear a path for the tensiometer. The rod was pushed into a point about 0.05m short of where the porous cup would eventually sit. It was hoped that this would aid in the installation as well as reduce the amount of peat which may tend to compact ahead of the porous cup. The reason for not pushing the rod in the entire way, was to ensure good contact between the porous cup and the peat. Once the tensiometer was installed, significant amounts of air came out of solution from the water in the tube due to its new exposure to negative pressures. To remove this unwanted air, the reservoir cap, which contained water, was pumped to insert more water into the tube. After a few initial pumpings, the tensiometer acclimated itself to its new environment and the rate at which air accumulated was reduced.

Due to the formation of these air bubbles in the tensiometer tube, the suppliers stated that the tensiometers could be placed at a maximum angle of 45° from vertical. At a greater angle, the air bubbles would not float up to the top but collect on the sides of the tensiometer tube, making it more difficult to expel. Hence, this maximum angle of 45° was used. An acrylic gasket was made to secure the tube in place and to seal the hole in the plywood box where the tube was installed (see Figure 4.2). This consisted of a flat plate with a shaft that was sealed tight to the tensiometer tube with an 'O' Ring. The shaft was at a 45° angle from the plate which had four holes through which the gasket could be screwed to the plywood thereby securing the whole system in place.

The negative pressure that existed in the pore water of the peat would come to equilibrium with the water in the pores of the porous cup. This pressure is then transmitted through the water column to the transducer (Point 1 on Figure 4.2). Since the transducer is at a greater elevation than the porous cup, the measured pressure was therefore corrected using a form of Bernoulli equation assuming static conditions.

$$\frac{P_1}{\rho \cdot g} + z_1 = \frac{P_2}{\rho \cdot g} + z_2 \quad (4.1)$$

$$\Psi_2 = \frac{P_2}{\rho_w \cdot g} = \frac{P_1}{\rho_w \cdot g} + (z_1 - z_2) = \Psi_1 + (z_1 - z_2) \quad (4.2)$$

Where Ψ_1 , Ψ_2 , z_1 and z_2 are the pressure heads (expressed in metres of water column) at points 1 and 2 on Figure 4.2, and the elevation heads at points 1 and 2, respectively.

4.5.2 TRANSDUCER CALIBRATION

A calibration curve was required for each transducer to determine the relationship between the measured voltage and pressure head at the transducer. The relationship was known to be linear from the suppliers specifications, therefore the slope and intercept needed to be determined. As the current ranges between 4 and 20mA, the voltage will vary between 250 and 1250mV with a 62.5Ω resistor. This relationship was determined by subjecting the transducer to several pressure heads and monitoring the voltage response. To carry out this exercise the tensiometer and transducer were assembled as previously described. A large bucket was filled with water and used to generate negative pressures. The tip of the tube was placed into the water at a known position (see Figure 4.5). Just outside the porous cup, the pressure head was equal to the vertical distance between it and the water level in the bucket (Ψ_3 on Figure 4.5). Inside the cup, the pressure head was also equal to Ψ_3 as the porous cup comes to equilibrium with the pressure head surrounding it. If the datum is assumed to be at the porous cup, then the elevation head is equal to zero. The elevation head at the transducer (point 4) is the vertical distance between the datum and the transducer (z_4 on Figure 4.5). From equation 4.2, the pressure head at the transducer could be calculated ($\Psi_4 = z_4 - \Psi_3$). The result of this calculation is that the negative pressure head of the transducer was found to equal the vertical distance between the water level and the transducer. This pressure head was measured and the corresponding voltage was recorded. This process was repeated at numerous different pressure heads to obtain the calibration relationship. The whole apparatus was then taken apart, re-assembled and the total procedure was repeated. This

Figure 4.5 - Schematic of arrangement used to calibrate transducers

was done to verify that the previous results were reproducible. According to the transducer specifications, the maximum deviation should be less than 0.0025 of the full scale (16mA).

$$MaxAllowableDeviation = \pm 0.0025 \cdot 16mA \cdot 62.5\Omega = \pm 2.5mV \quad (4.2)$$

The maximum allowable deviation of $\pm 2.5mV$ corresponds to $\pm 0.026m$ of water pressure for the -1bar transducer and $\pm 0.013cm$ for the 0.5bar range transducers. A linear relationship existed between the pressure head and voltage.

$$\Psi(m) = Voltage(mV) \cdot d + b \quad (4.3)$$

Where d and b are the slope and intercept of the calibration line and Ψ is the pressure head of the water phase. The values of the coefficients, d and b , and the maximum deviation across the range tested presented in Table 4.3. The numbers in the first column with an attached asterisks represent those transducers with the 0.1 to -0.4bar range. Some of the transducers, as shown in bold font, were found to have a larger deviation than the specified maximum. As the deviation did not greatly exceed the maximum, the transducers were still used in this exercise.

Table 4.3 - Coefficients of calibrated line for each transducer and maximum deviation

Number	d	b	Maximum deviation (mV)
1	-0.01070	2.821	0.987696
2	-0.00974	2.353	2.693941
3	-0.01049	2.769	1.040122
4	-0.01067	2.805	1.631606
5	-0.01057	2.812	0.753857
6	-0.01038	2.758	4.695387
7*	-0.00626	2.489	5.911079
8*	-0.00625	2.494	4.106187
9*	-0.00629	2.497	5.58792
10	-0.01056	2.742	2.215489
11*	-0.00615	2.465	5.531561
12*	-0.00631	2.492	5.306079

4.5.3 WATER PRESSURE RESULTS

Once the system was pulsed the pressure heads were recorded at five second intervals. The equation for capillary head was shown in Equation 3.8, as the negative difference between the total and elevation head. By definition the total head is the sum of the pressure and elevation head. Re-arranging, this equation in terms of pressure head gives the following.

$$\Psi = h_w - z = -h_c \quad (4.4)$$

Hence, the capillary pressure is equal to the negative water pressure or the capillary head is equal to the negative water pressure head.

An example of how the capillary heads varied with time, for a single pulse of the 0.3m tensiometer column is illustrated in Figure 4.6. Seven pulses in total were examined and the results are presented in Appendix B. The capillary heads at the porous cup

Figure 4.6 - Capillary head versus time from the beginning of a pulse of STE onto the peat filter. The labels assigned to each curve depict the measuring tensiometer

located above the distribution pipe, T30-1, remained fairly constant with a slight amount of variation. The observed variations are all within the $\pm 0.026\text{m}$ accuracy specification for the transducer. The tensiometer located right below the distribution pipe, T30-2, showed a large decrease in the capillary head shortly after the pulse. The capillary head decreased to 0.10m and then slowly increased back towards the initial condition. The rate at which the capillary head rebounds is much slower than the initial decrease. At the next observed elevation (T30-3), the tensiometer exhibited a smaller response which commenced at a slightly later time. This lag in time indicates that it took approximately 5 minutes for the infiltrating water front to move from an elevation of 1.44m (T30-2) to 1.26m (T30-3). The response at T30-3 is similar to the response recorded at T30-2, except that the magnitude of the response was much less (0.22m compared to 0.35m). As saturation is known to increase with decreasing capillary head, these results show that the soil does not become as saturated at the location of T30-3 as the location of T30-2. The transducer located at the next elevation (T30-4), showed a slight decrease in capillary head, which occurred at an even later time. The minimum capillary head recorded at T30-4 was approximately the same as the minimum capillary head recorded at T30-3. The bottom two elevations showed very little change in capillary head with time. As the water table is located near the bottom of the peat, approximately saturated conditions exist at these locations. The bottom transducer exhibited negative capillary heads (i.e. positive water pressure). This is either approaching the limit or is outside of the range depending on which transducer was used. As the error increases towards the limits the values at this location were thought to be suspect.

To determine whether the flow is primarily vertical, the total heads just prior to a pulse are plotted in Figure 4.7. Only the 0.3m and 0.6m tensiometers located within the peat layer below the distribution system are presented. An estimated vertical gradient is also plotted on Figure 4.7. Given the stated accuracy of the transducers ($\pm 0.026\text{m}$) and the total heads measured at the various tensiometer locations, the assumption of vertical flow is reasonable. To further support this argument, the magnitude of the responses measured at tensiometers located at approximately the same elevation can be compared. A comparison of the pulses provided in Appendix B illustrates the magnitude of the responses recorded at T30-3 and T60-3 and at T30-4 and T60-4 are similar. Likewise, the times at which the response begins and the maximum response occurs are similar. This supports the assumption of vertical flow. The response recorded at T30-6 and T60-6 were not considered in the estimation of the equipotential lines as they were thought to be slightly erroneous as discussed previously.

Two graphs, one of the elevation versus pressure head and one of the elevation head versus total head are shown in Figures 4.8 and 4.9, respectively. The line representing time equal to zero, represents the conditions just prior to the system being pulsed with STE. Each of the other lines represents different times after the pulse commenced, as indicated on the legend.

From Section 3.3.1, Darcy's law defined the hydraulic gradient as the driving force causing flow. The hydraulic gradient is represented by the inverse of the slope of the graph of elevation versus total head. A zero hydraulic gradient, would be represented by a vertical line showing no change in total head with elevation. Figure 4.9 shows that a

Figure 4.7 - Total head values at 10 different locations just prior to a pulse. The equipotential lines are illustrated.

Figure 4.8 - Elevation versus pressure head at selected times (datum is at bottom of sand).

Figure 4.9 - Elevation versus total head at selected times (datum is at bottom of sand).

slightly positive slope existed above an elevation of 0.95m prior to the pulse. This indicates a positive gradient and that the system had not yet reached steady state and that some small amount of flow was still occurring when the system was initially pulsed with STE. The gradient between the two bottom elevations showed a negative gradient which would mean that the flow would be upward. However, as previously stated the pressure heads at the bottom elevation were thought to be erroneous. The difference in total heads between the bottom two transducers was approximately 0.02m. This is within the specified error of the transducers.

The first time interval examined after the pulse commenced was at five minutes. Figure 4.8 shows that the pressure head at the elevation of 1.44m (T30-2) increased significantly due to the influx of STE into the system. At each of the other locations there was no change in the measured pressure heads, indicating no change in saturation. This indicates that the water had not moved this far down into the system at this time. From the total head relationship it was evident that the hydraulic gradient had greatly increased at the top of the peat layer forcing the STE to move down through the peat layer.

The second considered time was at ten minutes, and it showed another slight increase in the pressure head at T30-2. There was also a small increase at an elevation of 1.26m (T30-3), indicating that the water had reached this point. No quantifiable changes were found at the other elevations. The hydraulic gradient between T30-2 and T30-3 is approximately the same as that at five minutes as the lines are parallel.

At thirty minutes after the pulse started, the pressure head at the top decreased showing a decline in the water content. The pressure head at T30-3 increased a significant

amount, and a slight increase was observed at T30-4. This showed that the water was slowly being displaced down through the peat. It was noticed that the hydraulic gradient had decreased in the upper 0.20m as indicated by the steeper slope of the elevation versus total head plot.

At fifty minutes, the pressure head at T30-2 continued to decline returning to its initial condition. The pressure head at T30-3 remained approximately the same and the pressure head at T30-4 increased, indicating an increase in saturation at this point.

Over time, the pressure pulse moved through the system indicating that the pore water within the peat is being displaced downwards by the STE pulse at the top of the peat layer. Eventually the pressure heads at each location returned to those conditions which existed prior to pulsing. If the system was left long enough without being pulsed further, it was expected that the system would reach a point of steady state in which no more water would flow and the total head would be constant with depth.

It would be expected that the hydraulic gradient at the bottom of the peat layer would be slightly positive to conduct the flow of water. There was very little if any change in the gradient over the observed time. This error is attributed to transducer error as the pressure heads at this location are too close or exceeded the upper limit.

4.6 TEMPERATURE

A standard Type K thermocouple was used to monitor the temperature within the peat filter at selected depths (See Figure 4.1). The thermocouple consisted of two wires with metals which exhibited very different conductivities. The wires were twisted together

and fused at one end. The wire was twisted around a thin steel rod, with the fused end at least 0.025m beyond the end of the rod, and inserted 0.30m horizontally into the peat. The monitoring end was not allowed to touch the rod as the metal may have affected the readings. The purpose of the rod was to maintain the wire rigid during installation, so that the final position of the thermocouple tip was known. The other end of the thermocouple was then attached to the datalogger, which measured the voltage difference and calculated the temperature.

The temperatures were monitored at two hour intervals during various periods throughout the winter of 1997-1998, commencing in early October. A graph of how these temperatures changed with time is presented in Figure 4.10. The graph shows the temperature profile at five different elevations as indicated. Data for the remaining locations can be found in Appendix B. Initially, the temperatures were very close at all depths with recordings between thirteen and fifteen degrees Celsius. With time and as winter progressed the temperature varied.

Thermocouple, T1, was the only thermocouple situated above the distribution pipe. T1 recorded the largest temperature fluctuations with time and the lowest temperature ($< 1^{\circ}\text{C}$) in comparison to the lower thermocouples. This indicates that the temperature at the uppermost location was affected most by the external temperatures. The other four thermocouples were all below the distribution pipe in areas where STE was flowing through the peat. The STE itself would provide some heat due to the temperature of the waste stream. Also the microbial activity occurring within the peat during treatment would release some energy in the form of heat. In addition, the peat acts as an insulator to

Figure 4.9 - Temperature versus time at different elevations.

reduce the flow of heat. For these reasons it was expected that the temperature would not decrease as much below the distribution pipe as above the distribution pipe during the winter months. This was found to be true and as expected, the temperature increased with decreasing elevation. However, once spring approached this relationship was reversed, as the external temperature increased. This shows that the temperatures within the higher elevations within the peat were more affected by the external conditions. The lower elevations are more insulated due to the overlying peat.

4.7 SUMMARY

The dry density within the filter was found to vary between 129 and 178kg/m³ and generally increased with depth.

The peat filter was instrumented to measure temperatures and pressures. It was found that the system never really reached steady state and that the pressures changed as the water flowed down through the peat. It would be interesting to observe how dry the peat filter would get during the summer months when no STE would be applied. Also, some changes in the hydraulics may occur due to this amount of drying as well as peat decomposition, which should be examined.

At no time did the temperature drop below zero degrees Celsius at any elevation. However, it was a mild winter with very few long cold spells. Those areas within the core of the filter were affected by outdoor temperatures, but still remained above freezing. It was found that in the winter months, the temperature increased with depth. As spring approached, the reverse of this relationship was observed in that the temperatures

decreased with depth. These two observations illustrate the insulating nature of peat to the external environment.

5 UNSATURATED FLOW MODEL DEVELOPMENT

5.1 INTRODUCTION

This chapter covers the development of a one dimensional, unsaturated flow code to be used to model the flow of water through the peat filter. The equations and relationships presented in this chapter were formulated by different researchers. These relationships were used to develop the numerical code which was prepared as part of this research. The model considers hysteretic properties of the pressure-saturation relationship and air entrapment. The code was verified against two existing two-phase air-water models to ensure that the equations were implemented properly. The numerical model was then used with the parameters determined in the lab, to simulate the scenario monitored in the field.

The input parameters required for the model are read from a text file, which can be constructed using any editor (e.g. DOS) (see Appendix C). The data includes grid space and time step data, soil properties, boundary conditions (BCs) and initial conditions (ICs). The model calculates the total head, water saturation and volumetric moisture content for each grid block at each time step.

5.2 DARCY RELATIONSHIP

The Darcy relationship was described in Chapter 3, Section 3.3.1. As the flow through peat is assumed to be slow enough such that laminar conditions exist everywhere, Darcy's law is valid.

5.2.1 UNSATURATED FLOW

There is evidence that Darcy's Law is still valid for unsaturated flow, except that the hydraulic conductivity term in the Darcy equation needs to be modified to account for the reduced water saturation (Bedient et al., 1994). For unsaturated flow, the equation can be written as

$$q = -K(\theta) \frac{\partial h}{\partial x} \quad (5.1)$$

where $K(\theta)$ is the unsaturated hydraulic conductivity.

The unsaturated hydraulic conductivity is then approximated by

$$K(\theta) = K_{sat} \cdot K_{rel}(\theta) \quad (5.2)$$

with $K_{rel}(\theta)$ defined as the relative permeability which varies between zero and one.

5.3 GOVERNING EQUATIONS

The governing equations for a two phase model include equations for both water and air phases. As the air pressure was assumed to be negligible, only the water phase equation needs to be developed.

The equation for flow of the water phase, can be determined by carrying out a mass balance on a control volume. Celia and Binning (1992), took this approach and obtained the following equation:

$$\frac{\partial(\phi \cdot \rho_w \cdot S_w)}{\partial t} + \frac{\partial(\rho_w \cdot q_w)}{\partial y} = S / S \quad (5.3)$$

where ϕ is the porosity, q_w the volumetric flux vector for the water phase and S/S is the source/sink term. This equation is subject to the constraint that the saturation term is less than or equal to one.

$$S_w \leq 1.0 \quad (5.4)$$

5.4 CONSTITUTIVE RELATIONSHIPS

In order to solve the governing equation, several relationships are required. These include a moisture retention relationship, and a method of determining the relative permeability.

5.4.1 TWO-PHASE PRESSURE-SATURATION RELATIONSHIP

The relationship between the pressure and saturation was described in Section 3.6. The van Genuchten relationship was used in the developed model.

5.4.1.1 HYSTERESIS

It has been found by several researchers that neglecting hysteresis in numerical simulations of two-phase flow results in considerable error (Hoa et al., 1977; Gillham et al., 1976; Kool and Parker, 1987; Kaluarachchi and Parker, 1988).

Different approaches have been taken to incorporate hysteresis into numerical solutions. One early method was to include tabulated data (Whisler and Klute, 1965; Staple, 1969; Freeze, 1969). Parker and Lenhard (1987) developed an algorithm to incorporate hysteretic k - S - P relationships. They used van-Genuchten's equation to

describe the main drainage (equation 3.12) and wetting branches (equation 3.11) of the apparent saturation relationship.

The following sections describe the hysteretic model proposed by Parker and Lenhard (1987). Assuming that the system was initially saturated, any increase in capillary head above the air entry pressure will cause water to flow out of the system. The capillary pressure-saturation path will follow the main drainage path. If the capillary head were to subsequently decrease, the process would switch from drainage to wetting and follow the first imbibition scanning curve (see Figure 5.1). The scanning curves are scaled from the main drainage or wetting curve to pass through the reversal points. Consider a wetting scanning curve beginning at $(\overline{S}_{dw}, h_{dw})$, corresponding to the most recent reversal from drainage to wetting, and ending at $(\overline{S}_{wd}, h_{wd})$, corresponding to the reversal from wetting to drainage on the preceding drainage curve. The equation of the scaled wetting branch is given by

$$\overline{S}_{w,w}(h_c) = \frac{\left\{ \left(\overline{S}_{w,w}(h_c) - \overline{S}_{w,w}(h_{wd}) \right) \cdot \left(\overline{S}_{dw} - \overline{S}_{wd} \right) \right\}}{\left[\overline{S}_{w,w}(h_{dw}) - \overline{S}_{w,w}(h_{wd}) \right]} + \overline{S}_{wd} \quad (5.5)$$

where $\overline{S}_{w,w}(h_c)$ is the apparent saturation for the wetting scanning curve and $\overline{S}_{w,w}(h_{wd})$ and $\overline{S}_{w,w}(h_{dw})$ are the values of $\overline{S}_{w,w}$ at $h_c = h_{wd}$ and $h_c = h_{dw}$ respectively. Figure 5.1, shows a system initially following the main drainage curve (Path 1→2), which is described by equation 3.12. The system is subsequently subjected to a process reversal to a wetting

Figure 5.1 - Pressure-saturation relationship showing scanning curves.

curve (Path 2→3) which is described by the scaling equation (equation 5.5), with $(\overline{\overline{S_{dw}}}, h_{dw})$ and $(\overline{\overline{S_{wd}}}, h_{wd})$ corresponding to points 2 and 1, respectively.

A similar approach was taken to describe the drainage scanning path beginning at point $(\overline{\overline{S_{wd}}}, h_{wd})$, corresponding to the most recent reversal from wetting to drainage and ending at $(\overline{\overline{S_{dw}}}, h_{dw})$, corresponding to the reversal from drainage to wetting on the preceding wetting curve. The equation becomes

$$\overline{\overline{S_{w,d}}}(h_c) = \frac{\left\{ \left(\overline{\overline{S_{w,d}}}(h_c) - \overline{\overline{S_{w,d}}}(h_{dw}) \right) \cdot \left(\overline{\overline{S_{wd}}} - \overline{\overline{S_{dw}}} \right) \right\}}{\left[\overline{\overline{S_{w,d}}}(h_{wd}) - \overline{\overline{S_{w,d}}}(h_{dw}) \right]} + \overline{\overline{S_{dw}}} \quad (5.6)$$

where $\overline{\overline{S_{w,d}}}(h_c)$ is the apparent saturation for the drainage scanning path and $\overline{\overline{S_{w,d}}}(h_{dw})$ and $\overline{\overline{S_{w,d}}}(h_{wd})$ are the values of $\overline{\overline{S_{w,d}}}$ at $h_c = h_{dw}$ and $h_c = h_{wd}$ respectively. In Figure 5.1, another process reversal from wetting to drainage occurs at point 3. Path 3→4, is described by equation 5.6 with $(\overline{\overline{S_{dw}}}, h_{dw})$ and $(\overline{\overline{S_{wd}}}, h_{wd})$ corresponding to points 2 and 3 respectively.

5.4.1.2 AIR ENTRAPMENT

The apparent water saturation includes the entrapped air saturation (see Figure 5.2). To approximate the effective saturation from the apparent saturation, an algorithm developed by C.S. Land (1968) was employed (Lenhard et al. 1991). The algorithm determines the maximum amount of air that can be trapped in the pores after a saturation

Figure 5.2 - Effective pressure-saturation curve showing effects of air entrapment

reversal from the main drainage path. The relationship developed by Land (1968) is as follows.

$$\overline{S}_{nr} = \frac{1 - {}^{\Delta}\overline{S}_w}{1 + R_{nw} \cdot (1 - {}^{\Delta}\overline{S}_w)} \quad (5.7)$$

Where

$$R_{nw} = \frac{1}{{}_i\overline{S}_{nr}} - 1 \quad (5.8)$$

and ${}^{\Delta}\overline{S}_w$ is the effective water saturation at the first reversal from the main drainage path.

$\overline{S}_{nr}$ and ${}_i\overline{S}_{nr}$ are the amounts of trapped air for the initial wetting scanning path beginning at ${}^{\Delta}\overline{S}_w$ and that of the main wetting curve, respectively (see Figure 5.2). To determine any intermediary entrapped saturations between the reversal point and a capillary pressure of zero (an apparent saturation of 1.0) linear interpolation is conducted.

$$\overline{S}_{nt} = \overline{S}_{nr} \cdot \left(\frac{\overline{S}_w - {}^{\Delta}\overline{S}_w}{1 - {}^{\Delta}\overline{S}_w} \right) \quad (5.9)$$

This amount of trapped air is subtracted from the apparent saturation to determine the effective saturation. The only new piece of information required for this algorithm is the maximum amount of trapped air, ${}_i\overline{S}_{nr}$.

5.4.2 RELATIVE PERMEABILITY

It has been found that there is no universal relationship which is suitable for evaluating the relative permeability for all soils (Mualem, 1978). Even soils of the same

class can have significantly different relationships from each other. Therefore, Mualem (1976) derived a relationship for the relative permeability from knowledge of the soil-water retention curve.

$$K_{rel}(\Theta) = \sqrt{\Theta} \cdot \left[\frac{\int_0^\Theta \frac{1}{\Psi(x)} dx}{\int_0^1 \frac{1}{\Psi(x)} dx} \right]^2 \quad (5.10)$$

Van Genuchten (1980) used Mualem's formulation to develop a useful relationship for determining the relative permeability, again assuming that $m = 1 - \frac{1}{n}$.

$$K_{rel}(\Theta) = \sqrt{\Theta} \cdot \left[1 - (1 - \Theta^{1/m})^m \right]^2 \quad (0 < m < 1) \quad (5.11)$$

5.5 NUMERICAL SOLUTION

To solve the governing equation numerically, the finite difference approximation was used. The assumptions used in coding the model were constant grid size, time step and temperature. The constant temperature results in a constant water density. The datum for the elevation head is the base of the modeled column.

5.5.1 FINITE DIFFERENCE APPROXIMATION

A control volume approach was taken to determine the finite difference representation of the governing equations for a one-dimensional vertical column. The grid block layout is shown in Figure 5.3.

Figure 5.3 - Grid layout of column

The control volume approach involves performing a mass balance for each grid block as follows. Consider block j for this formulation.

$$Accumulation = mass_{in} - mass_{out} \quad (5.12)$$

The accumulation term is as follows:

$$Accumulation = \left(\frac{\phi_j \cdot \rho_w}{\Delta t} \cdot (S_{w,j}^{t+\Delta t} - S_{w,j}^t) \right) \cdot \Delta x \cdot \Delta y \cdot \Delta z \quad (5.13)$$

where Δt is the size of time step, Δx , Δy and Δz represent the size of grid blocks in the x , y and z directions. For a one dimensional vertical column in the y direction, Δx and Δz are

assumed to equal unity and Δy is the size of the grid blocks in y (vertical) direction which is assumed constant. The porosity of the j th block is ϕ_j , and $S_{wj}^{t+\Delta t}$ and S_{wj}^t are the water saturations of the j th block at times $t+\Delta t$ and t respectively.

The mass flux terms are a function of the flux across the interfaces. As previously stated, the flux is defined by the Darcy equation for unsaturated flow (equation 5.1). The equations for the mass of water in and out are as follows:

$$mass_{in} = \rho_w \cdot K(\theta)_{j-1/2} \cdot \left(\frac{h_{wj-1} - h_{wj}}{\Delta y} \right) \cdot \Delta x \cdot \Delta z \quad (5.14)$$

$$mass_{out} = \rho_w \cdot K(\theta)_{j+1/2} \cdot \left(\frac{h_{wj} - h_{wj+1}}{\Delta y} \right) \cdot \Delta x \cdot \Delta z \quad (5.15)$$

where $K(\theta)_{j\pm 1/2}$ is the unsaturated hydraulic conductivity, with the indices $j+1/2$ and $j-1/2$ indicating the face through which the flow occurs (see Figure 5.3). The h_w terms indicate the total heads with the subscripts $j-1$, j and $j+1$ referencing the grid block.

To determine the midblock hydraulic conductivity (at $j+1/2$ and $j-1/2$) the values of the surrounding blocks ($j-1$, j and $j+1$) were used. Haverkamp and Vauclin (1979) investigated different methods of calculating the midblock values and the error associated with each. They found that the geometric mean of the surrounding blocks resulted in the minimal amount of error.

$$K(\theta)_{j+1/2} = (K(\theta)_j \cdot K(\theta)_{j+1})^{1/2} \quad (5.16)$$

5.5.2 BOUNDARY CONDITIONS AND INITIAL CONDITIONS

In order to solve the governing system of equations for one-dimensional unsaturated flow, the initial and boundary conditions must be defined. The initial total heads within the column must be specified.

$$h_w(y) = h_{w,j} \quad \text{for all } y, t=0 \quad (5.17)$$

A value of the total head was required for each grid block to simulate the initial conditions.

Boundary conditions (BCs) are required at the first node (node 1) and the last node (node n) for all time steps. The model will accommodate two types of boundary conditions: Type 1 BC or Dirichlet BC (specified total head) and/or Type 2 BC or Neuman BC (specified flux). The user of the model decides which type of BC (Type 1 or Type 2) is desired at each of these nodes as well as the value of the BC. However, at least one end must have a Type 1 BC. The following equations are the required BCs for the model.

$$h_w = h_{w,1} \quad \text{or} \quad q = q_1 \quad \text{for } y=0, t>0 \quad (5.18)$$

$$h_w = h_{w,n} \quad \text{or} \quad q = q_L \quad \text{for } y=L, t>0 \quad (5.19)$$

If any other source or sink terms exist at any mid-block locations, they can be incorporated as well. The model can also accommodate several “stress periods”. A stress period is a time in which the type and value of each BC remains constant. If the type or value of a BC changes, then the model enters into a new stress period.

For a Type 1 BC, the value of the head at the grid block is known and this known value is incorporated into the equation for the adjacent grid block. A Type 2 BC was

incorporated into the formulation as a S/S term for the grid block. If the value of the flux is positive it is considered as a source term, whereas if the flux is negative it is treated as a sink term.

5.5.3 METHOD OF SOLVING NON-LINEAR EQUATIONS

Due to the non-linearity of the finite difference equation, a numerical method for solving the system of equations was required. In this code, the Newton-Raphson technique was used to solve the set of non-linear equations. The tri-diagonal matrix formed was solved numerically using the Thomas algorithm.

5.5.4 CODE VERIFICATION

To verify that the equations were properly incorporated in the numerical model, the results of the model were compared with model results presented by several different researchers. Two different cases were used for verification of the model used in this research.

5.5.4.1 CASE 1 - TWO-PHASE INFILTRATION

Touma and Vauclin (1986) conducted experiments investigating two-phase (water-air) infiltration. They set up several columns and monitored how a constant flux infiltrated into the partially saturated soil. They then modeled their results using a finite difference, one dimensional, two-phase flow code. These model results have also been

used by Celia and Binning (1992) and Van Vliet (1993) to verify that their respective codes functioned as expected.

Touma and Vauclin (1986) used the relationships derived by van Genuchten to describe the saturation (equation 3.9) and relative permeability (equation 5.11) of the soil used in the column. The van Genuchten parameters, θ_{ws} , θ_r , α and n were found to have values of 0.312, 0.0265, 4.4m^{-1} and 2.2 respectively. The saturated hydraulic conductivity of the soil was $4.28 \times 10^{-5}\text{m/s}$.

The column used for the experiments was 1.00m high, with the air phase being able to escape easily from both the top and bottom of the column. A constant water flux of $2.31 \times 10^{-5}\text{m/s}$ was applied to the top of the column and a constant pressure head of -0.099m was maintained on the bottom of the column. Touma and Vauclin (1986) modeled the system using the above parameters to validate their code. Van Vliet (1993) used these results for verification of a code using a grid size of 0.02m and a time step size of 1s.

The results of Touma and Vauclin (1986) are presented in Figure 5.4 and those of the current model are presented in Figure 5.5. The current model results and that of Touma and Vauclin (1986) seem to agree fairly well giving some indication that the equations were implemented correctly.

5.5.4.2 CASE 2 - VARIABLE WATER TABLE

Lenhard et al. (1991) reported results from a one-dimensional, water-air flow experiment. They instrumented a 0.72m sand column with porous cups and tensiometers

Fig. 8. Infiltration at $q_e(0, t) = 8.3 \text{ cm/h}$: time evolution of water content profiles. Horizontal bars correspond to uncertainties associated with the measurements.

Figure 5.4 - Model results of Touma and Vauclin (1986).

Figure 5.5 - Model results of infiltrating front using parameters given by Touma and Vauclin (1986)

with an accuracy of $\pm 0.02625\text{m}$ of water pressure. They used these instruments to monitor the water pressures and saturations at elevations of 0.70, 0.60, 0.50, 0.40, 0.30 and 0.20m from the bottom of the sand column. Initially, the column was allowed to saturate by placing a pressure head of 0.72m at the base of the column. The pressure head at the bottom was then lowered and raised according to Figure 5.6. The upper boundary remained as a no flow boundary throughout the experiment.

The laboratory results were modeled by Kaluarachchi and Parker (1989) using a multiphase flow code and Van Geel (1994) used the results of this model to verify another multiphase flow code. A grid space of 1 cm was used to discretize the column. The van Genuchten parameters, α_d , α_w , n , θ_r and maximum air entrapment were 4.2m^{-1} , 8.2m^{-1} , 5.25, 0.0612 and 0.25 respectively. The porosity and saturated hydraulic conductivity were 0.36 and $3.31 \times 10^{-4}\text{m/s}$ respectively.

Figure 5.6 - Water pressure at base of column for verification with variable water table case

The laboratory and calibrated model results obtained by Lenhard et al. (1991), for both the nonhysteretic and hysteretic models can be found in Figure 5.7. The results from the model developed for this research, for both nonhysteretic and hysteretic runs, are presented in Figure 5.8 and Figure 5.9, respectively. The similarity of the results for both cases indicate that the equations have been coded properly. The combination of the verification with the two separate cases confirms that the equations in the code have been implemented correctly.

5.6 SUMMARY

The code developed in this chapter has been found to function reasonably in comparison with other existing models. This code will be used to model the flow through the peat filter. The model will be run using the hydraulic parameters of peat determined in the laboratory study. The boundary conditions and soil parameters will be varied in a trial and error manner in attempts to match the model runs with the field data.

Fig. 4. Comparison of measured (solid symbols) and simulated full hysteretic (solid curves), fluid entrapment only (broken curves with equal-sized segments), and nonhysteretic (broken curves with alternating long and short segments) water saturation distributions at measurement locations of (a) 70 cm, (b) 60 cm, (c) 50 cm, (d) 40 cm, and (e) 30 cm.

Figure 5.7 - Model results of Lenhard et al. (1991) for the hysteretic and non hysteretic solutions

Figure 5.8 - Model results for verification with non-hysteretic solution of Case 2

Figure 5.9 - Model results for verification with hysteretic solution of Case 2

6 COMPARISON OF MODEL RESULTS AND FIELD DATA

The peat filter was modeled using the one dimensional unsaturated flow code developed in Chapter 5. By considering only one dimension, it was assumed that the flows in the horizontal directions were negligible. The model was run under two different scenarios. The first scenario was to model the entire filter from the bottom of the sand layer to the very top of the peat. This scenario was used to simulate a pulse of STE to the filter and to simulate steady state conditions. The second scenario simulated only the peat medium between the bottom of the distribution pipe and the top of the gravel layer below the peat.

6.1 COMPLETE FILTER MODELING

6.1.1 CONCEPTUAL MODEL

The entire filter from the bottom of the sand to the top of the peat was modeled. Two STE pulses were simulated. The filter was broken down into a vertical column and parameters determined in the laboratory study were assigned to each grid block.

6.1.1.1 GRID AND TIME DISCRETIZATION

The height of the peat filter was 2.1m from the bottom of sand to the top of the peat. The filter was broken down into a vertical column with 105 grid blocks each with a height of 0.02m as shown in Figure 6.1.

Figure 6.1 - Grid discretization, soil type numbers and boundary conditions used in model

As this is a transient case, the time component needed to be considered. From the observed frequency of pulsing, an average time of ninety minutes, between the beginning of one pulse to the beginning of the next pulse, was assumed. The model was run with 37600 time steps, and a time step size of 0.3 seconds. This corresponds to a total time of 188 minutes.

6.1.1.2 PARAMETER ASSIGNMENT

The model required the user to input the hydraulic parameters for each grid block in the vertical column. The filter was divided into several different layers based on the multiple types of media present (see Figure 6.1). The peat thickness itself was divided into four separate layers based on the dry bulk densities determined in the field. It was assumed that the density and hydraulic parameters were constant over the thickness of each particular layer. The initial hydraulic parameters assigned to the peat were derived from the relationships determined in the laboratory study.

Each of the four peat layers was assigned a dry bulk density based on the relationship found between dry bulk density and elevation presented in Chapter 4 (see Table 6.1). The values found in the first column of Table 6.1 correspond to each layer as shown in Figure 6.1. From the dry density, the saturated hydraulic conductivity was calculated using the equation presented in Chapter 3. The effective porosity of each layer was estimated using the relationship determined in Chapter 3. The pressure-saturation parameters were determined in Chapter 3, Section 3.6.4. The van Genuchten values for each of the four peat layers were assumed to be the same. The values of α_d , α_w , n , S_r and maximum allowed air entrapment, were initially assumed to be 8.016m^{-1} , 20m^{-1} , 2.2, 0.27

and 0.139 respectively. These values were purely a starting point for running the model and were varied in an attempt to calibrate the model.

Table 6.1 - Dry bulk density and hydraulic properties assigned to each peat layer

#	Elevation (m)	ρ_d (kg/m ³)	K_{sat} (m/s)	K_{sat} (cm/min)	Effective porosity
1	2.00	124	0.000526	3.155	0.852
2	1.70	140	0.000188	1.128	0.840
4	1.23	166	0.000038	0.225	0.819
5	0.85	187	0.000010	0.0612	0.803

The sand and two gravel layers were assigned typical hydraulic values obtained from the literature. The values initially assigned to each layer are shown in Table 6.2.

Table 6.2 - Initial hydraulic parameters assigned to each layer.

Soil #	K_{sat} (cm/min)	$\phi_{effective}$	α_d (m ⁻¹)	α_w (m ⁻¹)	n	θ_r	Maximum trapped air
1	3.3155	0.852	7.9111	20.697	2.14823	0.037	0.139
2	1.128	0.840	7.9111	20.697	2.14823	0.037	0.139
3	60	0.3	8	16	4	0.018	0.01
4	0.225	0.819	7.9111	20.697	2.14823	0.037	0.139
5	0.0612	0.803	7.9111	20.697	2.14823	0.037	0.139
6	60	0.3	8	16	4	0.018	0.01
7	0.6	0.3	4	12	3	0.03	0.05

6.1.1.3 BOUNDARY AND INITIAL CONDITIONS

Boundary conditions were required at each extremity of the column with at least one of them being a type 1 BC. A no flow boundary was established at the top of the column.

A sloped clay barrier layer existed below the peat filter. As the clay is a material with a relatively low hydraulic conductivity, water will pool on top of the clay and drain towards the end of the filter where the treated STE is allowed to infiltrate into the native

soil. For the water to flow, a small mound forms resulting in a positive pressure at the bottom of the sand. Therefore, the bottom grid block was assigned a constant total head of 0.05m.

From Chapter 4, it was known that the system was pulsed on average every hour and a half during school hours. It took approximately four to eight minutes for the entire volume to be pumped onto the filter. Each pulse consisted of 1.38m^3 . This volume was divided by the area of the filter (112.36m^2) to determine an average discharge of 0.0123m/dose per unit area of filter. A flow rate of $5.12 \times 10^{-5}\text{m/s}$ to $2.56 \times 10^{-5}\text{m/s}$ was determined by dividing the discharge (0.0123m/dose) by the time over which pulse occurred (4 to 8min). The method of incorporating this into the model was to inject this flow into the column for four to eight minutes at the location of the distribution pipe. The model was run for an additional 86 minutes to simulate how the water flows down through the filter. Another pulse was then initiated for four to eight minutes, and the model was run for a final 90 minutes with no further injection of STE.

The initial heads at each grid block needed to be specified for the model. The initial conditions were derived from one of the field data sets at a time just before the system was pulsed.

6.1.2 STEADY STATE SOLUTION

Before simulating the field results, the model was run to simulate the steady state conditions which exist during static water pressures. The pressure-saturation relationship is a function of soil type. The pressure-saturation relationship for the peat, sand and gravel are provided in Figure 6.2. In this case the water table was set at the bottom of the

Figure 6.2 - Pressure-saturation relationship for the peat, gravel and sand mediums. The fourth graph, entitled all, shows the relationship for the filter configuration.

column. At the base of the filter the saturations initially follow the pressure-saturation relationship of the sand. At the sand/gravel interface the saturation abruptly changes as a result of the differing pressure-saturation relationship. A similar change occurs at the gravel/peat interface. It was expected that the steady state solution simulated by the model should follow this pattern.

To run the steady state solution, the model was run for a sufficiently long time with no input of STE until there were no further changes in total head. The results of this exercise are shown in Figure 6.3.

6.1.3 MODEL RUNS

The model was initially run with the aforementioned parameters, which were then varied to improve the model results. All of the hydraulic parameters, boundary conditions and initial conditions were considered in this process. Approximately one hundred and twenty model runs were carried out under different conditions.

The results of one of the model runs is shown in Figure 6.4. Investigating the total heads directly below the distribution pipe shows a total head change of 0.20m. The field results show a larger change of 0.30m at this location.

At the base of the peat, the field data is suspect as it shows a positive gradient at the bottom. It was not possible to generate model results which match the field data at this location.

In the gravel and sand layers, no field data was available for comparison with the model results. There were also uncertainties related to the sand and gravel layers as the

Figure 6.3 - Steady state saturations for whole peat layer.

Figure 6.4 - Model results for whole filter simulation.

hydraulic parameters were not determined in the laboratory study. The values were estimated according to typical values, but the actual values may be very different. The boundary condition at the lower end of the model column was not known and was assumed. It was also difficult for the model to handle the large contrast in the parameter values.

From the uncertainties associated with the sand and gravel layers, it was difficult to match the model and field results. To simplify the modeling exercise a smaller portion of the system was considered. The next section shows the modeling exercise of the peat medium between the distribution pipe down to the top of the gravel layer.

6.2 MODELING EXERCISE OF PEAT LAYER ONLY

6.2.1 CONCEPTUAL MODEL

The peat layer was initially modeled as one soil type, meaning that the peat was assumed to have consistent values over its entire depth. The simulations under these conditions were beneficial as they gave an indication of how the different parameters affected the model results. To improve the model predictions and to better reflect the varying conditions with depth seen in the field, it was decided to break the peat up into five different layers of equal thickness.

6.2.1.1 GRID AND TIME DISCRETIZATION

The column was divided into 45 grid blocks, each with a height of 0.02m (see Figure 6.5). This provided the 0.90m vertical column height, which was the required thickness of the peat.

The time step size remained the same as the previous scenario. In this case, only one pulse of STE was pumped into the system.

6.2.1.2 PARAMETER ASSIGNMENT

Each of the layers had a uniform thickness as shown in Figure 6.5. They were assigned initial values based on the approximate density that would exist at the midpoint of the layer as indicated by an asterisks on Figure 6.5 (Table 6.3). The effective porosity was approximated for each layer with it decreasing with depth.

Table 6.3 - Assignment of hydraulic parameters for peat only modeling exercise.

#	ρ_d (kg/m ³)	K_{sat} (cm/min)	ϕ_e	α_d (m ⁻¹)	α_w (m ⁻¹)	n	θ_r	Max. air
1	155	0.4319	0.8646	7.91	20.697	2.148	0.17	0.203
2	165	0.233	0.8639	7.91	20.697	2.148	0.17	0.203
3	175	0.1257	0.8625	7.91	20.697	2.148	0.17	0.203
4	185	0.0678	0.8612	7.91	20.697	2.148	0.17	0.203
5	195	0.0366	0.8599	7.91	20.697	2.148	0.17	0.203

6.2.1.3 BOUNDARY AND INITIAL CONDITIONS

The top of the model column corresponds to the top of the peat layer immediately below the distribution pipe, which was where the STE discharged to the system. A type II

Figure 6.5 - Grid discretization for peat only modeling

BC was placed at the top grid block to simulate this condition and the flux was set equal to the flow rate determined for the previous scenario.

The boundary condition for the bottom grid block was assigned a constant total head equal to -0.24m.

The initial conditions for this case were taken from the data of Pulse 2. The total heads and elevations were adjusted to a datum corresponding to the base of the peat.

6.2.2 MODEL RESULTS

The model results for the scenario in which the peat was considered as a uniform soil layer are presented in Appendix D and are not discussed here.

Using the scenario in which the peat parameters varied with depth, the hydraulic parameters were varied using a trial and error approach. This modeling scenario gave much better results and it appeared that separating the peat into five layers significantly improved matters. The model was run numerous times and the results are presented in Appendix D.

Figure 6.6 shows three plots of elevation versus total head (with the Datum at the base of the peat layer). The first plot is the model results from the initial parameter assignment. The second is the results from the best model run in comparison with the field data. The last plot is generated from data found in the field study. The hydraulic parameters initially assigned to the grid and those of the best model run are shown in Table 6.4.

Considering the top tensiometer location, it was found that the initial model run took a longer time to respond to the influx of STE than that observed in the field. It was

Figure 6.6 - Initial and best model results and field data from Pulse 2 (datum at base of peat)

also found that the magnitude of the response was much lower (0.55m compared to 0.75m). The best model simulation showed a similar response to that found in the field. The response occurred at the same time and to approximately the same magnitude.

Table 6.4 - Hydraulic parameters assigned initially compared with those assigned to the best model results.

#	Parameter	Initial	Model	Difference
1	K_{sat} (cm/min)	0.4319	0.55	0.1181
	ϕ_c	0.8646	0.8646	0
	α_d (m ⁻¹)	7.91	2.91	-5
	α_w (cm ⁻¹)	20.697	24.697	4
	n	2.148	1.548	-0.6
	θ_r	0.17	0.3	0.13
	max. air	0.203	0.303	0.1
2	K_{sat} (cm/min)	0.233	0.527	0.294
	ϕ_c	0.8639	0.8639	0
	α_d (m ⁻¹)	7.91	7.91	0
	α_w (m ⁻¹)	20.697	24.697	4
	n	2.148	1.948	-0.2
	θ_r	0.17	0.3	0.13
	max. air	0.203	0.303	0.1
3	K_{sat} (cm/min)	0.1257	0.227	0.1013
	ϕ_c	0.8625	0.8625	0
	α_d (m ⁻¹)	7.91	7.91	0
	α_w (m ⁻¹)	20.697	20.691	0
	n	2.148	1.948	-0.2
	θ_r	0.17	0.27	0.1
	max. air	0.203	0.403	0.2
4	K_{sat} (cm/min)	0.0678	0.2266	0.1588
	ϕ_c	0.8612	0.8612	0
	α_d (m ⁻¹)	7.91	7.91	0
	α_w (m ⁻¹)	20.697	20.697	0
	n	2.148	1.948	-0.2
	θ_r	0.17	0.27	0.1
	max. air	0.203	0.403	0.2
5	K_{sat} (cm/min)	0.0366	0.01	-0.0266
	ϕ_c	0.8599	0.8	-0.0599
	α_d (m ⁻¹)	7.91	9.91	2
	α_w (m ⁻¹)	20.697	20.697	0
	n	2.148	1.948	-0.2
	θ_r	0.27	0.27	0
	max. air	0.203	0.403	0.2
	BC - top grid	5.1×10^{-5} m/s over 4 minutes	2.55×10^{-5} m/s over 8 minutes	same volume - over different time
	BC - bottom grid	-0.24m	-0.24m	no change

The second tensiometer location (T30-3) showed little response in the initial model run. The best model results does show a response, with a comparable magnitude to that found in the field. The times of the total head changes are approximately the same between the best model run and the field pulse.

The third tensiometer location (T30-4) showed no response for the initial model run and a slight response for the best model run. The field data does show a little response with a change in total head of approximately 0.2m.

The fourth tensiometer location (T30-5) shows little to no response in any of the three cases.

The bottom tensiometer location (T30-6) shows noticeable changes in the total head for both the model runs. The initial condition for the model runs was generated from the field data. As the field data showed a slightly positive gradient at the bottom, this was input to the model. As this was known to be unrealistic, the model corrected for this and therefore significant changes were found.

The values of the parameters for each soil layer and the boundary conditions are shown in Table 6.4 for the initial and best model. From this table, it was possible to examine the differences between those values assigned initially to the model with those assigned to the best model. The saturated hydraulic conductivities were increased slightly for the four top layers. This effectively reduces the hydraulic gradient required to force a certain volume of water through the peat. The values of the conductivity were within the range found in the laboratory study.

The effective porosities were not changed except in the bottom layer. It was observed that the porosity did not have a great effect on the model results.

The pressure-saturation relationship was very important to the model. A pulse of STE was added to the model at the beginning of the run. It was expected that this would result in a change in the saturation which would correspond to a change in pressure. The relationship between the two is depicted by the pressure-saturation curve. The van Genuchten parameters were changed between the initial and best model. By changing the pressure-saturation relationships, the change in pressure head measured in the field could be simulated with the model. With the influx of STE, the magnitude of the change in capillary head was a function of the pressure-saturation relationship and the speed at which the water was displaced in the vertical direction. By using the van Genuchten parameters of the initial and best model for layer 1, this change in saturation could be examined. A plot of the two pressure-saturation relationships is shown in Figure 6.7. From Figure 4.6, the capillary head at the tensiometer located below the drainage pipe (T30-2) dropped from 0.44m to 0.08m. For the pressure response measured in the field, the initial model parameters indicate a 25% increase in saturation in comparison to the 10% increase given using the best fit parameters. A 25% increase in saturation was not required to accommodate the flow and hence the initial model simulation underestimated the pressure change. To generate the pressure change measured in the field the pressure-saturation was altered to the values of the best fit model.

The values of the irreducible moisture content and the maximum amount of trapped air were changed significant amounts in each of the layers. The residual moisture content should have little impact on the final results. However, the maximum amount of trapped air would alter the saturations and hence the flow rates. As these values were variable in the laboratory study, these changes were found to be reasonable.

Figure 6.7 - Comparison of initial and best model pressure-saturation curves for Layer 1

The mean error (ME), mean absolute error (MAE) and root mean square (rms) were evaluated for each model run. The presented model results had a ME, MAE and rms values of 1.4843, 1.9877 and 3.0654, respectively. These values are very high, however most of the error was found to exist at the bottom of the column as previously described. If these points were not considered in the error calculation, the values were decreased to much smaller values of 0.02949, 0.9241 and 1.1319 for the ME, MAE and rms, respectively. This reduces the error by more than half demonstrating that the bulk of the error occurred at the bottom of the column.

7 CONCLUSIONS

A literature review indicated that peat has good contaminant removal properties that make it a viable filter medium for septic systems. However, concerns have been raised with regards to its hydraulic properties and the potential for clogging. This study focused on the flow properties of peat in an attempt to aid in the design of peat filter systems.

A laboratory study showed that the saturated hydraulic conductivity of sphagnum peat varied between 4.0×10^{-5} and 7.9×10^{-4} m/s over a range of dry density between 160 and 120 kg/m³. A linear relationship existed between dry density and the log of the hydraulic conductivity. These results indicate that the saturated hydraulic conductivity is very sensitive to the level of compaction of the peat. Hence, the compaction of the peat is an important consideration for peat filter design. For the peat filter studied as part of this research, the dry density was found to range between 130 and 180 kg/m³ using snow shows to compact the peat. During the initial two years of operation, no signs of clogging were observed and the hydraulic properties were sufficient to handle the design flow. Compaction of peat by heavy equipment will result in a lower hydraulic conductivity which may result in a peat filter that is unable to handle the design flow. In addition, a lower hydraulic conductivity may increase the potential for clogging to occur. The dry density in the peat filter should be high enough to allow the required flow to occur, but low enough so that a reasonable hydraulic retention time is maintained.

The total porosity was found to range between 87 and 92% which is consistent with published values. However, the effective porosity ranged between 61 and 97% which is higher than published values. A high effective porosity indicates that a greater surface area will be in contact with the STE as it flows through the filter. This may increase the removal efficiency of the filter. The higher effective porosity may be the result of the pretreatment process of drying and milling the peat before it was placed in the filter.

The pressure-saturation relationship determined for the sphagnum peat was comparable with that found in the literature. The relationship was not found to have any significant relationship with density. The van Genuchten parameters were found to be approximately 8m^{-1} , 20m^{-1} and 2.2 for α_d , α_w and n , respectively. More variability was found for the residual moisture content and amount of trapped air as they ranged between 0.1598 to 0.3338 and 0 to 13.9%, respectively. The hysteretic properties of the pressure-saturation relationship were found to be considerable and should not be neglected.

The field study provided valuable field data that could be used to validate the conceptual and numerical models developed as part of this study. The response of the water pressures to the STE pulse clearly illustrated the movement of the STE vertically through the peat filter. The measured water pressures also indicated that the peat filter never reached steady state conditions between successive pulses. The hydraulic retention time of the filter was estimated to be approximately 2.5 days.

A one dimensional hysteretic model, developed as part of this study, was able to simulate the field conditions using hydraulic properties in the range determined in the

laboratory. Two simulations were performed to illustrate the importance of the density dependent saturated hydraulic conductivity. One simulation assumed that the peat filter had a constant hydraulic conductivity with depth and a second simulation assumed that the hydraulic conductivity varied with depth according to the relationship determined in the laboratory study. The variable density simulation more accurately reflected the water pressures measured in the field in response to a STE pulse. In addition, the model results indicated that the simulated water pressures were sensitive to the capillary pressures-saturation relationship.

Thermocouples were used to monitor the temperatures within the peat filter. The temperatures were recorded over an eight month period which included the winter season. The temperatures were found to vary with depth in the filter and with time. The uppermost thermocouple measured the largest temperature variation in response to the outside air temperature. However, the temperature never fell below zero during the eight month period. As expected, the peat acted as an insulator which restricted the flow of heat into and out of the peat filter. This prevented the filter from freezing during the winter. In addition, the temperature of the STE entering the filter and any microbial degradation occurring within the peat would help in maintaining the positive temperatures. The temperature within the filter will be an important parameter when evaluating the contaminant removal mechanisms.

Further research is required to determine any long term changes which may affect the hydraulic parameters of the peat. For example, during the summer months, the peat

filter will receive considerably less STE and this may cause the peat itself to degrade, which may reduce the hydraulic conductivity over time.

The hydraulic study performed as part of this research will form the basis for a contaminant transport study to identify the removal mechanisms within the peat filter.

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APPENDIX A - LABORATORY METHODOLOGY AND DATA

A.1 - MOISTURE, ASH AND ORGANIC CONTENT

The ambient moisture, ash and organic contents were determined using the procedure outlined in ASTM D2974-87, with a few alterations. The peat was laid out, in a thin layer in a pan and allowed to dry to ambient conditions for at least twenty-four hours. As described in Chapter 3, two different sample types were examined: “Whole Sample”; and “Partial Sample”. The method for determining the moisture content remained the same as the ASTM standard, however, the procedure for the ash and organic contents were changed slightly. The moisture content was determined by drying a known mass of peat and calculating the moisture content from the following equation.

$$\text{MoistureContent} = \frac{m_w}{m_s} \cdot 100\% = \frac{m_a - m_d}{m_d} \cdot 100\% \quad (\text{A.1})$$

Where m_w , m_s , m_a and m_d are the water, soil, ambient peat and dry masses, respectively. The next stage was to burn the sample at 440°C in a muffle furnace to remove the organic portion of the sample. Problems arose during this combustion phase as the sample was lost when the peat began to burn. After the peat sample was heated in a muffle furnace at 440°C, portions of the sample were found on the roof and walls of the furnace and coated on the outside of the crucible. This illustrated that some of the peat had been lost, and something in the method had to be changed. It was thought that when the sample began to combust that it happened so quickly and with such force that some of the sample was pushed out of the crucible due to the light weight of the peat. To eliminate this problem, a

new method was formulated. This consisted of the sample being heated at 200°C for an hour, to ash the sample prior to burning it. It was then heated to 440°C for another hour, to burn off the organic material. This primary ashing phase eliminated the problem of losing any part of the sample from the crucible, while achieving the desired result of removing the organic material. The organic and ash contents were then determined using the following equations.

$$\text{OrganicContent} = \frac{m_o}{m_s} \cdot 100\% \quad (\text{A.2})$$

$$\text{AshContent} = 100 - \text{OrganicContent} \quad (\text{A.3})$$

Where m_o is the mass of the organic portion of the peat sample which was removed during combustion. The raw data obtained in the determination of the moisture, ash and organic contents for the Whole and Partial samples can be found in Tables A.1 and A.2, respectively. Six samples were analyzed for each of the two sample types and the numbers in the first row of each table indicate the sample number.

Table A.1 - Raw data for the Whole Sample, in the determination of the moisture, ash and organic contents

	#1	#2	#3	#4	#5	#6
mass of evaporating dish (g)	58.9619	62.9203	58.7762	61.7063	63.1271	59.3664
cover mass (g)	0.78	0.7	0.702	0.489	0.747	0.753
mass of peat + dish (g)	75.4608	77.9388	74.653	78.8028	80.1043	75.5686
mass of dried peat + dish (g)	73.361	76.178	73.5551	76.591	78.235	74.5071
Moisture Content (%)	12.727	11.7244	11.627	12.9369	11.0107	11.3291
mass of ashed peat + dish + cover (g)	61.996	65.2193	61.719	65.216	66.438	62.9188
organic mass (g)	12.145	11.6587	11.79	11.864	12.544	11.5672
Ash Content (%)	15.6545	12.0612	15.9707	20.2941	16.9705	19.4858
Organic Content (%)	84.3455	87.9388	84.0293	79.7059	83.0295	80.5142

Table A.2 - Raw data for Partial Sample, in the determination of the moisture, ash and organic contents

	#1	#2	#3	#4	#5	#6
mass of evaporating dish (g)	63.7131	66.7039	58.2584	62.731	60.1323	58.8879
cover mass (g)	0.712	0.499	0.512	0.503	0.75	0.49
mass of peat + dish (g)	80.6462	84.247	76.6528	79.5647	77.5956	75.7489
mass of dried peat + dish (g)	78.208	81.647	72.22	75.602	73.363	70.65
Moisture Content (%)	14.3992	14.8206	24.0988	23.5403	24.2372	30.2408
mass of ashed peat + dish + cover (g)	69.08	69.599	59.3727	63.9529	61.8198	61.673
organic mass (g)	9.84	12.547	13.3593	12.1521	12.2978	9.467
Ash Content (%)	32.1141	16.0349	4.31384	5.58542	7.05107	19.513
Organic Content (%)	67.8859	83.9651	95.6862	94.4146	92.9489	80.487

A.2 - DESCRIPTION OF COLUMN PACKING

The saturated hydraulic conductivity and the effective porosity were determined using a column apparatus (see Figure A.1). The column was constructed using a 0.1m, thick-walled ABS pipe and was 0.47m (1.5 feet) tall. The bottom of the pipe was sealed using an ABS cap attached using ABS glue. An outflow was constructed at the bottom of the column, using a 5/8" NPT male to 3/8" hose barb nylon fitting. A 0.03m drainage layer, consisting of a fine sand topped with a filter fabric, was placed in the bottom of the column. This layer served to support the peat column and to collect the permeate evenly from the base of the column through the outflow fitting.

Three similar fittings were placed 0.01, 0.06 and 0.11m above the top of the peat to control the water pressure head at the top of the column. The fittings not in use during

Figure A.1 - Column apparatus used in the determination of the saturated hydraulic conductivity and the effective porosity

The peat was initially laid out to dry for at least twenty-four hours to ensure a uniform moisture content before the peat was placed into the columns. The peat previously described as the “Partial Sample” was used as it was thought that the sticks might cause flow to short-circuit down through the column. The column was then packed to the desired bulk density to a height of 0.30m. As peat exhibits large amounts of swelling during the saturation process, a piece of filter fabric was secured on top of the peat to maintain the sample at the desired volume. The dry density was determined assuming the average ambient moisture content of 21.9%, as determined in Section 3.1 (the average for the Partial Sample).

$$\rho_d = \frac{\rho_b}{1 - \theta} = \frac{\rho_b}{1 - 0.219} = \frac{m_s}{V_t} \quad (\text{A.4})$$

Where ρ_d and ρ_b are the dry and bulk density of the peat.

A.3 - SATURATED HYDRAULIC CONDUCTIVITY

The saturated hydraulic conductivity was determined using the procedure described in ASTM D4511. The column apparatus was used for the experiments. This method states that the first step was to saturate that sample for at least twenty-four hours prior to taking any measurements. This was done by placing a constant head on top of the peat using the top overflow and allowing the water to flow down through the column and out through the outflow (see Figure A.2). The next step was to take flow and head measurements such that the saturated hydraulic conductivity could be solved using the following equation.

Figure A.2 - Column apparatus set-up for determination of saturated hydraulic conductivity

$$K_{sat} = -\frac{Q \cdot \Delta l}{A_c \cdot \Delta h} = \frac{Q \cdot \Delta l}{A_c \cdot (h_B - h_A)} \quad (\text{A.5})$$

The head difference (Δh) is the change in the head across the peat column. This is evaluated as the difference between the total head at point B (h_B) and at point A (h_A) (on Figure A.2). The resulting change in head is illustrated as the vertical difference between the water level on top of the column and the level of the outflow tube as illustrated in Figure A.2. Flow measurements were taken, ensuring that the flow rate was constant. The flow rates were determined by measuring the volume of water exiting (V) through the outflow in a recorded time (t) ($Q = V/t$). The length (Δl) and cross-sectional area (A_c) of the peat column remained constant at 0.30m and 0.008m², respectively.

The constant head tests were repeated for two additional head differences for a total of three saturated hydraulic conductivity measurements at each dry density. The values of each of the flow measurements and corresponding head differences for each dry density are presented in Table A.3.

The run numbers with an attached “a”, indicate that the column was packed a second time at the same density. This was done in circumstances where the relationship with density was not as expected. The averages presented in bold font were those which were used in the final relationship between dry density and saturated hydraulic conductivity. The erroneous results were thought to be due to packing problems causing inconsistencies in density within the column.

Table A.3 - Data used in determination of saturated hydraulic conductivity

Run number	Dry Density (kg/m ³)	Head Difference Δh (m)	Flow Q (10 ⁻⁶ m ³ /s)	K _{sat} (cm/s)	average K _{sat} (cm/s)	standard deviation (cm/s)
1	117	0.380	8.0	7.8x10 ⁻⁴	7.9x10 ⁻⁴	1.8x10 ⁻⁵
		0.324	6.8	7.8x10 ⁻⁴		
		0.268	5.9	8.1x10 ⁻⁴		
2	126	0.380	4.1011	4.0x10 ⁻⁴	4.1x10 ⁻⁴	1.2x10 ⁻⁵
		0.329	3.7529	4.2x10 ⁻⁴		
		0.271	3.0509	4.2x10 ⁻⁴		
3	135	0.382	2.6833	2.6x10 ⁻⁴	2.6x10 ⁻⁴	6.0x10 ⁻⁶
		0.322	2.2486	2.6x10 ⁻⁴		
		0.275	1.8497	2.5x10 ⁻⁴		
4	144	0.378	0.6612	6.5x10 ⁻⁵	7.4x10 ⁻⁵	1.1x10 ⁻⁵
		0.325	0.6245	7.1x10 ⁻⁵		
		0.273	0.6382	8.7x10 ⁻⁵		
4a	144	0.377	1.6369	1.6x10 ⁻⁴	1.5x10 ⁻⁴	7.8x10 ⁻⁶
		0.323	1.2818	1.5x10 ⁻⁴		
		0.276	1.1007	1.5x10 ⁻⁴		
5	153	0.380	0.95	9.3x10 ⁻⁵	9.2x10 ⁻⁵	6.3x10 ⁻⁷
		0.324	0.8074	9.2x10 ⁻⁵		
		0.274	0.676	9.1x10 ⁻⁵		
5a	153	0.382	1.2001	1.6x10 ⁻⁴	1.2x10 ⁻⁴	2.1x10 ⁻⁶
		0.322	1.0475	1.2x10 ⁻⁴		
		0.274	0.8698	1.2x10 ⁻⁴		
6	162	0.383	1.0614	1.0x10 ⁻⁴	1.0x10 ⁻⁴	5.3x10 ⁻⁷
		0.324	0.9068	1.0x10 ⁻⁴		
		0.280	0.7781	1.0x10 ⁻⁴		
6a	162	0.378	0.4306	4.2x10 ⁻⁵	4.2x10 ⁻⁵	1.3x10 ⁻⁷
		0.322	0.3673	4.2x10 ⁻⁵		
		0.273	0.3096	4.2x10 ⁻⁵		

A.4 POROSITY

A.4.1 TOTAL POROSITY

The total porosity was calculated from first principles and using a few assumed values. By definition the total porosity is expressed as the ratio of void to total volume.

$$\phi_t = \frac{V_v}{V_t} = \frac{V_t - V_s}{V_t} = 1 - \frac{V_s}{V_t} \quad (\text{A.6})$$

It was desired to be able to determine the value of the total porosity as a function of dry density.

The average and maximum solid densities of five different peats were used in the calculations. The fundamental equation for the solid density (ρ_s) is

$$\rho_s = \frac{m_s}{V_s} \quad (\text{A.7})$$

Substituting equations A.4 and A.7 into equation A.6 gives

$$\phi_t = 1 - \frac{V_s}{V_t} = 1 - \frac{1/V_t}{1/V_s} = 1 - \frac{m_s/V_t}{m_s/V_s} = 1 - \frac{\rho_d}{\rho_s} \quad (\text{A.8})$$

Where ρ_s is the solid density of peat.

A.4.2 EFFECTIVE POROSITY

The effective porosity was also determined from first principles, using the column apparatus described in Section A.2. By definition, the effective porosity was that which contributed to flow and could therefore be evaluated from the flow characteristics. By using the definition of the seepage velocity, the effective porosity could be calculated.

$$v_s = \frac{q}{\phi_e} \quad (\text{A.9})$$

From the literature, a solution of sodium chloride had been found to not adsorb to peat (Karamanev, 1994). It could therefore be used as a tracer to evaluate the complete breakthrough curve. A sodium chloride solution was placed on top of the peat at a known

constant head. The conductivity of the effluent had a linear relationship with the concentration of NaCl in solution. The time at which $C/C_0=0.5$ ($t_{1/2}$), where C is the conductivity, was used to calculate the seepage velocity. The water had a small background conductivity which was considered in the calculations.

$$v_s = \frac{\Delta L}{t_{1/2}} \quad (\text{A.10})$$

Where ΔL is the length of the column, equal to 0.30m. The darcy flux was determined by measuring the flow rate. The effective porosity was then calculated from the following equation.

$$\phi_e = \frac{q \cdot t_{1/2}}{\Delta L} \quad (\text{A.11})$$

The raw data for these calculations can be found in Table A.4.

A.4.3 DEAD SPACE PERCENTAGE

The percentage of dead space was calculated from the definition of primary and effective porosity. The primary porosity consists of all pores and the effective porosity considers only the porosity which contributes to flow. The quantity of dead space with respect to the voids was therefore determined using the following equation.

$$DeadSpace = \frac{V_{v,deadspace}}{V_v} = \frac{\frac{V_v - V_{ve}}{V_t}}{\frac{V_v}{V_t}} = \frac{\phi_t - \phi_e}{\phi_t} \quad (\text{A.12})$$

The raw data and results for each density are presented in Table A.5.

Table A.4 - Raw data used in calculating the effective porosity

#	Dry Density (kg/m ³)	Darcy velocity (10 ⁻² m/s)	Time at C/C ₀ = 0.5 (s)	Seepage velocity (10 ⁻² m/s)	Effective porosity
1-1	117	0.067	425	0.071	0.95
1-2		0.073	392	0.077	0.95
1a-1	117	0.026	929	0.032	0.80
1a-2		0.031	759	0.040	0.78
2-1	126	0.036	739	0.041	0.88
2-2		0.034	877	0.034	0.82
2-3		0.033	556	0.054	0.61
2-4		0.028	968	0.031	0.90
2-5		0.028	957	0.031	0.90
3-1	135	0.020	1282	0.023	0.87
3-2		0.025	1014	0.030	0.84
3-3		0.025	1169	0.026	0.97
3-4		0.013	2099	0.014	0.94
3-5		0.006	3824	0.0078	0.82
3-6		0.006	3824	0.0078	0.83
4-1	144	0.005	4759	0.0063	0.86
4-2		0.007	4396	0.0068	0.96
4-3		0.009	3254	0.0092	0.93
4a-1	144	0.012	1685	0.018	0.69
4a-2		0.011	1944	0.015	0.70
5-1	153	0.007	3063	0.0098	0.74
5-2		0.010	2392	0.013	0.82
5-3		0.013	1917	0.016	0.83
5a-1	153	0.009	2239	0.013	0.67
5a-2		0.010	2431	0.012	0.79
5a-3		0.010	2379	0.013	0.82
6-1	162	0.010	2794	0.011	0.96
6-2		0.010	2848	0.011	0.97
6-3		0.0086	2898	0.010	0.83
6-4		0.0092	2930	0.010	0.90
6-5		0.0079	3498	0.0085	0.92
6a-1	162	0.0035	6192	0.004845	0.73
6a-2		0.0038	5778	0.005192	0.74
6a-3		0.0039	5650	0.00531	0.74

A.5 SHRINKAGE LIMIT

The shrinkage limit is that moisture content below which no further reduction in total volume occurs. The fundamental equation of the shrinkage limit follows:

$$ShrinkageLimit = \omega_{s.L.} = \frac{m_{w.s.L.}}{m_s} \quad (A.21)$$

Table A.5 - Determination of dead space percentage in peat.

Dry Density (kg/m ³)	Effective porosity	Total porosity ($\rho_s=1340\text{kg/m}^3$)	Dead space percentage	Total porosity ($\rho_s=1570\text{kg/m}^3$)	Dead space percentage
117.31	0.95	0.912	-0.045	0.925	-0.030
	0.95		-0.038		-0.024
	0.80		0.13		0.14
	0.78		0.14		0.15
126.33	0.88	0.906	0.023	0.919	0.038
	0.82		0.096		0.11
	0.61		0.32		0.34
	0.90		0.0027		0.018
	0.90		0.0070		0.022
135.36	0.87	0.899	0.028	0.913785	0.043
	0.84		0.061		0.077
	0.97		-0.082		-0.065
	0.94		-0.043		-0.026
	0.82		0.083		0.098
	0.83		0.080		0.095
144.38	0.86	0.892	0.037	0.908	0.054
	0.69		0.22		0.23
	0.96		-0.07		-0.053
	0.70		0.22		0.23
	0.93		-0.043		-0.025
153.40	0.738	0.886	0.17	0.902	0.18
	0.666		0.25		0.26
	0.824		0.069		0.087
	0.789		0.11		0.13
	0.828		0.065		0.083
	0.815		0.080		0.097
162.43	0.963	0.879	-0.095	0.897	-0.074
	0.970		-0.14		-0.082
	0.831		0.054		0.073
	0.899		-0.023		-0.0029
	0.923		-0.050		-0.029
	0.726		0.17		0.19
	0.736		0.16		0.18
	0.741		0.16		0.17
Including all data points		Average Standard Deviation	0.063		0.079
			0.11		0.10
Including only positive values		Average Standard Deviation	0.11		0.13
			0.082		0.081

Where $m_{w,S.L.}$ and $\omega_{S.L.}$ are the mass of water and moisture content at the shrinkage limit, respectively.

To determine the value of the shrinkage limit for peat, traditional methods were not possible due to the nature of the peat. Instead the following procedure was followed to obtain the approximate the value. A large diameter tin can was cut such that it had a height of approximately 0.025m. Small holes were drilled into the bottom of the can and a filter fabric was placed in the bottom providing complete coverage. The filter fabric served to maintain the peat within the can such that it did not exit via the drainage holes. A sample of ambient peat was placed into the can such that it had uniform thickness. The peat was then saturated by continuously pumping water for at least twenty four hours onto the top of the sample and allowing the water to filter through and out the holes at the bottom. The dimensions of the sample were taken to determine the total saturated volume and the mass was measured. Again, assuming that the solid density of peat was 1340kg/m^3 and 1570kg/m^3 , the solid and water volume, and mass of solids were calculated by solving the following two equations simultaneously:

$$m_t = V_s \cdot \rho_s + V_w \cdot \rho_w \quad (\text{A.22})$$

$$V_t = V_s + V_w \quad (\text{A.23})$$

where m_t is the total mass of the peat sample.

The sample was then allowed to dry past the shrinkage limit, and the new volume was measured. By definition, this volume will not decrease upon any further reduction in moisture content lower than the shrinkage limit. The mass and volume of the solid portions of the sample will remain the same throughout the drying process. The volume

of water, at the shrinkage limit, was calculated as the difference between the new total volume and the solid volume. The shrinkage limit was then calculated from

$$\text{ShrinkageLimit} = \frac{V_{w,SL} \cdot \rho_w}{V_s \cdot \rho_s} \quad (\text{A.19})$$

where $V_{w,SL}$ is the water volume at the shrinkage limit.

All of the raw data and intermediate calculations are presented in Table A.6.

Table A.6 - Raw data for determination of shrinkage limit

	Sample 1		Sample 2	
<i>Before saturation:</i>				
sample volume (10^{-6}m^3)	113		81.2	
mass of ambient peat (kg)	0.0144		0.0094	
density (kg/m^3)	127		116	
dry density (kg/m^3)	162		148	
<i>After saturation:</i>				
peat solid density (kg/m^3)	1340	1570	1340	1570
mass of saturated peat (kg)	0.1346		0.0855	
sample volume (10^{-6}m^3)	128		81.2	
water volume (10^{-6}m^3)	109	117	68.4	73.5
solid volume (10^{-6}m^3)	18.7	11.2	12.8	7.62
<i>After drying past shrinkage limit moisture content:</i>				
sample volume (10^{-6}m^3)	124		78.3	
water volume (10^{-6}m^3)	106	113	61.2	66.3
Shrinkage limit	4.2	6.5	3.6	5.5

A.6 PRESSURE-SATURATION RELATIONSHIP

The pressure-saturation relationship was determined by the pressure cell method. A glass flask fitted with a fine porous plate was used for this procedure (see Figure A.3). The flask was initially dried to remove all moisture from the pores in the porous plate. A long piece of C-Flex tubing was attached to a 50mL buret. The entire length of tubing and

Figure A.3 - Pressure-saturation apparatus

the buret were filled with water ensuring that all air bubbles and pockets were removed. The stop cock of the buret was closed to maintain the level of water in the buret. The free end of the tubing was attached to the bottom spout of the flask. The buret was raised above the flask and the stop cock was opened to cause a hydraulic gradient to drive water into the bottom chamber of the flask. The flask was kept as parallel as possible such that the water level would raise evenly in the chamber below the porous plate. Once the water level was close to the porous plate the gradient was increased so that the plate would be saturated quickly in attempts to push all air bubbles out of the plate. Water was allowed to flow up through the plate until an approximate depth of 0.01m of water sat on top of the plate. The water was poured out of the top chamber of the flask and a peat sample was immediately placed in the flask. The peat was then saturated by passing a minimum of one pore volume of water through it. Once the saturation phase was completed, a piece of plastic was secured to the top of the flask and a piece of parafilm was placed on top of the buret. A very small pin hole was made in each of these covers. The purpose of the covers was to minimize losses due to evaporation via these areas and the hole was to maintain atmospheric pressure. The buret provided a means of placing a pressure on the peat sample which was transferred via the porous plate. This was why it was important to ensure that all air was removed from the porous plate, as described earlier.

In a saturated system with no trapped air, the sample will be 100% saturated at zero capillary head. It was assumed that the middle of the peat layer was representative of the whole sample and the capillary head measurements were taken from this point. The capillary head was defined in Chapter 3, Section 3.6.1, and was measured as the vertical

difference between the middle of the peat sample to the water level in the buret (see Figure A.3). Therefore, at the first point of 100% saturation, the level of water in the buret was at the same height as the midpoint of the peat. From this situation the main drainage branch could be determined by draining the system. The initial water level in the buret was recorded as a starting point. To obtain another point on the curve, a negative pressure was exerted on the bottom of the sample by lowering the buret to level₁ as indicated in Figure A.3. This created a pressure gradient, which caused water to drain out of the peat sample until a state of equilibrium between saturation of the peat and capillary head was achieved (level₂). The new water level was recorded and the amount of water removed (ΔV) was calculated as the difference between these two levels. Evaporation may be an issue in these measurements as the system sits for several hours while coming to equilibrium. This is discussed in the next section. The new capillary head (h_c) was measured as shown in Figure A.3. The lowering of the buret was repeated numerous times until no further water drained out of the system with an increase in negative pressure.

A.6.1 RATE OF EVAPORATION

The entire experiment (primary drainage and wetting curve) lasted approximately 3 weeks. To determine if evaporative losses were significant over this period a control was established. From this control a rate of evaporation was determined. The observed losses due to evaporation are displayed in Table A.8.

In the calculations of the saturations, the quantity of water removed or added to the system was determined for each change in pressure. During the time that the system reached equilibrium, water may have evaporated. The total evaporative losses and the overall rate of evaporation are located in the second last row of Table A.8. Since the evaporative losses were so small in comparison to the volume changes measured during the experiment, evaporation was neglected.

Table A.7 - Rate of evaporation from six feet of C-Flex tubing

Date and time	Change in time (hours)	Buret level (mL)	Change in water level (mL)	Rate (mL/ft.hr)
Jan 21 - 9:30am		31.1		
Jan 22 - 8:25am	22.92	31.2	0.1	0.000727
Jan 26 - 8:55am	95.5	31.35	0.15	0.000262
Jan 27 - 10:34am	26.65	31.35	0	0
Jan 28 - 7:52am	20.3	31.4	0.05	0.000177
Jan 30 - 8:20am	48.46667	31.5	0.1	0.000344
Feb 2 - 7:29am	71.15	31.6	0.1	0.000234
Feb 3 - 8:32am	25.05	31.7	0.1	0.000665
Feb 4 - 8:00am	23.46667	31.7	0	0
Feb 6 - 8:25am	48.41667	31.8	0.1	0.000232
Feb 9 - 8:17am	71.86667	31.9	0.1	0.000232
Feb 10 - 8:12am	23.91667	31.9	0	0
Feb 11 - 8:12am	24	32.0	0.1	0.000348
overall	501.7		0.9	0.000299
average				0.000352
standard deviation				0.00019

A.6.2 CALCULATIONS

Once the previously described procedure was completed a data set with capillary heads and corresponding buret levels was attained. However, a relationship between capillary head and apparent saturation was desired. Therefore, the apparent saturations at each of the capillary heads was calculated.

The first point on the main drainage curve was known to be at 100% saturation. The total volume of the peat sample was known and the total porosity was assumed. From this data the initial volume of water within the sample could be determined.

$$V_{w,init} = \phi_t \cdot V_t \quad (\text{A.20})$$

Where $V_{w,init}$ is the initial volume of water within the saturated sample of peat.

The volume of water within the sample could then be determined at each of the other points using the data on the buret levels.

$$V_{w,k+1} = V_{w,k} - (level_1 - level_2) \quad (\text{A.21})$$

Where $V_{w,k+1}$ and $V_{w,k}$ are the volumes of water at one point and the previous point, respectively. The buret level at the previous point and the point under consideration are represented by $level_1$ and $level_2$, respectively.

The water contents and saturations could then be determined at each of the points. The raw data each of the three data sets are presented in Tables A.9 through A.11.

Table A.8 - Pressure-saturation raw data for data set #1

$V = 242.5 \text{ cm}^3$ $m_{\text{cruc}} = 138.8 \text{ g}$
 porosity = 0.87 $m_{\text{dry+cruc}} = 167.6 \text{ g}$
 $V_{w,i} = 210.975$ $\text{dry density} = 118.7629 \text{ kg/m}^3$
 $S_r = 0.17692$

4.994259 4.262616

Drainage Curve							Wetting Curve						
buret level (mL)	head dif	Water Volume (cm^3)	Water Content	Actual Sat.	Effective Sat.	Apparent Sat.	buret level (mL)	head dif	Water Volume	Water Content	Actual Sat.	Effective Sat.	Apparent Sat.
43.9							43	173.8	37.725	0.155567	0.178813	0.0023035	0.0022185
43.7	-2.4	210.975	0.87	1	1	1	24						
22.4	-0.5	210.975	0.87	1	1	1	26	155.2	39.725	0.163814	0.188292	0.0138209	0.0133111
14.05	0.7	202.625	0.835567	0.960422	0.951915	0.951915	28.2	131.6	41.925	0.172887	0.19872	0.0264901	0.025513
46.2							29.6	115.8	43.325	0.17866	0.205356	0.0345523	0.0332779
34.7	3.2	191.125	0.788144	0.905913	0.88569	0.88569	31.4	96.3	45.125	0.186082	0.213888	0.0449179	0.0432612
7.1	8.9	163.525	0.67433	0.775092	0.726749	0.726749	34.5	79.8	48.225	0.198866	0.228582	0.0627699	0.0604548
47							13.6						
30.2	13.5	146.725	0.605052	0.695462	0.630003	0.630003	16	66.6	50.625	0.208763	0.239957	0.0765908	0.0737659
15.9	15.8	132.425	0.546082	0.627681	0.547653	0.547653	19	64	53.625	0.221134	0.254177	0.093867	0.0904049
6.5	17.5	123.025	0.50732	0.583126	0.493521	0.493521	1.1						
48.3							7.1	50.9	59.625	0.245876	0.282616	0.1284192	0.1236828
30.2	20.3	104.925	0.43268	0.497334	0.389289	0.389289	13.2	37.9	65.725	0.271031	0.31153	0.1635474	0.1575153
16.4	27.1	91.125	0.375773	0.431923	0.309819	0.309819	21.8	24.6	74.325	0.306495	0.352293	0.2130723	0.2052135
47							2.7						
34.1	33.3	78.225	0.322577	0.370779	0.235531	0.235531	15.1	19.2	86.725	0.357629	0.411068	0.2844803	0.2739878
22.4	44.6	66.525	0.27433	0.315322	0.168154	0.168154	18.3	18.2	89.925	0.370825	0.426235	0.3029081	0.291736
47.2							1.1						
38.7	59.2	58.025	0.239278	0.275033	0.119205	0.119205	12	11.4	100.825	0.415773	0.4779	0.3656781	0.3521908
33.25	73.6	52.575	0.216804	0.2492	0.08782	0.08782	22.4	8.8	111.225	0.45866	0.527195	0.4255687	0.4098724
28.1	96.7	47.425	0.195567	0.22479	0.058163	0.058163	35.1	6.9	123.925	0.511031	0.587392	0.4987043	0.4803106
25	115.7	44.325	0.182784	0.210096	0.040311	0.040311	47.1	5.9	135.925	0.560515	0.644271	0.5678088	0.5468663
49.6							0.6						
47.4	131.8	42.125	0.173711	0.199668	0.027642	0.027642	21.4	6.2	156.725	0.646289	0.742861	0.68759	0.6622296
45.5	141	40.225	0.165876	0.190662	0.0167	0.0167	32.6	3.7	167.925	0.692474	0.795947	0.7520875	0.7243483
44	169.1	38.725	0.159691	0.183553	0.008062	0.008062	44.9	2.9	180.225	0.743196	0.854248	0.8229197	0.7925679
42.6	177	37.325	0.153918	0.176917	0	0	1						
42.6	182.8	37.325	0.153918	0.176917	0	0	15.9	2.1	195.125	0.804639	0.924873	0.9087244	0.875208
							32.9	0.4	212.125	0.874742	1.005451	1.0066225	0.9694953
							0.4						
							5.9	0	217.625	0.897423	1.03152	1.0382954	1

Table A.9 - Pressure-saturation raw data for data set #2

$m_{\text{peat}} = 28.5 \text{ g}$ dry density 0.1188433 g/cm^3 maxtrap = 0.1410993
 $V = 197.21348 \text{ cm}^3$ $m_{\text{cruc}} = 159.7 \text{ g}$
 porosity = 0.85 $m_{\text{dry+cruc}} = 178.7 \text{ g}$
 $V_{\text{wi}} = 167.63146 \text{ cm}^3$ dry density = 0.0963423 g/cm^3
 $S_r = 0.2728095$ 4.2288557 3.6093418

Drainage Curve						Wetting curve						
buret level (mL)	delta h (cm)	Water Volume	Moisture Content	Saturation	Apparent Saturation	buret level	head dif	volume added	moisture content	actual saturation	Effective Saturation	Apparent Saturation
45.1	-0.7	167.63146	0.85	1	1	30.1						
43.35	1	165.88146	0.8411264	0.9895604	0.985644	30.7	178.6	46.331459	0.2349305	0.2763888	0.0049221	0.0057307
36.3	2.9	158.83146	0.8053783	0.9475039	0.9278097	31.35	166.3	46.981459	0.2382264	0.2802664	0.0102543	0.0119389
28.3	5.5	150.83146	0.7648131	0.8997801	0.8621821	31.7	160.8	47.331459	0.2400011	0.2823543	0.0131255	0.0152818
4.7	12.3	127.23146	0.6451459	0.7589951	0.6685808	32.55	151.8	48.181459	0.2443112	0.2874249	0.0200984	0.0234002
19.5						33.1	137.1	48.731459	0.2471	0.2907059	0.0246103	0.0286533
14.5	16.9	122.23146	0.6197926	0.7291678	0.6275636	33.9	121.7	49.531459	0.2511566	0.2954783	0.0311731	0.0362942
6.4	18.9	114.13146	0.5787204	0.6808475	0.5611157	35	108.1	50.631459	0.2567343	0.3020403	0.0401969	0.0468004
23.9						36.4	83.8	52.031459	0.2638332	0.310392	0.0516817	0.0601719
8.6	23.4	98.831459	0.5011395	0.5895758	0.435603	38.1	70.3	53.731459	0.2724533	0.3205333	0.0656276	0.0764088
26.3						40.3	55.3	55.931459	0.2836087	0.3336573	0.0836751	0.0974212
14.6	29	87.131459	0.4418129	0.5197799	0.3396226	24.2						
33.3						26.55	40.2	58.281459	0.2955247	0.3476761	0.1029532	0.1198663
28	31.7	81.831459	0.4149385	0.4881629	0.2961444	12.7						
20.1	37.6	73.931459	0.3748804	0.4410357	0.2313372	16.3	28.9	61.881459	0.3137791	0.3691518	0.1324856	0.1542502
14.4	45.1	68.231459	0.3459777	0.4070325	0.1845775	1.4						
9.7	53.9	63.531459	0.3221456	0.3789948	0.1460213	9.1	19.1	69.581459	0.352823	0.4150859	0.1956522	0.2277937
6.6	62.1	60.431459	0.3064266	0.3605019	0.1205906	17.2	12.3	77.681459	0.3938953	0.4634062	0.2621001	0.3051576
3.8	79.9	57.631459	0.2922288	0.3437986	0.097621	28.5	8.8	88.981459	0.4511936	0.530816	0.354799	0.413085
47.9						37	7.5	97.481459	0.4942941	0.5815225	0.4245283	0.4942693
45.15	87.7	54.881459	0.2782845	0.3273936	0.0750615	42.5	6.7	102.98146	0.5221827	0.6143325	0.4696473	0.5468004
43.4	99.7	53.131459	0.2694109	0.316954	0.0607055	18.9						
42	112.5	51.731459	0.262312	0.3086023	0.0492207	22.7	4.7	106.78146	0.5414511	0.6370013	0.5008203	0.5830946
40.7	126.15	50.431459	0.2557201	0.3008472	0.0385562	30.8	4.6	114.88146	0.5825234	0.6853216	0.5672683	0.6604585
39.9	139.8	49.631459	0.2516636	0.2960749	0.0319934	38.6	3.8	122.68146	0.6220744	0.7318522	0.6312551	0.734957
39	154	48.731459	0.2471	0.2907059	0.0246103	43.2	2.8	127.28146	0.6453994	0.7592934	0.668991	0.7788921
38.4	161.1	48.131459	0.2440577	0.2871267	0.0196883	16						
38.3	165.8	48.031459	0.2435506	0.2865301	0.0188679	22.5	1.3	133.78146	0.6783586	0.7980689	0.7223134	0.8409742
37.9	171.8	47.631459	0.2415223	0.2841439	0.0155865	28.1	1	139.38146	0.7067542	0.8314755	0.7682527	0.8944604
37.5	177.5	47.231459	0.2394941	0.2817577	0.0123052	34.4	0.9	145.68146	0.7386993	0.869058	0.8199344	0.9546323
48.7						2.3						
48	190.8	46.531459	0.2359446	0.2775819	0.0065628	7.05	0	150.43146	0.7627849	0.897394	0.8589007	1
47.6	194	46.131459	0.2339164	0.2751957	0.0032814							
47.2	198.6	45.731459	0.2318881	0.2728095	0							

Table A.10 - Pressure-saturation raw data for data set #3

$V_{\text{total}} = 316.1 \text{ cm}^3$ dry density = 0.162 g/cm^3
 porosity = 0.85 md 51.2082 g
 $V_{w,i} = 268.685 \text{ cm}^3$ mamb = 62.422796 g
 $S_r = 0.327837$

water content = 4.228856 3.609342

Drainage Curve							Wetting Curve						
buret level (mL)	head dif	Water Volume	Water Content	Saturation	Effective sat	Apparent sat	buret level (mL)	head dif	Water Volume	Water Content	Actual sat	Effective sat	Apparent sat
44.5							45.9	242.8	88.785	0.2808763	0.3304427	0.003876	0.0036784
41.7	-2.8	268.685	0.85	1	1	1	26.6						
24.6	-0.2	268.685	0.85	1	1	1	26.8	220.1	88.985	0.281509	0.3311871	0.0049834	0.0047294
8.4	3.8	252.485	0.7987504	0.9397063	0.910299	0.910299	28.9	203.1	91.085	0.2881525	0.3390029	0.0166113	0.0157646
45.5							30.9	181.3	93.085	0.2944796	0.3464466	0.0276855	0.0262743
34.2	7.7	241.185	0.7630022	0.8976497	0.8477298	0.84773							
5.5	14.2	212.485	0.6722082	0.7908331	0.6888151	0.688815	32.6	154.4	94.785	0.2998576	0.3527737	0.0370986	0.0352076
48.5							35.3	144.3	97.485	0.3083992	0.3628226	0.0520487	0.0493957
33.3	17.8	197.285	0.6241221	0.7342613	0.6046512	0.604651	10						
17.6	24	181.585	0.5744543	0.6758286	0.5177187	0.517719	13.2	120.3	100.685	0.3185226	0.3747325	0.0697674	0.0662112
47.2							17.9	53.6	105.385	0.3333913	0.3922251	0.0957918	0.0909091
30.2	24	164.585	0.5206738	0.6125575	0.423588	0.423588	22	35.6	109.485	0.3463619	0.4074846	0.1184939	0.112454
17.2	30.7	151.585	0.4795476	0.5641737	0.3516058	0.351606	33.2	28.1	120.685	0.3817937	0.4491691	0.1805094	0.1713085
5.2	36.5	139.585	0.4415849	0.5195117	0.2851606	0.285161	2.8						
48.4							20.4	22.6	138.285	0.4374723	0.5146733	0.2779623	0.263794
30	54.2	121.185	0.3833755	0.45103	0.183278	0.183278	29.4	14.1	147.285	0.4659443	0.5481698	0.3277962	0.3110878
21.8	65.2	112.985	0.3574344	0.420511	0.1378738	0.137874	45.2	10.4	163.085	0.5159285	0.6069747	0.4152824	0.3941146
14.5	86.3	105.685	0.3343404	0.3933416	0.0974529	0.097453	1.3						
9.9	104.65	101.085	0.319788	0.3762212	0.0719823	0.071982	19.9	7.2	181.685	0.5747706	0.6762008	0.5182724	0.491855
46.4							38.1	5.7	199.885	0.6323474	0.7439381	0.6190476	0.5874934
43.4	122.05	98.085	0.3102974	0.3650557	0.055371	0.055371	1						
39.9	139.35	94.585	0.2992249	0.3520293	0.0359911	0.035991	35.9	3	234.785	0.7427555	0.8738299	0.8122924	0.7708881
38.6	166.7	93.285	0.2951123	0.3471909	0.0287929	0.028793	48.5	2	247.385	0.7826163	0.920725	0.8820598	0.8370993
36.2	179.3	90.885	0.2875198	0.3382586	0.0155039	0.015504	0						
35.6	195.8	90.285	0.2856216	0.3360255	0.0121816	0.012182	20.9	0.1	268.285	0.8487346	0.9985113	0.9977852	0.9469259
34.3	200.8	88.985	0.281509	0.3311871	0.0049834	0.004983	31	-0.6	278.385	0.8806865	1.0361018	1.0537099	1
47.5													
47.5	217.1	88.985	0.281509	0.3311871	0.0049834	0.004983							
47.2	222.7	88.685	0.2805599	0.3300705	0.0033223	0.003322							
46.85	231.9	88.335	0.2794527	0.3287679	0.0013843	0.001384							
46.85	236.7	88.335	0.2794527	0.3287679	0.0013843	0.001384							
46.6	242.3	88.085	0.2786618	0.3278374	0	0							
46.6	249	88.085	0.2786618	0.3278374	0	0							

APPENDIX B - FIELD STUDY

B.1 - FIELD DENSITY

The density of the peat in the field was required to determine the density dependent hydraulic parameters at different depths of the peat filter. A soil sampler was designed to take relatively undisturbed samples of peat from which the dry density could be determined (see Figure B.1). A stainless steel tube was fitted with a removable end and 'O' Ring seal. This end could be fit into the tube and held in place by a screw. The other end of the tube was sharpened such that it would easily cut through the peat material. The capped end was attached to a drill and rotated rapidly while being pushed into the peat. The tube was then withdrawn while rotating in the opposite direction. The end cap was removed and a plunger was used to push the peat sample out of the tube. The volume of the sample was measured and the dry mass was determined by drying the sample (Table B.1). The dry bulk density was then calculated as the ratio of the dry mass to the total volume.

$$V_{sample} = L_s \cdot \left(\frac{d_s}{2} \right)^2 \cdot \pi \quad (B.1)$$

Where L_s and d_s are the length and diameter of the peat sample taken from the filter at the different depths.

Figure B-1 - Schematic of soil sampler used to obtain peat samples

$$\rho_d = \frac{m_d}{V_t} \quad (\text{B.2})$$

Table B.1 - Raw data obtained for peat dry bulk density

Elevation (m)	Length of Sample (m)	Volume of Sample (cm ³)	Dry Mass of Sample (kg)	Dry Bulk Density (kg/m ³)
2.00	0.059	27.73x10 ⁻⁶	3.596x10 ⁻³	129
1.79	0.066	31.02x10 ⁻⁶	4.5505x10 ⁻³	146
1.57	0.06	28.2x10 ⁻⁶	4.0621x10 ⁻³	144
1.40	0.056	26.32x10 ⁻⁶	4.104x10 ⁻³	155
1.22	0.061	28.67x10 ⁻⁶	5.115x10 ⁻³	178
1.04	0.064	30.08x10 ⁻⁶	5.1205x10 ⁻³	169

B.2 - TEMPERATURE DATA

The temperatures were recorded at 2 hour intervals at selected elevations. The results of five of those elevations were presented in Chapter 4, and will not be re-shown here. The results of the other five locations are shown in Figure B.2. Three other elevations (0.56m, 0.83m and 1.34m) were monitored and the in box and outdoor temperatures were also observed.

B.3 - DATA OBTAINED FROM FIELD PULSES

A total of seven pulses of STE to the peat filter were monitored. The pulses are numbered from one to eight, excluding three and are shown in Figures B.3 to B.23. The results from each of these pulses are shown in the following pages. The data was presented in three different ways to illustrate the results. The first was a graph of the capillary head versus time at the six different locations within the vertical column. There is one graph for both the 0.30m and 0.60m tensiometer columns. The second

Figure B.2 - Temperature measurements at different elevations over time

Figure B.3 -Capillary head versus time for **Pulse 1**. Both results from **A)** 0.3m tensiometers and **B)** 0.6m tensiometers are shown. (Oct. 7, 1997)

Figure B.5 - Elevation versus total head at selected times for **Pulse 1**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 7, 1997)

Figure B.6 - Capillary head versus time for **Pulse 2**. Both results from **A)** 0.3m tensiometers and **B)** 0.6m tensiometers are shown. (Oct. 7, 1997)

B

Figure B.9 - Capillary head versus time for **Pulse 4**. Both results from **A**) 0.3m tensiometers and **B**) 0.6m tensiometers are shown. (Oct. 21, 1997)

Figure B.10 - Elevation versus pressure head at selected times for **Pulse 4**. Both the results of **A) 0.3m** and **B) 0.6m** tensiometers are shown. (Oct. 21, 1997)

Elevation (cm)	t = 0	t = 5 min	t = 10min	t = 50min	t = 90min
129.09	90.57065	117.3886	115.2575	97.30867	93.40241
111.09	76.57701	77.0982	79.31024	85.79256	84.18616
93.08999	61.73897	62.0176	62.3184	63.47198	69.83305
75.08999	57.88961	57.93598	57.85087	58.51824	58.98193
57.08999	65.23889	65.22927	65.15893	65.67717	65.87196
1.44	0.905706	1.173886	1.152575	0.973087	0.934024
1.26	0.76577	0.770982	0.793102	0.857926	0.841862
1.08	0.61739	0.620176	0.623184	0.63472	0.69833
0.9	0.578896	0.57936	0.578509	0.585182	0.589819
0.72	0.652389	0.652293	0.651589	0.656772	0.65872

(cm)	t = 0	t = 5 min	t = 10min	t = 50min	t = 90min
142	122.3456	122.9414	122.5413	123.8839	125.205
120	82.3813	82.89627	84.21358	98.04529	96.30007
103	70.93594	70.99887	71.12899	75.34084	76.63245
85	74.22522	74.27489	74.26644	74.75146	75.1583
67	71.59312	71.61698	71.83166	72.51097	73.14983
1.42	1.223456	1.229414	1.225413	1.238839	1.25205
1.2	0.823813	0.828963	0.842136	0.980453	0.963001
1.03	0.709359	0.709989	0.71129	0.753408	0.766324
0.85	0.742252	0.742749	0.742664	0.747515	0.751583
0.67	0.715931	0.71617	0.718317	0.72511	0.731498

Figure B.11 - Elevation versus total head at selected times for **Pulse 3**. Both the results of **A)** 30cm and **B)** 60cm tensiometers are shown.

Figure B.13 - Elevation versus pressure head at selected times for **Pulse 5**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 21, 1997)

Figure B.11 - Elevation versus total head at selected times for **Pulse 4**. Both the results of **A) 0.3m** and **B) 0.6m** tensiometers are shown. (Oct. 21, 1997)

Figure B.15 - Capillary head versus time for **Pulse 6**. Both results from **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.16 - Elevation versus pressure head at selected times for **Pulse 6**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.19 - Elevation versus pressure head at selected times for **Pulse 7**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.17 - Elevation versus total head at selected times for **Pulse 6**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.21 - Capillary head versus time for **Pulse 8**. Both results from **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.22 - Elevation versus pressure head at selected times for **Pulse 8**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

Figure B.20 - Elevation versus total head at selected times for **Pulse 7**. Both the results of **A)** 0.3m and **B)** 0.6m tensiometers are shown. (Oct. 24, 1997)

representation was a graph of elevation versus pressure head at selected times. Again there were two separate graphs showing the results for the two different lengths of tensiometers. The last method was a graph of elevation versus total head. In the calculation of the total heads for these graphs, the bottom of the sand layer, as shown on Figure 4.2, was assumed to be the datum.

APPENDIX C - SAMPLE MODEL INPUT FILE

2 45 1 **grid size, number of grids, print every x grids**
0.005 18800 1000 **time step size, number of time steps, print every t time steps**

5 **number of soil types**

K_{sat}	ϕ_e	α_d	α_w	n	θ_r	max. trapped air	
0.5	0.86459	0.039111	0.20697	1.54823	0.27	0.302501	1
0.125715	0.863854	0.039111	0.20697	1.54823	0.27	0.302501	2
0.125715	0.862528	0.079111	0.20697	1.54823	0.27	0.302501	3
0.036592	0.861203	0.079111	0.20697	1.54823	0.27	0.302501	4
0.01	0.861203	0.079111	0.20697	1.54823	0.27	0.302501	5

1 1 1 1 1 1 1 1 1 2 **indicates what soil type is in each grid**
2 2 2 2 2 2 2 2 3 3
3 3 3 3 3 3 3 4 4 4
4 4 4 4 4 4 5 5 5 5
5 5 5 5 5

45.74998	43.82347	41.89696	40.65103	39.97328	
39.29554	38.6178	37.94006	37.26232	36.58457	
35.90683	35.22909	34.26582	33.06417	31.86252	
30.66087	29.45922	28.25757	27.05592	25.85427	
24.65262	23.58817	22.63827	21.68836	20.73846	
19.78856	18.83865	17.88875	16.93885	15.98895	
15.69926	15.96078	16.2223	16.48383	16.74535	
17.00687	17.26839	17.52991	17.79144	14.13194	
7.198862	0.265782	-6.6673	-13.6004	-20.5335	

indicates initial total head for each grid

2 **number of stress periods**
1600 **1st stress period is valid from 0 to 1600**
2 0 0 0 0 0 0 0 0 0 **indicates BCs for each grid for 1st stress period -**
0 0 0 0 0 0 0 0 0 0 **0 - no BC**
0 0 0 0 0 0 0 0 0 0 **1 - specified head BC**
0 0 0 0 0 0 0 0 0 0 **2 - specified flow BC**
0 0 0 0 1
0.153 -24 **specifies values (in order) for each non-zero BC**

18801 **2nd time step is valid from 1601 to 18801**
2 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0
0 0 0 0 1
0 -24

Those values not in bold are those which are to be included in the input file. The bold font text is an explanation of the file and should NOT be included in the file.

APPENDIX D - MODEL RESULTS

Modeling peat between drainage pipe and top of gravel
Peat considered as one soil type

#	Hydraulic parameters							Comments
	K_{sat} (cm/min)	ϕ_e	α_d	α_w	n	θ_r	maximum trapped air	
111	1	0.862	0.07911	0.207	2.1482	0.17	0.202501	too much drainage, small peak
113	0.568205							less of a peak than 111
114	0.025971							less of a peak than 113
115	0.568205	0.8						larger peak than 113
116	0.025971							slightly larger peak than 116
117	0.568205	0.9						less peak than 115 and 113
118	0.025971							less peak than 116
119			0.1					not much different from 118
120			0.2					not much different from 118
121	0.568205	0.8	0.1					less of a peak than 115
122			0.2					less of a peak than 121
123			0.04					more peak than 122 and more drainage
124			0.08	0.15				less peak than 115
125				0.25				more peak and less drainage than 115
126					1.5			bigger peak than 125
127					6			much lower peak than 125
128					1.5	0.05		smaller peak than 126, bit more drainage
129						0.3		larger peak than 126
130							0	smaller peak than 129
131							0.3	bigger peak than 129

The following pages show the results from the model runs in which the peat was divided into five separate layers. Only the peat between the distribution pipe and the top of the gravel was considered. The final three columns show the mean error, mean absolute error and root mean square for each of the model runs.

[illegible]

#	#1					#2					#3					#4					#5											
	Ksat	por.	ad	aw	n	heta	max	Ksat	por.	ad	aw	n	heta	max	Ksat	por.	ad	aw	n	heta	max	Ksat	por.	ad	aw	n	heta	max	ME	MAERMS		
166	0.5	0.8	0.04	0.21	1.4	0.17	0.3	0.13	0.88	0.04	0.21	2.05	0.17	0.2	0.13	0.88	0.08	0.25	2.15	0.27	0.2	0.01	0.86	0.08	0.25	2.15	0.27	0.2	2.58	2.84	4.38	
167																													2.64	2.85	4.38	
168			0.08							0.08																			4.18	4.82	7.89	
169			0.06							0.06																			3.57	4.18	6.17	
170																													3.54	4	5.88	
171	1																												3.35	3.58	6.32	
172											0.25																		3.34	3.61	5.34	
173						0.27																							3.13	3.44	5	
174	0.5		0.04	0.25	1.5		0.2	0.13		0.04		1.5	0.27		0.13				1.5									2.13	2.43	4.04		
175																													2.25	2.54	4.27	
176					2.2		0.3													1.8								2.17	2.57	4.21		
177												2.2								1.5								2.42	2.74	4.35		
178												1																	1.1	3.08	5.21	
179												1.8																	2.35	2.71	4.3	
180					1.5							1.5																	1.57	2.32	3.77	
181	change time of pulse to 8min																												-0.75	4.14	5.6	
182	changed flow to half																															
183	1																												1.87	2.49	3.93	
184			0.21																										1.5	2.77	4.07	
185																													1.88	2.47	3.98	
186																													1.83	2.5	4	
187																													1.81	2.46	3.88	
188																													1.81	2.47	4	
189	0.5				2		0.3																						1.78	2.74	4.14	
190																													2.28	2.61	4.34	
191																													0.3	1.53	2.62	3.87
192																													1.41	2.82	3.89	
193																													1.87	2.88	4.33	
194																													4.07	5.44	6.52	
195	1																												1.25	2.71	4.03	
196	0.75																												1.03	2.85	4.17	
197	0.25																												1.3	4.33	6.61	
198	0.5																												1.31	3.19	4.5	
199																													1.14	2.74	4	
200																													1.21	2.72	3.93	
																													1.08	3.21	4.65	

[illegible]

IMAGE EVALUATION TEST TARGET (QA-3)

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